BUFFALO CROSSING BRIDGE
APACHE-SITGREAVES NATIONAL FOREST
GREENLEE COUNTY, AZ
AZ FS 24(1)

FINAL Geotechnical Report
Report No. AZ-FX-0024-20-01

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 study and provides recommendations for the replacement of the Buffalo Crossing Bridge in Greenlee County, Arizona. The project proposes to replace the existing Buffalo Crossing Bridge, which is located within the Apache-Sitgreaves National Forest. A project vicinity map is presented on Plate 1 of this report.

The Buffalo Crossing Bridge was constructed in 1962 and designed by 1957 AASHTO Bridge Specifications. The superstructure consists of a cast-in-place concrete deck with a haunched pier section. The substructure consists of vertical concrete cantilever abutment walls on spread footings extending to approximately 17 feet deep as indicated on the as-built drawings. Concrete wingwalls are present and rest on spread footings. Solid concrete wall piers are also founded on spread footings at approximately 19.5 and 17 feet for piers 1 & 2, respectively. All foundations are noted as founded on rock and have been reported as stable according to the most recent inspection report. This inspection report, dated December 2018, stated that the bridge did not meet the current load requirements. The outdated bridge specifications, as noted earlier, also did not include seismic design requirements.

The bridge has seen increased logging truck traffic in recent years and the capacity has deteriorated due to both the loading and age. The Arizona Department of Transportation (ADOT) announced a permanent rule in March 2019 that increased the allowable weight limits from 80,000 pounds to 90,800 pounds on some state highways. This rule applied to the National Forest System as well and caused a number of bridges to be rated poor or deficient. Therefore, replacement of the Buffalo Crossing Bridge was determined to be the desired option, as opposed to maintenance, due to the limited remaining lifespan of the existing bridge and the new load requirements.

The proposed replacement structure type is assumed to be a simple span prefabricated steel structure on deep foundations. Deep foundations are proposed due to the higher costs anticipated with providing shallow foundations at the depth to rock near 17 feet. Additional materials, such as concrete, rebar, excavation, backfill, and dewatering, as well as possible increased construction time, would be required to construct shallow foundations. Considerations related to the abutments for the Buffalo Crossing Bridge are presented in Section 4 of this report.

1.2 SCOPE AND PURPOSE

The scope of work included a geotechnical investigation, analysis, and recommendations for bridge foundations for use in design and 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 bridge location and develop recommendations concerning bridge foundations, retaining structures, geologic hazards, and construction considerations.

We understand that a hydraulics report has been prepared under separate cover by the CFLHD Hydraulics Engineer. The hydraulics report includes discussion of flood impacts, such as bridge scour and water surface elevations.

SECTION TWO - Geology and Seismicity

2.1 REGIONAL GEOLOGY

The project is located within the Mexican Highland Section of the Basin and Range Province. This province is part of the Intermontane Plateaus physiographic region. The Basin and Range Province extends from eastern California to central Utah and from southern Idaho to the state of Sonora in Mexico. This province is characterized by steep climbs up mountain ranges alternated with long treks across flat basins. Features and landscapes within this province include pediments, alluvial fans, bajadas, bolsons, Inselbergs, playas, mud flats, salt flats, lakes, sand dunes, canyons, and the Rio Grande Rift (NPS, 2020).

Quaternary and Tertiary rocks and deposits that exist within the region include siltstone, sandstone, shale, limestone, conglomerate, basalt, tuff, agglomerate, rhyolite, andesite, dikes, plugs, sills, alluvium, gravel, sand, and silt. Cretaceous aged units include the Mesa Verde group, Mancos shale, Dakota sandstone, diabase, and granite and related crystalline intrusive rocks. Jurassic, Triassic, and Permian units include the Morrison formation, San Rafael group, Glen Canyon Group, Chinle formation, Shinarump conglomerate, Moenkopi formation, Kaibab limestone, Coconino sandstone, and Supai formation. Older units, Carboniferous and Devonian to older Precambrian aged, include quartzite, Mescal limestone, Apache group, schist, and granite and granite gneiss (Wilson & Moore, 1958; Wilson et al., 1960).

2.2 SITE GEOLOGY

Although not mapped, the soil conditions at the site are assumed to be alluvium based on the stream bed deposits. The project site is mapped as underlain by basalt. The basalt locally includes tuff and agglomerate. This observation is consistent with the borings as will be discussed in Section 3 of this report. The deposits and landforms are volcanic in nature and the terrain can be described as mountainous and rugged. A geology map of the project area and corresponding map legends are presented in Plates 2 & 3.

2.3 REGIONAL SEISMICITY

No seismic hazard faults are located within 50 miles of the project. However, six known Quaternary faults are mapped within approximately 50 miles of the bridge site. These faults are considered inactive and are summarized in Table 2.1. The Vernon Fault Zone, Concho Fault, Coyote Wash Fault, and Red Hills Faults are located north and northeast of the project site and the Alma Mesa Faults and Unnamed Faults East of Alma are located south and southeast of the project site. All fault systems are considered normal faults, with the exception of Vernon Fault Zone which is considered to be left lateral, with slip rates less than 0.008 inches per year. No known active faults underlie or are closely associated with the project site (USGS, 2020a).

Table 2.1 - Summary of Nearby Quaternary Faults

FAULT OR FAULT ZONE	DISTANCE FROM CENTER OF PROJECT *(miles)*	SLIP-RATE CATEGORY *(inch/year)*	DIP DIRECTION	AVERAGE STRIKE	FAULT LENGTH *(miles)*	TIME OF MOST RECENT DEFORMATION *(years)*
Vernon Fault Zone, (Class A) No. 1016	19.1	<0.00787	NE	N46°W	35.4	<750,000
Alma Mesa Faults, (Class A) No. 941	26.0	<0.00787	E; SE; W; NW	N23°E	9.32	<1,600,000
Coyote Wash Fault, (Class A) No. 1015	29.5	<0.00787	SW	N42°W	26.1	<750,000
Concho Fault, (Class A) No. 1014	30.3	<0.00787	NE	N37°W	24.2	<750,000
Red Hill Faults, (Class A) No. 2138	34.5	<0.00787	SE; NW	N25°E	9.3	<1,600,000
Unnamed Faults East of Alma, (Class A) No. 2011	35.6	<0.00787	W; E	N12°W	7.5	<1,600,000

2.4 SEISMIC DESIGN PARAMETERS

Recommended seismic response parameters for the Buffalo Crossing Bridge project site design 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 100-year return period). The 1000-year return period uniform hazard spectrum for the Buffalo Crossing Bridge project site, located at 33.76109°N latitude and 109.35701°W longitude, was obtained in accordance with the AASHTO ground motion maps.

Based on subsurface conditions encountered during drilling and the USGS Vs30 online map viewer, the onsite soils can be assumed to be reasonably dense. Therefore, the site is classified as **Class C** according to the site class definitions specified in Table 3.10.3.1-1 of AASHTO. The recommended spectral acceleration coefficient values for probabilistic design are summarized in Table 2.2.

Table 2.2 - Summary of Seismic Parameters Corrected for Site Class C

Horizontal Peak Ground Acceleration, (PGA)	0.071 g
Horizontal Response Spectral Acceleration at Period of 0.2 sec, (S_s)	0.165 g
Horizontal Response Spectral Acceleration at Period of 1.0 sec, (S_1)	0.049 g
Site Factor at Zero-Period of Acceleration Spectrum, (F_{pga})	1.2
Site Factor at Short-Period Range of Acceleration Spectrum, (F_a)	1.2
Site Factor at Long-Period Range of Acceleration Spectrum, (F_v)	1.7
Factored Horizontal Peak Ground Acceleration, (A_s)	0.085 g
Factored Horizontal Response Spectral Acceleration at Period of 0.2 sec, (SD_s)	0.198 g
Factored Horizontal Response Spectral Acceleration at Period of 1.0 sec, (SD_1)	0.083 g
Seismic Zone	**Zone 1**

Based on the long acceleration coefficient S_{D1} value of 0.083, the project site is assigned to seismic hazard "Zone 1" in accordance with Table 3.10.6-1 of AASHTO. Based on this assignment, seismic loading is not likely to control design of structures for the project.

2.5 GEOLOGIC HAZARDS

Potential geologic hazards at the bridge site include those related to floods, earthquakes, and mass movement. Flooding, however, is the most common, widespread, and damaging of these geologic hazards and could pose a threat to the Buffalo Crossing Bridge. Much of Arizona is prone to flooding caused by thunderstorms that happen year-round. Severe thunderstorms are most common during the monsoon season from mid-June through September. Flooding can cause many effects due to contact with flood waters such as extensive erosion, debris flows, and slope instability.

Earthquakes, as discussed in the previous section, are another potential geologic hazard. Eastern Arizona and the project site are located in an area of low potential for damaging earthquakes, although it is impossible to accurately predict the timing or location of future earthquakes.

Mass movements are also common in Arizona due to the steep mountainous and hilly terrain, heavy rains, and the fines content of the soil. These rock and soil movements include debris flows, landslides, and rockfalls. Although the bridge is located in terrain susceptible to mass movement, no indication of these events were observed during the site reconnaissance and are considered unlikely to impact this bridge site.

SECTION THREE - Subsurface Investigation

3.1 SUBSURFACE EXPLORATION PROGRAM

A subsurface investigation targeting the bridge site was performed by a Central Federal Lands Highway Division (CFLHD) of the Federal Highway Administration (FHWA) geotechnical engineer on September 21 & 22, 2020. The geotechnical subsurface exploration program consisted of drilling a total of two borings each to a depth of 26-feet deep. One boring was drilled near each bridge abutment. Hollow-stem augers were used to drill through the overburden soils and bedrock samples were recovered using NQ diamond core drilling. Standard penetration testing (SPT) and sample collection was performed at 5-foot intervals for each boring when possible. Grab samples of the upper subsurface were also collected. 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 4, respectively. Photographs related to this exploration can be found in Appendix C. A summary of the field exploration is provided in Table 3.1.

Table 3.1 - Summary of Field Exploration Program

EXPLORATION DESIGNATION	LOCATION	APPROXIMATE GROUND ELEVATION (ft.)	TERMINATION DEPTH (ft.)	DEPTH TO GROUNDWATER (ft.)
BH20-01	North Abutment; STA 104+90, 8.5 ft. RT of centerline	7,541	26	Not encountered.
BH20-02	South Abutment; STA 102+35.5, 15.5 ft. LT of centerline	7,538	26	Not encountered.

Note: The exploration locations were estimated relative to existing features. Ground elevations were estimated from Google Earth.

3.2 LABORATORY TESTING PROGRAM

Grab samples and 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 T 11 and T27), classification (AASHTO M145), Atterberg limits (AASHTO T89 and T90), resistivity (AASHTO T 288), and soil pH (AASHTO T 289). 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 summarized below in Table 3.2 and are presented in Appendix B.

Table 3.2 - Summaries of Laboratory Index Test Results

BORING NUMBER	SAMPLE DEPTH (*feet*)	PERCENT GRAVEL	PERCENT SAND	PERCENT PASSING #200	LIQUID LIMIT	PLASTIC LIMIT	USCS CLASS.	AASHTO CLASS.
BH20-01	0 - 2	31	41	28	38	24	SC	A-2-6(1)
BH20-01	4 - 5	8	64	28	31	18	SC	A-2-6(0)
BH20-02	0 - 2	46	38	16	NV	NP	GM	A-1-b(0)
BH20-02	11 - 12.5	36	4	60	28	25	ML	A-4(0)

Soil test results indicated a range of material types including gravel, clayey sands, and silt, classifying as A-1-b to A-4 by AASHTO and GM, SC, and ML by USCS.

Corrosivity and chemistry testing of select samples were sent to an outside laboratory for testing and are summarized in Table 3.3.

Table 3.3 - Summaries of Laboratory Corrosivity Results

BORING NUMBER	SAMPLE DEPTH (*feet*)	RESISTIVITY (*ohm-cm*)	PH	SULFATE ION CONTENT (*ppm*)	CHLORIDE ION CONTENT (*ppm*)
BH20-01	4 - 5	2,790	8.4	20	4

- Tests not completed.

Unconfined compressive strength (UCS) of rock materials was conducted on intact samples of rock core recovered from the borings. Results of the UCS testing are presented in Table 3.4.

Table 3.1 - Summary of Uniaxial Compressive Strength Results

BORING NUMBER	SAMPLE DEPTH (*feet*)	RQD[1] (*%*)	LENGTH / DIAMETER[2]	UCS (*psi*)
BH20-01	11	20	2.4	2,030
BH20-01	21	92	2.4	1,850
BH20-01	26	92	2.5	660[3]
BH20-02	21	82	2.5	1,250
BH20-02	22	82	2.5	1,490

[1]Rock Quality Designation for core run from which sample was taken.
[2]Values less than 2.0 do not meet the requirements of ASTM D 7012. Not included in analysis if encountered.
[3]Specimens broke in less than the 2 and 15-minute requirement per ASTM D 7012. Not included in analysis.

3.3 SUMMARY OF SITE CONDITIONS

This section presents the results of the surface reconnaissance and subsurface exploration.

3.3.1 General Topography

The general topography of the project site is characterized by steep mountains and lengthy deserts that encompass southeastern Arizona and southwestern New Mexico. Rock outcroppings are present in the area surrounding the bridge site. Thick vegetation typical of both ponderosa pine and pinyon-juniper woodland forests is abundant at the project site. The East Fork Black River flows under the Buffalo Crossing Bridge from the north to the south and converges with the West Fork Black River approximately 0.75 miles downstream of the project site.

3.3.2 Surface Reconnaissance

Existing rock basket structures are present upstream of the bridge on the west river bank. These structures supplement and mostly tie into the northeast wingwall of the north bridge abutment. They appear to be only one basket high and are anchored by steel railroad ties installed at the front facing. Details regarding the installation of these structures is unknown at this time. A layer of rock embankment overlays the rock baskets and extends to roughly the top of the river bank. The rock baskets and rock embankment continue upstream along the length of the river in a northeast direction for at least a few hundred feet, possibly more.

Rock outcroppings protrude from a slope roughly 60 feet west of the north bridge abutment. The slope is roughly 30 feet high and appears to be around or slightly steeper than a 1V:1.5H slope ratio. Vegetation including junipers, small pines, and native grass species heavily cover much of the slope. Plate-shaped rock outcroppings protrude along the face of the slope and more block-shaped rock masses line the crest of the slope. The rock type is consistent with the geology discussed in Section 2. Much of the exposed rock faces are partially covered in moss, at least slightly weathered, and show signs of natural fracture patterns. A campground area is present on top of this slope to the northwest of the bridge.

Boulders roughly 4 feet in diameter and larger were observed in the river channel beneath and surrounding the bridge. These boulders could have been placed around the existing piers as scour protection and may have been locally sourced during the original bridge construction. Additional rock embankment surrounds the southeast bridge wingwall but is relatively shorter in length and less robust than the previously discussed embankment on the northwest river bank. Thick vegetation, as previously described, lines both banks of the river and alluvial soils and sediments encompass the bridge site. Forest Service Road 24A intersects with State Route 24 near the south abutment of the Buffalo Crossing Bridge and continues to the northeast parallel to the river channel. A gravel surfaced parking area roughly 160 feet long by 45 feet wide occupies the area just to the southwest of the bridge site. One covered U.S. Forest Service informational sign is present at the parking area and a short footpath begins near this sign and terminates at the river. Rock outcroppings again line the southwest river bank and trail area and boulders measured to be up to 3 feet in diameter were observed.

3.3.3 Subsurface Conditions

The subsurface conditions at the north abutment of the Buffalo Crossing Bridge were investigated by drilling boring BH20-01. The ground surface surrounding the boring location consisted of typical alluvial soils: namely silt, sand, and some gravel. The boring encountered sandy silt with gravel to a depth of approximately 4 feet. The material then transitioned to cobbles and basalt rock boulders less than 1 foot in diameter to a depth of approximately 15 feet. The upper 2 feet of the aforementioned cobble and boulder layer was intermixed with lean clay and gravel. The lower 9 feet of the boulders were intermixed with poorly graded sand with gravel. Tuffaceous bedrock was encountered beneath the cobbles and boulders and extended to the termination depth of 26 feet. This rock was light red, slightly to moderately weathered, and classified as weak rock (R2).

The subsurface conditions at the south bridge abutment were investigated by drilling boring BH20-02. Similarly, the ground surface consisted of silt, sand, and varying amounts of gravel. The boring encountered cobbles and boulders less than 1 foot in diameter intermixed with sandy silt and gravel to a depth of approximately 4 feet. The cobbles and boulders continued and became intermixed with lean clay with gravel from roughly 4 feet to 6 feet. The cobbles and boulders then became intermixed with poorly graded sand with gravel from roughly 6 to 16 feet. Tuffaceous bedrock was encountered beneath the cobbles and boulders and extended to the termination depth of 26 feet. This rock was light red, moderately weathered, and classified as weak rock (R2).

3.3.4 Groundwater

Groundwater was not encountered during drilling in either borings BH20-01 or BH20-02. However, fluctuations in the groundwater level due to seasonal and climatic effects are expected and will likely follow water levels in the adjacent river.

3.4 ROCK CHARACTERISTICS

A natural rock mass is rarely a continuous, isotropic, homogeneous material. It can be highly variable and may be very difficult to characterize in a generalized manner. A rock mass is generally composed of intact or weathered blocks of rock separated by discontinuities such as joints or bedding planes. The discontinuity and weathering characteristics help establish the design recommendations for a foundation on rock. These characteristics include, but are not limited to, the rock lithology, strength properties, deformation properties, frequency of discontinuities (RQD), spacing of discontinuities, orientation of discontinuities, aperture, filling, condition, degree of weathering, and the groundwater condition. This information was observed at limited borehole locations and laboratory testing of select rock samples. The following sections discuss the rock strength properties and rock characterization based on the field investigation and lab testing results.

3.4.1 Generalized Strength Criterion for Intact Rock

Intact samples of rock core were taken from the Buffalo Crossing Bridge boreholes and laboratory testing, such as the uniaxial (unconfined) compression test, was performed. Table 3.4 summarizes the results of the UCS testing. In the field, the rock could be scraped with a pocket knife or similar tool and it was estimated that the rock corresponded with the description of "weak rock," R2, as described in the FHWA Rock Description Guidelines. The range of relative strength for R2 rock as listed in this reference is between 725-3,500 psi. The rock also appeared to have slight to moderate weathering.

Based on the aforementioned UCS testing, the rock strength for the Buffalo Crossing Bridge is conservatively assumed to be approximately 1,370 and 1,940 psi for abutments 1 & 2, respectively. These values are equal to the average UCS test results at the two bridge abutments and are within the range of R2, "weak rock," as discussed in the previous paragraph. For comparison, the cohesive strength of very stiff clay is generally between roughly 55 and 111 psi (Texas Geosciences, 2021).

3.4.2 Rock Characterization

The Geologic Strength Index (GSI) (Hoek and Marinos, 2000) system was used to characterize the rock. The GSI is a slight modification of the Rock Mass Rating system (RMR) (AASHTO, 2017) which is more appropriate for the bridge foundation evaluation at this site. The GSI is the algebraic sum of ratings assigned for the following rock mass properties:

- Intact rock strength
- Rock Quality Designation (RQD)
- Discontinuities spacing
- Discontinuities condition
- Presence of groundwater

For this project, the GSI was estimated based on collected rock core samples. The rock composition was identified as tuff with slight to moderate weathering and blocky structure. The estimated GSI values given these assumptions corresponds to a range of 60 to 70 (Hoek and Marinos, 2000). A GSI value of 65 was then assumed for the rock. Other pertinent rock properties include the Hoek-Brown constant, m_i (estimated from Table 10.4.6.4-1 in AASHTO 2017), rock unit weight, γ, and the damage factor, D. These values were assumed to be 13 for tuff, 145 pcf, and 0.5 as an intermediate value for drilled shafts, respectively. For intact rock, the following equations were then used to determine the appropriate curve fitting coefficients:

$$m_b = m_i * \exp\left(\frac{GSI-100}{28}\right)$$

For GSI > 25: $$\alpha = 0.5 + \frac{1}{6} * (\exp\left(\frac{-GSI}{15}\right) - \exp\left(-\frac{20}{3}\right))$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3 * D}\right)$$

A summary of the assumed rock properties and the Hoek Brown criteria calculations is presented in Table 3.5 below. These values are directly related to the bearing resistance discussed in Section 4.1.3.

Table 3.5 - Strength Parameters Calculated for Rock Based on Hoek Brown Criterion

GSI	σ_{ci} (psi)	s	m_b	m_i	α
65	1,370 (abutment 1) 1,940 (abutment 2)	0.009	18.6228	13	0.502

SECTION FOUR - Analysis & Recommendations

This section presents analysis and recommendations for the bridge foundations, abutment wingwalls, earth retaining structures, permanent earthworks, and construction considerations for the design and construction of the Buffalo Crossing Bridge project. Based on discussions with the project team, a replacement of the existing bridge is the preferred alternative. Generalized subsurface profiles were developed based on field reconnaissance, surficial visual evaluation, and subsurface investigations.

4.1 FOUNDATIONS

The existing three span bridge will be replaced with a simple single-span structure. The existing piers will be removed and the existing abutments will remain in place. Based on the 95 percent design plans, the proposed abutment centerline station and cap elevation are shown in Table 4.1.

Table 4.1 - Proposed Bridge Foundation Location

FOUNDATION	APPROXIMATE STATION	TOP OF SHAFT ELEVATION*
Abutment 1 (South)	102+66	7,529.48 FT
Abutment 2 (North)	103+66	7,530.44 FT

*As presented on the 95% design plans.

4.1.1 Bridge Foundation Selection

A drilled shaft foundation system is proposed for the Buffalo Crossing Bridge due to the relatively deep depth to bedrock (greater than 10 feet). Spread footers were considered but determined impractical due to additional materials, time, and cost that would be required for construction. Spread footers would overall likely be less economical due to the size of the associated excavation and may not be able to accommodate the structure loads without being excessively large. Driven piles were also considered but determined impractical due to the presence of cobbles and boulders that could cause pile damage during driving as well as the relatively short distance, in terms of pile construction, to bedrock potentially not meeting minimum length requirements.

A drilled shaft foundation is feasible from a geotechnical perspective. Drilled shafts are able to provide a small footprint, support large foundation loads, and provide lateral resistance. The risk of scour potential is discussed in detail in the final hydraulics report. Design recommendations for the drilled shafts are provided in this report.

The following bridge loads factored for the strength limit state were provided by the CFL bridge engineer on January 8, 2020:

North & south abutments = 320 kips (per shaft; 3 shafts per abutment)

4.1.2 Site Characterization

The subsurface profile was assumed to be sand underlain by tuffaceous bedrock (weak, R2, bedrock) for analysis purposes. Although gravel, silt, cobbles, and boulders were encountered in the overburden materials of the designated boreholes, the soil matrix, in general, was of a sand-like nature and composition. Groundwater effects were not considered as groundwater was not encountered during drilling in either borehole.

4.1.3 Drilled Shaft Axial Resistance

Drilled shafts for the Buffalo Crossing Bridge will be socketed into tuffaceous bedrock. The side resistance provided by overburden soils and the rock socket are neglected for the shaft capacity analysis when shafts are socketed into competent bedrock. Side resistance is neglected due to potential strain-softening behavior of the sidewall, degradation of material in the borehole wall, and uncertainty of roughness in the socket sidewall. The competent bedrock is considered non-scourable; therefore, depth of scour was only considered in effects on vertical stress.

The UCS values of the competent bedrock were somewhat variable, and weaker than typical concrete mixes (Table 3.4). Due to these results and based on experience in similar projects and geotechnical materials, average values of 1,370 and 1,940 psi for abutments 1 and 2, respectively, were used for UCS to evaluate the axial capacities of the drilled shafts. These values are less than the typical strength of structural concrete and are used in accordance with AASHTO design specifications.

The shaft tip axial bearing resistance was calculated in accordance with the provisions of Article 10.8.3.5.4c. Based on the assumption of pyroclastic or otherwise blocky and fractured bedrock, the tip resistance was calculated using Equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3.

$$q_p = A + q_u \left[m_b \frac{A}{q_u} + s \right]^a$$

$$A = \sigma'_{vb} + q_u \left[m_b \frac{\sigma'_{vb}}{q_u} + s \right]^a$$

In these equations, m_b, s, and a are Hoek-Brown strength parameters for fractured rock masses and q_u is the unconfined compressive strength of intact rock. σ'_{vb} is the vertical stress at the tip elevation. A minimum estimated tip elevation for each foundation element has been used to calculate the nominal and factored tip resistance for various socket geometries at the strength limit state for each foundation element. This minimum tip elevation assumes a rock socket length equal to at least 150 percent of the diameter of the shaft (45 inches for a 30 inch diameter shaft). The factored resistance for the strength limit state was calculated by applying a 0.50 resistance factor for tip resistance. Based on the aforementioned calculations and the selected drilled shaft diameter of 30 inches, a minimum rocket socket length of 3.75 feet (45 inches) is required to meet axial resistance. The rock socket length shown in the plans has been rounded up to 4 feet. Assumed tip elevations and resistances are summarized on Table 4.2. The surface elevations referenced on this table are located at approximately the bottom of the new abutment caps, or the top of drilled shafts, as discussed with the Bridge Engineer and noted in Table 4.1.

Table 4.2 - Shaft Tip Resistance in Rock

STRUCTURE ELEMENT	**TOP OF SHAFT ELEV.** *(feet)*	**ESTIMATED SHAFT LENGTH** *(feet)*	**ESTIMATED TIP ELEV.** *(feet)*	**NOMINAL TIP RESISTANCE** *(kip)*	**FACTORED TIP RESISTANCE** *(kip)*
Abutment 1 (south)	7,529.48	18.48	7,511.00	1,583.0	791.6
Abutment 2 (north)	7,530.44	19.44	7,511.00	2,012.0	1,006.0

4.1.4 Group Effects on Axial Resistance

The resistance of a shaft group to the applied axial loads is not necessarily the sum of the axial resistance of individual shafts within the group. The zone of influence from an individual pile in a pile group may intersect with other piles, depending on the pile spacing. Historically, the efficiency of groups of drilled shafts has not been a concern as long as the center-to-center spacing between shafts is greater than three times the shaft diameter (3D) to avoid interference between adjacent shafts, assuming a single row shaft group configuration. An efficiency factor (η) should be applied for spacing less than 3D as shown in Table 10.8.3.6.3-1.

Besides the effect of overlapping zones of influence, effects of construction on ground conditions in and around the group can be significant. Excavated deep foundation elements in cohesionless soils tend to decrease the effective stress of the surrounding soils. Poorly controlled shaft construction methods can result in soil loosening during drilling and adversely reduce the lateral stress around other shafts within the group. Casing driven in advance of excavation may increase the relative density and effective stress of the surrounding soil and prevents caving of overburden material.

4.1.5 Lateral Loads on Deep Foundations

Lateral load analysis was performed by the 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. Table 4.3 provides preliminary LPILE input parameters for the foundation soils based on available subsurface information and presumptive engineering correlations. It is also recommended that lateral support within five feet of the final road grade be neglected due to the potential loss of support from frost penetration or other shallow ground disturbance.

Table 4.3 - LPILE Parameters

APPROXIMATE ELEVATION (FT)	**DEPTH BELOW TOP OF SHAFT (FT)**	**LPILE P-Y MODEL**	**EFFECTIVE UNIT WEIGHT (PCF)**	**FRICTION ANGLE (DEG)**	**SOIL MODULUS (PCI)**
MODEL No. 1 (SAND & STRONG ROCK)					
7,529.48 to 7,515.00 (Abutment 1) 7,530.44 to 7,515.00 (Abutment 2)	0 to 14.48 (Abutment 1) 0 to 15.44 (Abutment 2)	Sand (Reese)	120	32	90
TRANSITION TO ROCK PROPERTIES		LPILE P-Y MODEL	EFFECTIVE UNIT WEIGHT (PCF)	UNIAXIAL COMPRESSIVE STRENGTH (PSI)	
7,515.00 to 7,511.00 (Abutments 1 & 2)	14.48 to 18.48 (Abutment 1) 15.44 to 19.44 (Abutment 2)	Strong Rock (Vuggy Limestone)	145	1,655	

Material properties provided are for single shafts and do not account for the reduced lateral resistance of shafts in a group. P-multipliers are a function of the number of rows of shafts and center-to-center shaft spacing in the direction of loading. P-multipliers are required even for a single row of shafts if the center-to-center spacing is less than 5 shaft 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 conditions.

4.1.6 Settlement

A resistance factor of 1.0 for the service limit state is recommended to assess the ability of the foundation to meet the specified deflection criteria. Elastic settlements will occur immediately and be essentially complete at the end of construction and are estimated to be less than one inch at all locations based on the loads provided. Differential settlements are not anticipated. Shaft group settlement is not anticipated.

4.1.7 Field Testing

Steel crosshole sonic logging tubes (1.5-inch diameter steel tubes) should be installed in all drilled shafts prior to concrete placement for integrity testing. Crosshole sonic logging tubes cannot be installed in shafts less than 2 feet in diameter; therefore, a minimum shaft diameter of 2.5 feet is recommended. The recommended number of access tubes and tube spacing are dependent on the selected shaft diameter; refer to Table 4.4. The crosshole sonic logging tests should be conducted in accordance with ASTM D-6760 and FP-14 for quality assurance/quality control of the drilled shafts.

Table 4.4 - Recommended Number of CSL Access Tubes by Shaft Diameter

SHAFT DIAMETER D *(ft)*	**NUMBER OF TUBES**	**TUBE SPACING** *(degrees)*
2.5 ≤ D < 3.5	3	120
3.5 ≤ D < 5	4	90
5 ≤ D < 7	6	60

4.1.8 Scour Potential and Erosion

Contraction scour and local abutment scour depths were estimated by the hydraulics engineer. Section 4.4.3 in the Final Hydraulic Report details the scour depth estimates.

Riprap for embankment protection is shown in the plans near the abutment wingwalls per guidance from the bridge and hydraulics engineers. For additional scour information please refer to the previously mentioned Final Hydraulic Report dated January 2021.

4.2 ABUTMENT & WINGWALL DESIGN

Abutments and wingwalls should be designed to resist lateral earth pressures and other applicable lateral loads in accordance with AASHTO. Lateral earth pressure is influenced by the strength of the abutment backfill, the presence or absence of water, and the ability of the abutment or wall to move in response to lateral loads. Other loads, such as live loads, construction loads, and soil compaction loads should also be considered in the design.

Unbalanced water behind an abutment or wall adds significant lateral pressure and should be avoided by using free draining gravity outlets for water. Abutment and wingwall backfill should consist of structural backfill as specified in Section 704.04 of FP-14.

The coefficient of at-rest earth pressure should be used for design if the abutment is so restrained that it cannot be expected to rotate (deflect at the top) 0.002 times the wall height. Where deflection of the abutment can be expected, a coefficient of active earth pressure should be used for wall design. Active and at-rest lateral earth pressures of native materials and properly placed and compacted structural backfill above the water table are presented in Table 4.5. The values are unfactored loads and assume that the surface of the soil slope behind the wall is horizontal.

Table 4.5 - Lateral Earth Pressures for Bridge Abutments & Wingwalls

BACKFILL TYPE	ASSUMED BACKFILL PROPERTIES	CASE	UNFACTORED EQUIVALENT FLUID DENSITY (PCF)	NOMINAL FRICTION FACTOR
Structural Backfill	c = 0 psf ϕ = 34 deg. γ = 125 pcf	Active	35	0.28
		At-Rest	55	0.44
Native Soil	c = 0 psf ϕ = 32 deg γ = 120 pcf	Active	37	0.31
		At-Rest	56	0.47

4.3 EARTH RETAINING STRUCTURES

The designer has proposed to raise the roadway grade leading to the north bridge abutment by approximately 7 feet. Adjacent to the northeast bridge abutment is a designated wetland area that is prohibited from environmental impact and harm, as well as an existing rock basket structure of questionable integrity. This existing structure, as mentioned in Section 3.3.2, can be seen in Figure 1. Therefore, it has been proposed to construct an earth retaining structure at the northeast quadrant of the bridge to retain the new embankment material from the bridge approach (roadway) grade raise, allow widening for the proposed guardrail sections, and to avoid impacts to the protected wetland. The following sections discuss the design alternatives that were considered as well as the design approach and methodology for the chosen earth retaining structure.

Figure 1 - Existing Rock Basket Structure

4.3.1 Design Considerations

Six design alternatives were considered to address the aforementioned roadway grade raise and conflict with the adjacent wetland area. These alternatives were considered to be the most economical and practical for the project and the Forest Service. These six design alternatives were as follows:

- Extend the proposed concrete bridge wingwalls further & found on either drilled shafts or spread footers below the scour depth.
- Steepen the fill slope leading to the bridge, if possible by design standards, and maintain revegetation.
- Construct a reinforced soil slope (RSS).
- Construct a gabion retaining wall.
- Construct a gabion face MSE wall.
- Place gabion baskets on top of existing rock basket structures.

The gabion retaining wall was ultimately chosen as the most cost effective option and would have the lowest potential impact to the wetland. The other options would presumably have larger footprints which would risk disrupting the environmentally sensitive areas. The existing rock basket structures will be removed and riprap will be placed at the base of the proposed gabion wall to protect the wall from scour. The following section discusses the gabion wall design.

4.3.2 Gabion Wall Design

Gabion walls are earth retaining structures constructed using large twisted wire baskets filled with rock that are stacked, typically without connections between the baskets. These structures rely on the weight, size, shape, and interface friction of the individual baskets to resist earth pressures and provide overall stability. Gabion walls are frequently used due to their lower cost and improved aesthetics compared to traditional concrete retaining walls. A cross section of a typical FHWA gabion retaining wall is shown in Figure 2.

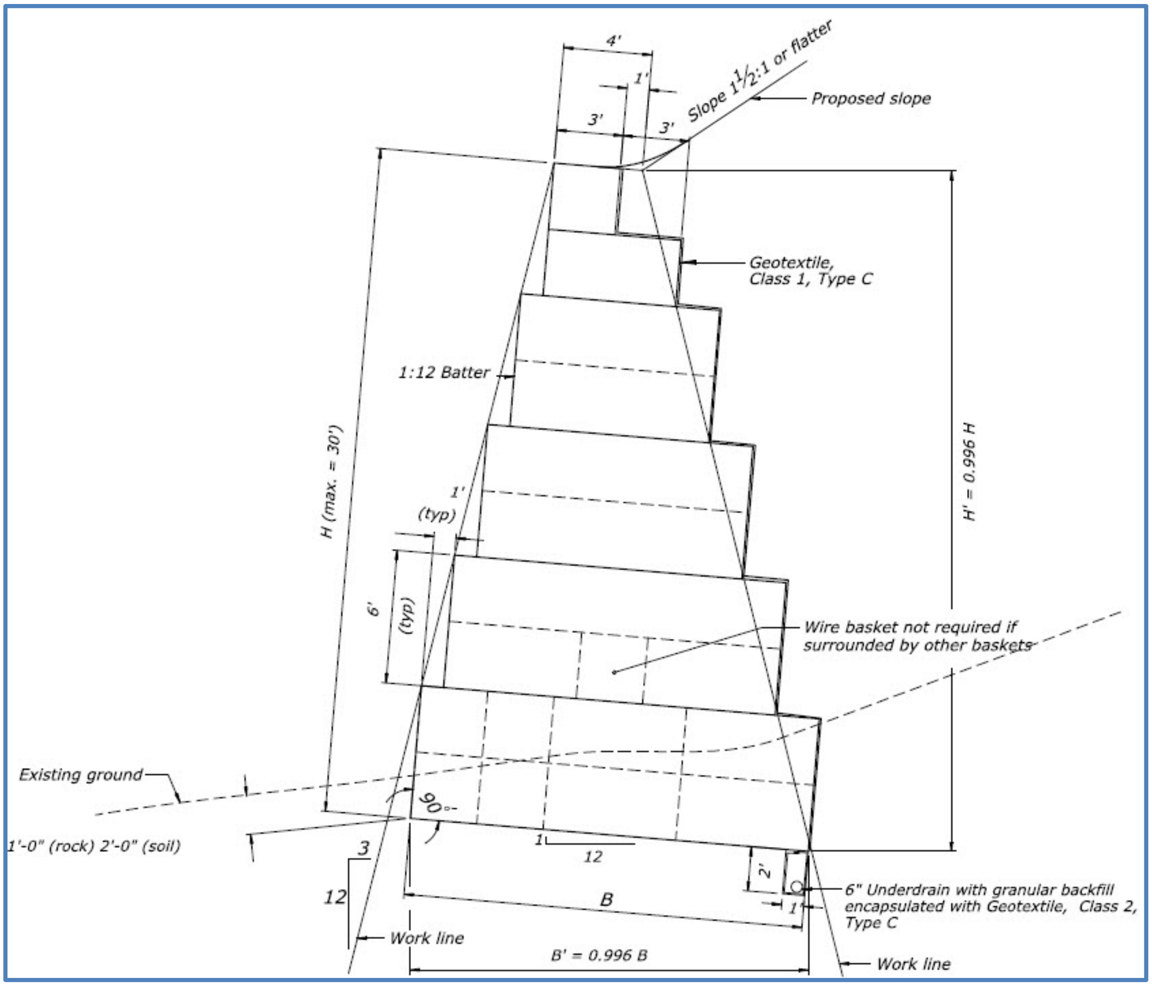

Figure 2 - Cross Section of Gabion Retaining Wall (Conceptual)
[provided by Eastern Federal Lands Highway Division, Detail E253-01]

The gabion wall was designed based on traditional earth pressure theory using the GAWACWin software published by Maccaferri, Inc. The software uses limit equilibrium theory to analyze sliding, overturning, eccentricity, bearing, and global stability. Limited surficial geotechnical investigations were conducted near the wall site; therefore, a conservative design approach was followed. Presumptive soil strength properties for retained and foundation soils were assumed for observed on-site soils. The gabion wall was designed as a gravity retaining wall and a surcharge load of 250 pounds per square foot was applied to the backfill to account for traffic loading. Static lateral earth pressures were applied to the back of the wall; however passive earth pressure was neglected at the toe of the wall. As stated in Section 2.4, due to the classification of the site as Seismic Hazard Zone 1, seismic loading was not considered in the design.

The design was checked for adequate factors of safety against sliding, overturning, and bearing capacity. The recommended factors of safety for static gabion wall design are presented in the table below:

Table 4.6 - Recommended Safety Factors for Static Design

MODE OF FAILURE	STATIC FACTOR OF SAFETY
Sliding	1.5
Overturning	2.0
Bearing	3.0
Global	1.3

Assumed material properties used in the design of the gabion walls are presented in Table 4.7. Stability analysis of the wall did not include hydrostatic pressures behind the gabions. The gabions and the wall backfill are assumed to be free draining, such that elevated water levels in the wall backfill will not occur. Wall geometries and the calculated wall design factors of safety for each wall analysis will be discussed in the following paragraphs.

Table 4.7 - Material Properties Used in the Analysis for Gabion Wall

ASSUMED LOCATION	φ RETAINED SOIL FRICTION ANGLE	γ_S RETAINED SOIL UNIT WEIGHT	c SOIL COHESION	μ SLIDING FRICTION FACTOR	γ_R GABION FACE UNIT WEIGHT
STA 103+82 to STA 103+35	32°	120 pcf	0 psf	0.625	125 pcf

The gabion wall should be constructed starting at the end of the northeast bridge wingwall near STA 103+82 with an offset of approximately 18.4 feet. The gabion wall should follow the approximate angle of the bridge wingwall and terminate around STA 103+35. At this location, the toe of the embankment slope has been eroded by intermittent stream flow. At their maximum, the exposed height of the gabion wall is estimated to be 9 feet. The base of the wall should also be embedded a minimum of approximately 2 feet. The base width will be determined by the height of the proposed wall at each location. The backslope angle, β, should not be greater than 26.6 degrees. The gabions should be constructed in accordance with Special Detail 253-B, including a 12V:1H face batter and 1-foot set back at courses where the width changes. An underdrain is not necessary due to the free-draining nature of gabions.

The gabion baskets exposed to the stream flow should be coated with polyvinyl chloride (PVC), epoxy, or other inert materials to reduce the potential for damage to the wall structure. Gabions exposed to the site soils, not treated with a coating should be galvanized and provided with sacrificial wire thickness to account for corrosion during the service life of the wall. Non-woven geotextile should be placed against the back of the gabions to prevent backfill soil from migrating into the baskets. The gabion wall should be backfilled with properly compacted structural backfill material. The toe of the gabion wall should be embedded below the scour depth identified by the CFLHD hydraulics engineer. Table 4.8 summarizes the required base widths with a given wall height and Table 4.9 shows the related factors of safety.

Table 4.8 - Gabion Wall Geometry

H MAXIMUM HEIGHT	B MINIMUM BASE WIDTH
6 ft	6 ft
9 ft	9 ft
12 ft	9 ft
15 ft	12 ft
18 ft	12 ft
21 ft	15 ft

Table 4.9 - Summary of Calculated Factors of Safety for Static Conditions

MAXIMUM HEIGHT	SLIDING FACTOR OF SAFETY	OVERTURNING FACTOR OF SAFETY	BEARING CAPACITY FACTOR OF SAFETY	STATIC GLOBAL STABILITY FACTOR OF SAFETY
6 ft	2.5	3.5	3.6	1.4
9 ft	2.1	3.7	5.3	1.4
12 ft	2.1	4.2	6.8	1.5
15 ft	1.9	3.0	4.0	1.4
18 ft	2.0	3.5	5.1	1.5
21 ft	1.8	2.7	3.6	1.4

The contractor will verify the limits of the structure and submit gabion drawings according to FP-14, Section 253 Gabions and Revet Mattresses and 104 Control of Work.

4.4 PERMANENT EARTHWORKS

4.4.1 Embankment Construction

Embankment construction will be necessary with regards to the proposed roadway grade raise and concrete wingwalls. Construct permanent long-term embankments with a maximum slope ratio of 1V:2H to maintain slope stability and promote slope vegetation.

4.4.2 Shrink/Swell Recommendations

On-site soils expected to be encountered within the project limits generally consist of clayey sand and silt with varying amounts of gravel. It is estimated that these soils will have a 10 percent shrink percentage, corresponding to a shrink/swell factor of 0.90. The recommended shrink/swell factor is 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.

4.5 CONSTRUCTION CONSIDERATIONS

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

Based on the subsurface investigation and surface geologic mapping, the rock encountered, primarily basalt and tuff, is expected to be rippable in the area surrounding the abutments of the Buffalo Crossing Bridge; however, final determination of bedrock rippability is the responsibility of the contractor. Shear wave velocity of the geologic layers was not evaluated by geophysical methods. However, based on Table D.1 in Appendix D, the rock encountered can be presumed to be between the soft and hard rock descriptions. Additional information regarding the excavation characteristics of rock and rippability charts can also be found in Appendix D.

Evidence of definitive in-place bedrock was encountered from the subsurface investigation detailed in this report. Appropriate construction equipment should be mobilized to the site based on the rock information detailed above. The contractor is responsible for safety of excavations and shoring design.

Drilled Shafts: The contractor will be required to submit a drilled shaft construction plan according to Section 565.04 of the FP-14, which includes outlining the proposed methods to maintain borehole stability, manage the excavation of rock, concrete placement, and dewatering.

Tuffaceous rock was encountered at all the exploration boreholes. Be prepared for cobbles and boulders ranging in size from approximately 12 to 48 inches and larger. Loose subsurface materials were also encountered at shallow depths, which may warrant the use of temporary casing.

It is anticipated that groundwater could be encountered at the foundation excavations. The likelihood of flowing sands could also present a challenge for the driller as the mixture of fine sand and water could infiltrate the drilling equipment. The contractor should be prepared for wet construction methods as groundwater can infiltrate even when temporary casing is used.

Corrosivity: Based on guidelines outlined in subsection 704.08(b) of FP-14, a pH below 5 or above 10, a resistivity below 3,000 ohm-cm, and sulfate and chloride concentrations above 200 and 100 parts per million, respectively, are representative of an aggressive soil environment. The sample tested at boring BH20-01 from 4-5 feet resulted in a resistivity of 2,790 ohm-cm. This test result is not within the guidelines defined in subsection 704.08(b) of FP-14 and minor caution should be taken with regards to corrosion potential.

Dewatering: Dewatering may be needed due to water infiltration from the river. Perform dewatering according to Section 208.07 of the FP-14.

Slope Instability: Slopes could become unstable during construction operations due to precipitation, flooding events, or heavy loading on soft subgrade soils. Precautions should be taken if movement or cracking is observed. Seepage is a contributor to slope instability and similar dewatering and stabilization methods could be used to suspend slope movement.

Hard Layers: Hard layers, such as rock, were encountered during the geotechnical investigation. The Contractor should be prepared with appropriate equipment based on the local geology as discussed in Sections 2.1 and 2.2, and Appendix D (Rippability).

4.6 SPECIFICATIONS

Special provisions were provided prior to the 95% design milestone to be consistent with geotechnical recommendations stated above and were incorporated into the special contract requirements (SCR) to amend the FHWA Standard Specification for Construction of Roads and Bridges on Federal Highway Projects; known as FP-14.

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

American Association of State Highway and Transportation Officials (AASHTO), 2017, LFRD Bridge Design Specifications, 8th Edition.

Federal Highway Administration (FHWA), 2007, Geotechnical Technical Guidance Manual, dated May.

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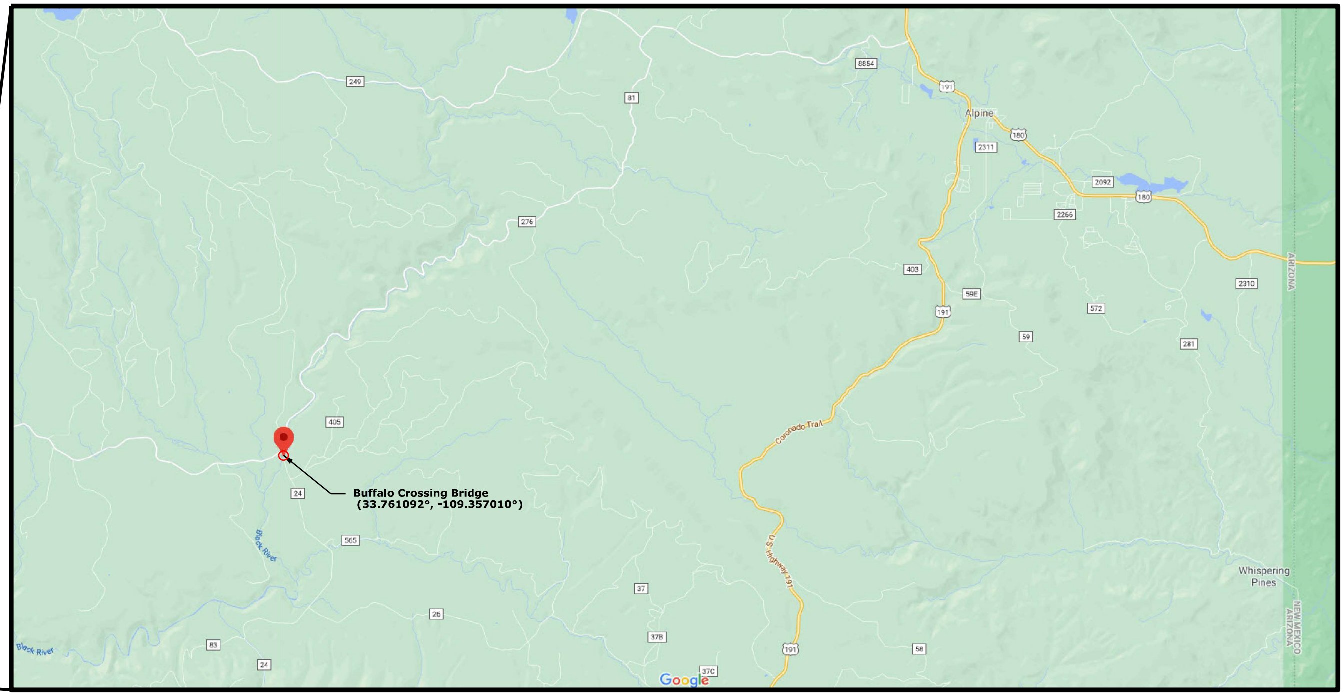
Wilson, E.D., Moore, R.T., and O'Haire, R.T., 1960, "Geologic Map of Navajo and Apache Counties, Arizona": Arizona Bureau of Mines - Univ. of Arizona, Arizona County Map Series 03-07, scale 1:375,000.

PLATES

PROJECT VICINITY MAP,
GEOLOGY MAP & LEGEND,
AND GEOTECHNICAL BORING LOCATIONS MAP

STATE	PROJECT	PLATE
AZ	AZ FS 24(1) BUFFALO CROSSING BRIDGE	1



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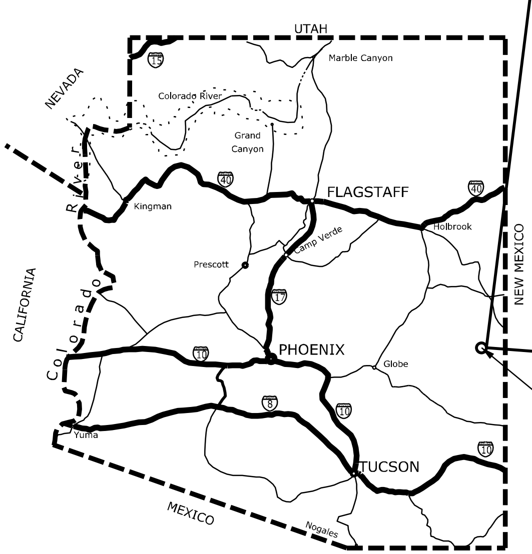

KEY MAP OF ARIZONA

N

NOT TO SCALE

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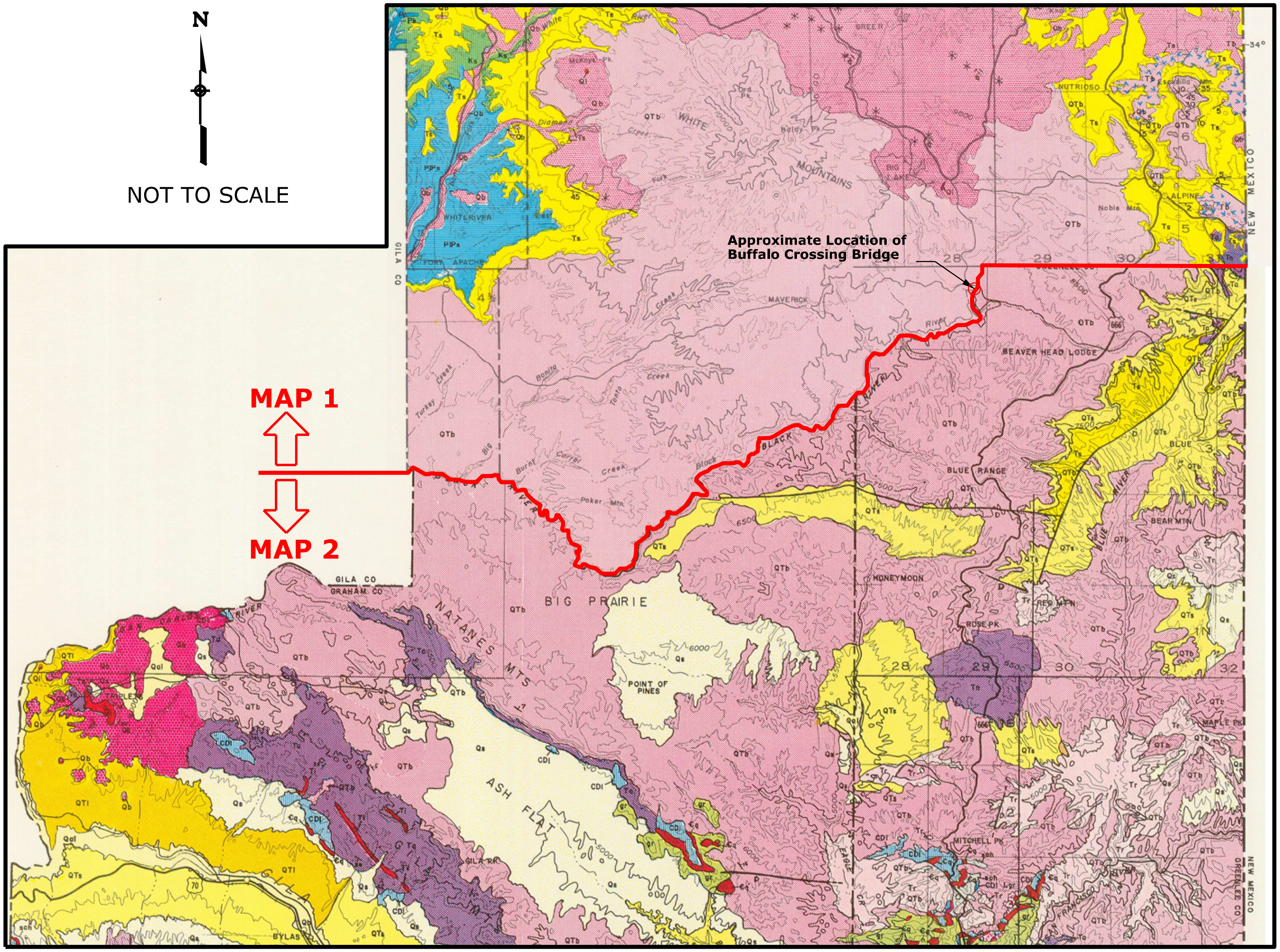
PROJECT VICINITY MAP

STATE	PROJECT	PLATE
AZ	AZ FS 24(1) BUFFALO CROSSING BRIDGE	2

NOT TO SCALE



MAP 1: [Wilson, E.D., Moore, R.T. and O'Haire, R.T., 1960, "Geologic Map of Navajo and Apache Counties, Arizona": Arizona Bureau of Mines - Univ. of Arizona, Arizona County Map Series 03-07, scale 1:375,000.]

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MAP 2: [Wilson, E.D., and Moore, R.T., 1958, "Geologic Map of Graham and Greenlee Counties, Arizona": Arizona Bureau of Mines, County Geologic Map Series M-3-4, scale 1:375,000.]

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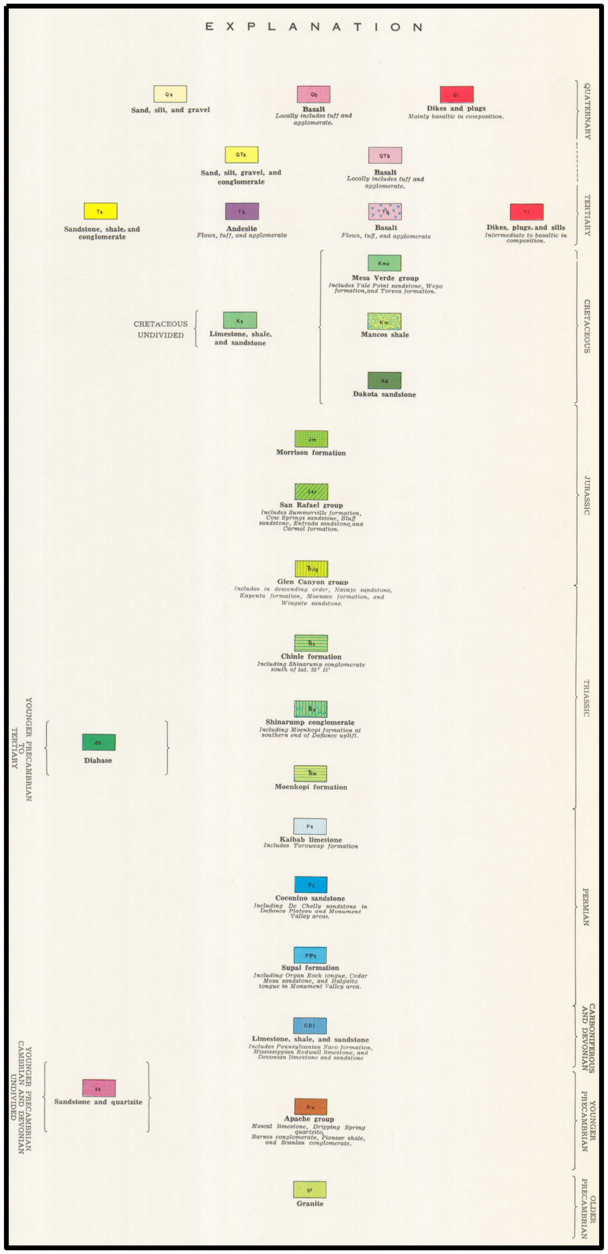
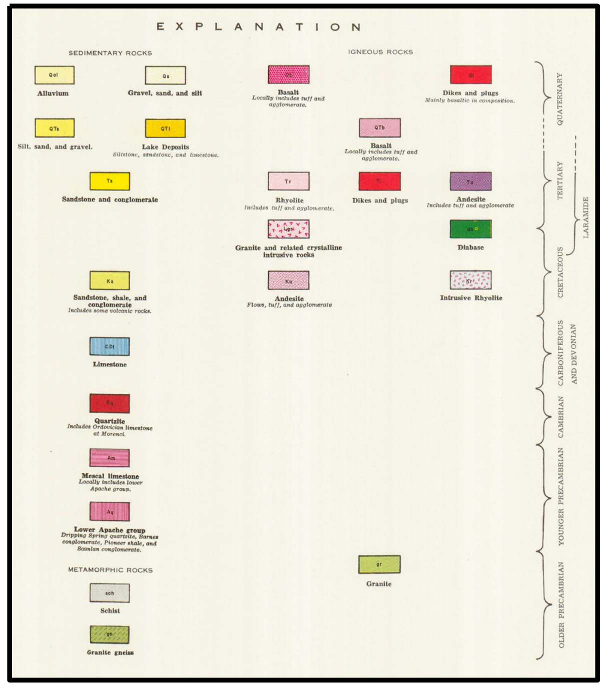
GEOLOGY MAP

STATE	PROJECT	PLATE
AZ	AZ FS 24(1) BUFFALO CROSSING BRIDGE	3

MAP 1 LEGEND



MAP 2 LEGEND



MAP 1: [Wilson, E.D., Moore, R.T. and O'Haire, R.T., 1960, "Geologic Map of Navajo and Apache Counties, Arizona": Arizona Bureau of Mines - Univ. of Arizona, Arizona County Map Series 03-07, scale 1:375,000.]

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MAP 2: [Wilson, E.D., and Moore, R.T., 1958, "Geologic Map of Graham and Greenlee Counties, Arizona": Arizona Bureau of Mines, County Geologic Map Series M-3-4, scale 1:375,000.]

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GEOLOGY MAP LEGEND

STATE	PROJECT	PLATE
AZ	AZ FS 24(1) BUFFALO CROSSING BRIDGE	4

Boring Locations

Boring	Location	Latitude	Longitude	Elevation
BH20-01	North Abutment	33.761522	-109.357098	7,541
BH20-02	South Abutment	33.760839	-109.356975	7,538

Boring locations are approximate. Elevations are estimated from Google Earth.

Google Maps, 2020. Buffalo Crossing Bridge. Google Maps [online]. Accessed 31 August 2020.

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GEOTECHNICAL BORING LOCATIONS

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 AZ FS 24(1), Buffalo Crossing Bridge, from September 21st through September 22nd, 2020. The scope of work for the field exploration program included drilling a total of two borings each to a depth of 26-feet. One boring was drilled near each bridge abutment. The field exploration program was coordinated and observed by a Geotechnical Engineer from CFLHD. Field exploration locations are illustrated on the "Geotechnical Boring Locations" sheet in Plate 4. 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 and rock 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

Borings

Geomechanics Southwest, Inc. of Phoenix, AZ provided the drilling services for the soil and rock borings. Borings were completed using a CME 75 drill rig. Borings were advanced through overburden using hollow stem augers with drive sampling until practical auger refusal was encountered. After refusal, the borings were advanced using a rock coring and continuous sampling system. Following drilling activities, field personnel backfilled the borings with cuttings generated during the drilling in accordance with applicable local, state, and federal regulations.

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.

A.3 SOIL AND ROCK SAMPLING

Borings

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. Recent energy measurements of the automatic hammer system employed for the SPT's on this project indicate an efficiency of 92%. The summary report, excluding detailed tables and appendices, indicating the energy measurements and related data analysis conducted by GRL Engineers, Inc. (GRL) for seven drill rigs is included at the back of this appendix. Rig number 109 was the chosen equipment for this project. Representative portions of the split-spoon sample obtained in conjunction with the SPT were placed in plastic baggies and transported to the CFLHD Materials Laboratory for testing.

Rock samples were collected from borings using triple-tube HQ wireline coring methods. During coring, cuttings were removed from the borehole by circulating water down the drill rods and back up the annulus of the boring. Core runs ranged between approximately two and five feet in length. Percent recovery and rock quality designation (RQD) values are shown on the boring logs. Percent recovery is the ratio of the length of recovered core to the total length of the core run. RQD is the ratio of the sum of the length of intact core pieces greater than 4 inches long in a run to the total length of the core run. Following drilling, the core was logged by CFLHD Geotechnical personnel, then stored in cardboard core boxes and transported to the CFLHD Materials Laboratory.

A.4 SOIL AND ROCK CLASSIFICATION SYSTEM

During the completion of borings and test pit excavations, CFLHD Geotechnical personnel collected soil/rock 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. Rock samples were classified based on the stratigraphic structure, rock strength, degree of weathering, and other properties. The rock classification system is summarized in the attached Rock Classification Field Reference.

SOIL CLASSIFICATION CHART & LEGEND

Unified Soil Classification System

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LESS THAN 15% FINES	GW	WELL-GRADED GRAVELS WITH OR WITHOUT SAND
			GP	POORLY-GRADED GRAVELS WITH OR WITHOUT SAND
		GRAVELS WITH 15% OR MORE FINES	GM	SILTY GRAVELS WITH OR WITHOUT SAND
			GC	CLAYEY GRAVELS WITH OR WITHOUT SAND
	SANDS MORE THAN HALF COARSE FRACTION IS FINER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 15% FINES	SW	WELL-GRADED SANDS WITH OR WITHOUT GRAVEL
			SP	POORLY-GRADED SANDS WITH OR WITHOUT GRAVEL
		SANDS WITH 15% OR MORE FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OL	ORGANIC SILTS OR CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CH	INORGANIC CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OH	ORGANIC SILTS OR CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

NOTE: Coarse-grained soils with between 5% and 12% passing the No.200 sieve and fine-grained soils with limits plotting in the gray zone on the plasticity chart have dual classifications.

Abbreviations

▽	Water Level at Time of Drilling
▼	Stabilized Water Level
UC	Unconfined Compression
NR	No Recovery
LL	Liquid Limit
PI	Plastic Index
W	Moisture Content
DD	Dry Density
NP	Non Plastic
NV	No Valve

Central Federal Lands Soil Description

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

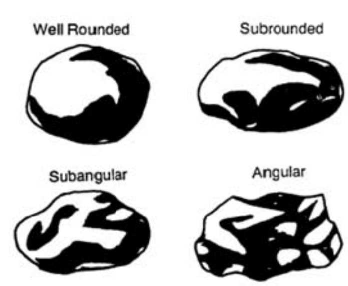
6a. Particle Size

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

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

Category	No.	Item
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?

Category	No.	Item
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)
VATER 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)
VATER LEVEL READINGS – ON VATER	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.

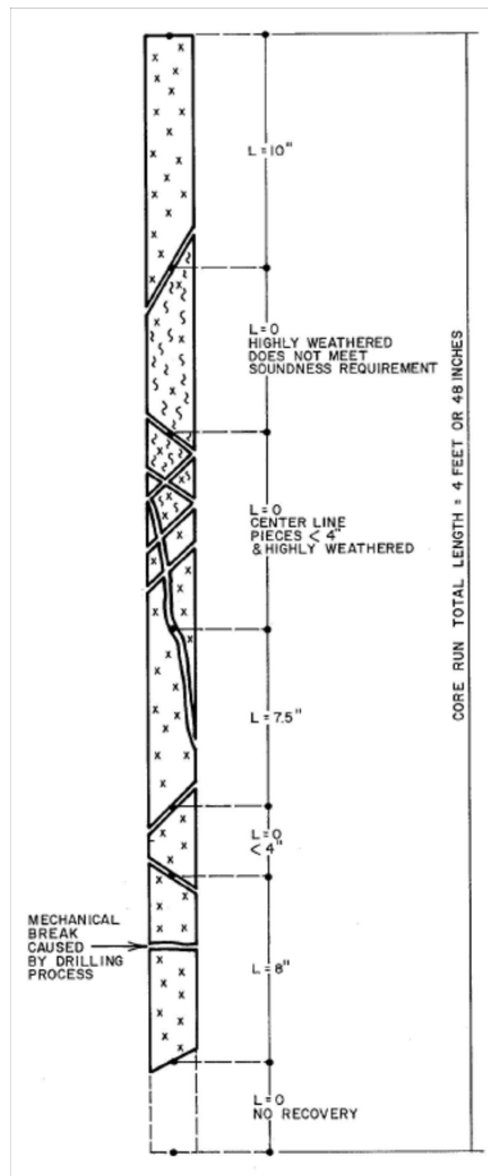
6d. Orientation Discontinuities

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

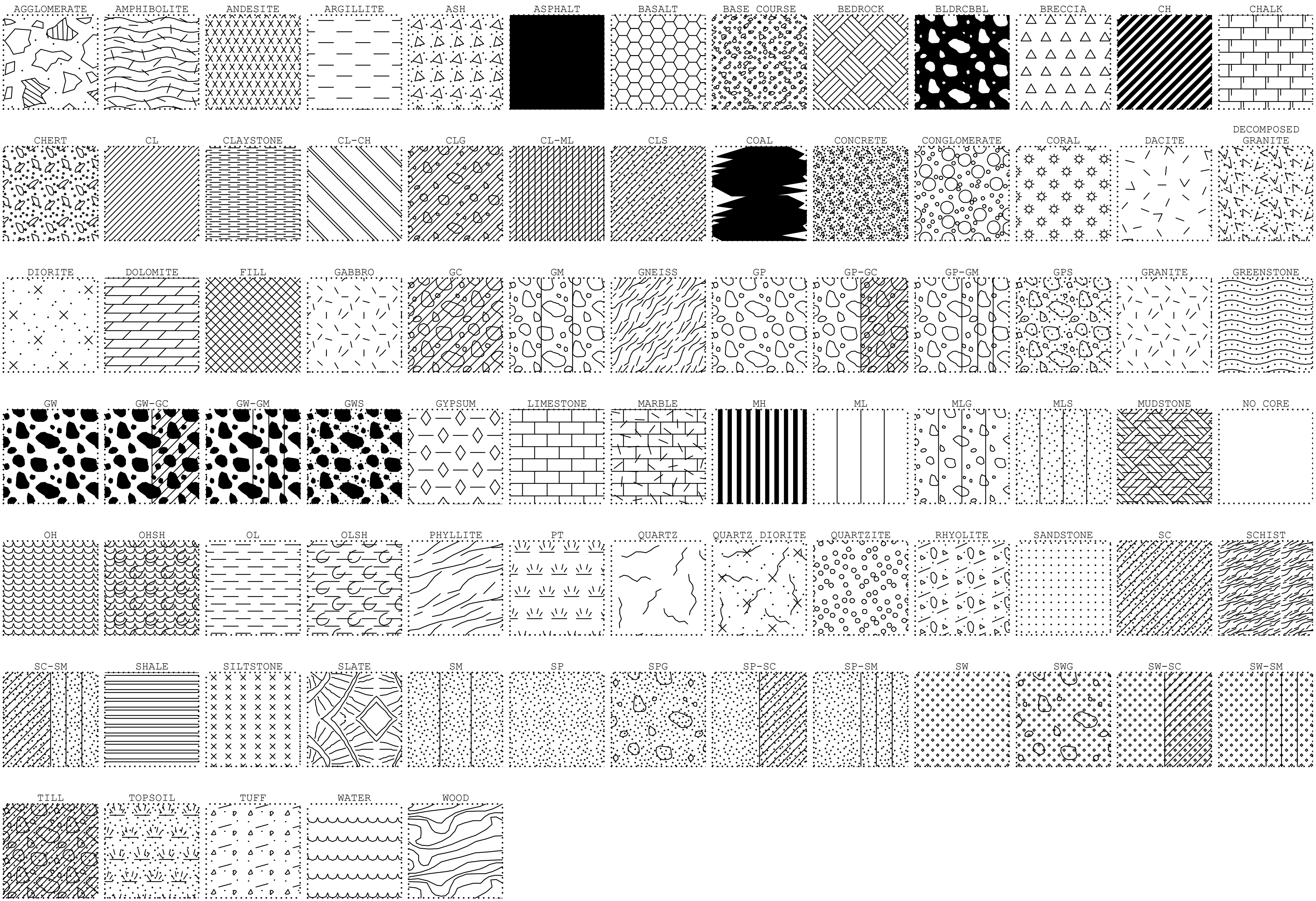
Iron Oxide
Hydrocarbon
None

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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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
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GP-GC
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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
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SW
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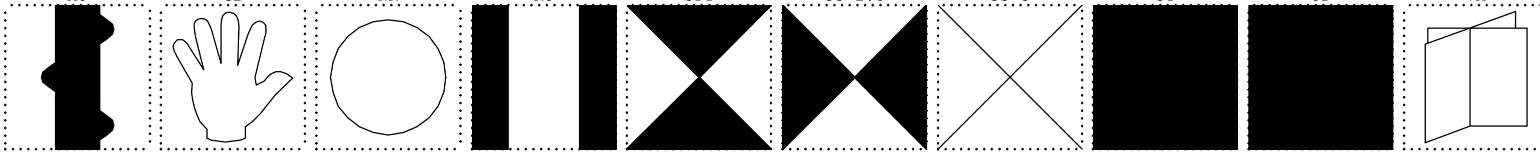
SW-SM
TILL
TOPSOIL
TUFF
WATER
WOOD

AU GB NR RC SPT SS 2.5 SS 3 ST UD VA



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

BORING LOG BH20-01

Project Name:	Buffalo Crossing Bridge		
Project Location:	Apache-Sitgreaves National Forest, Arizona	Surface Elevation:	7541 ft
Groundwater Depth:		Station and Offset:	104+90 8.5 ft Rt.
While Drilling:	---	Latitude:	33.761522° Longitude: -109.357098°
At Completion:	---	Date Started:	9/22/20 Date Completed: 9/22/20
After Drilling:	---	Driller/Company:	C. Fiesler/GSI Drill: CME-75
		Hammer Type:	140 lbs Automatic Hammer Efficiency: 92 %
		Logger/Company:	B. McGarity/FHWA-CFLHD

Notes:
Due to boulders, cored subsurface even though bedrock was not encountered until 15'. Subsurface between 4.5 and 15' is relatively unknown due to no soil samples/drill cuttings.

Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	Drilling Method	Sample Type	No.	Field Blow Count (Recovery) / Core Rec., RQD, and Frac. Freq.	Test Results
7541–7535	0–6	Clayey SAND with gravel (SC), brown, with boulders, dry to slightly moist, angular to subangular gravel.	HSA		01	(12" = 100%)	Fines = 28%
			HSA		02	(12" = 100%)	
	4.5–6	Boulder (~1.5') El. 7535 ft 6 ft	CORE		1	Rec = 50% RQD = 35%	Fines = 28%
7535–7526	6–15	COBBLES AND BOULDERS, Gravelly SILT (ML), Unable to retrieve soil samples in this layer. Classification based off SPT-01 from BH20-02.	CORE		2	Rec = 12% RQD = 0%	
		El. 7526 ft 15 ft	CORE		3	Rec = 54% RQD = 20%	UC = 2030 psi
7526–7515	15–26	TUFF, light red, slightly weathered to moderately weathered, weak rock (R2), very fine to very coarse, angular to subrounded, no discontinuities, slightly vesicular at very top of rock layer.	CORE		4	Rec = 60% RQD = 56%	
		El. 7515 ft 26 ft	CORE		5	Rec = 100% RQD = 92%	UC = 1850 psi

Elevation scale: 7540, 7535, 7530, 7525, 7520, 7515. Depth scale: 5, 10, 15, 20, 25.

N VALUE: 20 40 60 80
PL WC LL: 20 40 60 80
RQD (%) Recovery (%): 20 40 60 80

Auger refusal at 4 ft.
Bottom of borehole at 26 ft.

UC = *660 psi
*This specimen broke in less than the 2 to 15-minute requirement per ASTM D 7012

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

BORING LOG BH20-02

Project Name: Buffalo Crossing Bridge
Sheet: 1 of 1
Project Location: Apache-Sitgreaves National Forest, Arizona
Surface Elevation: 7538 ft
Groundwater Depth:
Station and Offset: 102+35.5 15.5 ft Lt.
While Drilling: ---
Latitude: 33.760839° Longitude: -109.356975°
At Completion: ---
Date Started: 9/21/20 Date Completed: 9/21/20
After Drilling: ---
Driller/Company: C. Fiesler/GSI Drill: CME-75
Notes:
Hammer Type: 140 lbs Automatic Hammer Efficiency: 92 %
Due to boulders, cored subsurface even though bedrock was not encountered until 15'. Subsurface between 1.5 and 16' is relatively unknown due to no soil samples/drill cuttings.
Logger/Company: B. McGarity/FHWA-CFLHD

Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	Drilling Method	Sample Type	No.	Field Blow Count (Recovery) / Core Rec., RQD, and Frac. Freq.	Test Results
	0	Silty GRAVEL with sand (GM), brown, dry, subangular to subrounded gravel. Slow drilling	HSA		01	(12" = 100%)	Fines = 16%
7536.5	1.5	El. 7536.5 ft 1.5 ft					
7535		Clayey SAND with gravel (SC), with boulders. Unable to retrieve soil samples in this layer. Classification based off infill in core run and GRAB-02 in BH20-01. Boulder encountered at 1.5' depth (~2').	CORE		1	Rec = 44% RQD = 15%	
7532	6	El. 7532 ft 6 ft					
7530		COBBLES AND BOULDERS, Gravelly SILT (ML), brown to dark brown, moist to wet (likely from drill fluid).	CORE		2	Rec = 26% RQD = 7%	
	10				01	4-6-11 (13" = 72%)	
7525			CORE		3	Rec = 40% RQD = 13%	Fines = 60%
7522	16	El. 7522 ft 16 ft					
7520		TUFF, light red, moderately weathered, weak rock (R2), medium to very coarse, angular to subangular, no discontinuities, slightly rough.	CORE		4	Rec = 77% RQD = 47%	
7515	20–25		CORE		5	Rec = 90% RQD = 82%	UC = 1490 psi UC = 1250 psi
7512	26	El. 7512 ft 26 ft					

Chart columns: ● N VALUE (20, 40, 60, 80); PL, WC, LL (20, 40, 60, 80); RQD (%) and Recovery (%) (20, 40, 60, 80).

Auger refusal at 1.5 ft.
Bottom of borehole at 26 ft.

SUBSURFACE DIAGRAM

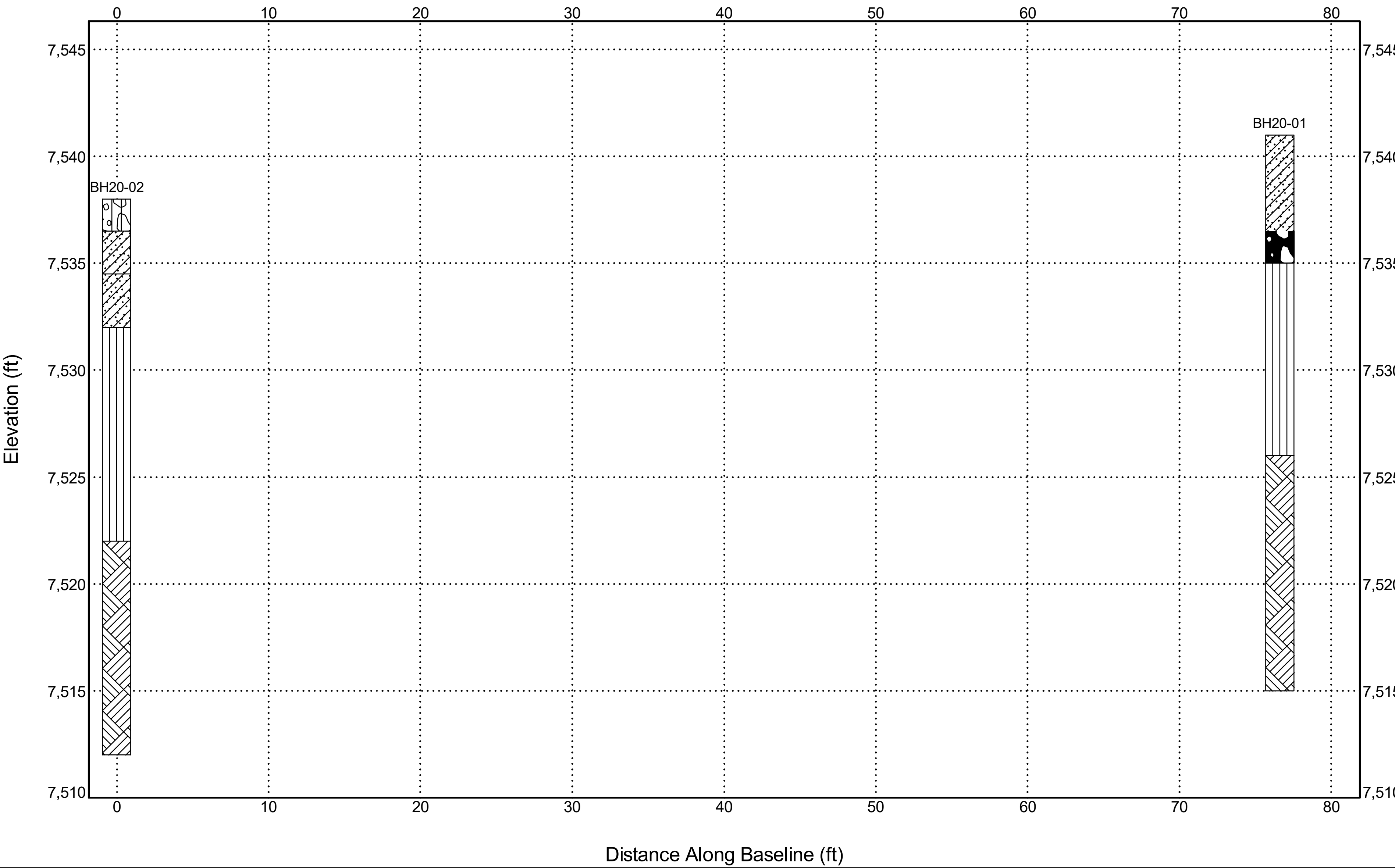
U.S. Department of Transportation
Federal Highway Administration

CLIENT USDA Forest Service, Region 3

PROJECT NUMBER AZ FS 24(1)

PROJECT NAME Buffalo Crossing Bridge

PROJECT LOCATION Apache-Sitgreaves National Forest, Arizona



STRATIGRAPHY & GW - A SIZE - FHWA_DATATEMPLATE_20171103.GDT - 1/7/21 09:05 - N:\AZ\24(1)\GEOTECH\3_DATA\GINT LOGS\AZ FS 24(1) BUFFALO CROSSING BRIDGE.GPJ

GRL Dynamic Measurements and Analyses
engineers, inc.

August 14, 2020

Mike Shelquist, Operations Manager - Phoenix
Geomechanics Southwest, Inc.
5435 West Mohave Street
Phoenix, Arizona 85043

Re: Energy Measurement for Dynamic Penetrometers
Standard Penetration Tests (SPT)
Phoenix, Arizona GRL Job No. 202033-1

Dear Mr. Mike Shelquist:

This report transmits our findings from energy measurements and related data analysis conducted by GRL Engineers, Inc. (GRL) for your seven drill rigs, with measurements collected at your yard in Phoenix, Arizona. Seven automatic hammers and penetrometer systems were monitored during Standard Penetration Tests. Dynamic testing summarized in this report was conducted on Sunday, May 17, 2020.

A Pile Driving Analyzer® Model 8G recorded, processed and displayed the dynamic data to meet the objectives of the hammer system calibration. Discussions on the test methods, limitations and implementation are provided in Appendix A. The energy measurement results are summarized in Tables 1A through 1G, with the average and standard deviation provided in Appendix B. Representative plots of force and normalized velocity are provided in Appendix C.

EQUIPMENT

Hammer and Penetrometer System

Energy measurements were recorded during standard penetration tests conducted for seven automatic hammers and the following drill rig types and serial numbers.

Rig Number (Reference)	Drill Rig Type	Drill Rig Serial Number
128 (A)	CME 75	279177
113 (B)	CME 75	238027
109 (C)	CME 75	225107
120 (D)	CME 75	255137
97 (E)	CME 75	234517
118 (F)	CME 85	381236
67 (G)	CME 75	228107

Measurements were recorded for one boring location for each of the seven drill rigs. Geomechanics Southwest, Inc. (GSI) advanced the penetrometer to a minimum depth of an approximate 20 feet prior to energy measurements. The instrumented subassembly was connected to the top of the drill rod string and measurements recorded at intervals for no less than five depths of data.

Measurements were recorded for every blow required to advance the sampler 18 inches or terminated upon encountering refusal conditions. Results are provided for the final 12 inches of the sampler advancement alone (i.e., excluding the initial 6 inches of advancement) with the exception of depths with less than 18 inches of advancement (i.e., encountering refusal conditions). ASTM Standard D4633 states that tests for energy evaluation should be limited to SPT N-values between 10 and 50. Energy measurements of samples not meeting the lower bound N-value of 10 (i.e., corrected N-value(s)) have been excluded from the averages reported herein.

The following drill rod dimensions, of rod size AWJ, were employed during testing.

For Drill Rigs A, B, C

Drill Rod Area sq. inch			Outside Diameter Inch			Inside Diameter inch		
1.20			1.75			1.23		
Depth of Penetrometer feet			Drill Rod Section Lengths * feet			Transducer to Penetrometer Length * feet		
A	B	C	A	B	C	A	B	C
20.0	21.5	20.0	20	25	20	23.7	28.3	23.3
22.5	23.0	22.5	25	25	25	28.7	28.3	28.3
25.0	25.0	25.0	25	25	25	28.7	28.3	28.3
27.5	27.5	27.5	30	30	30	33.7	33.3	33.3
30.0	30.0	30.0	30	30	30	33.7	33.3	33.3
n/a	n/a	32.5	n/a	n/a	35	n/a	n/a	38.3

* A (CME 75 Serial Number 279177) with adapter from AW to AWJ;
B (CME 75 Serial Number 238027); C (CME 75 Serial Number 225107).

For Drill Rigs D, E, F

Drill Rod Area sq. inch			Outside Diameter Inch			Inside Diameter inch		
1.20			1.75			1.23		
Depth of Penetrometer feet			Drill Rod Section Lengths * feet			Transducer to Penetrometer Length * feet		
D	E&F	G	D	E&F	G	D	E&F	G
20.0	20.0	20.0	20	20	20	23.8	23.3	23.3
22.5	22.5	22.5	25	25	25	28.8	28.3	28.3
25.0	25.0	25.0	25	25	25	28.8	28.3	28.3
27.5	27.5	27.5	30	30	30	33.8	33.3	33.3
30.0	30.0	30.0	30	30	30	33.8	33.3	33.3

* D (CME 75 Serial Number 255137); E (CME 75 Serial Number 234517);
F (CME 85 Serial Number 381236); G (CME 75 Serial Number 228107).

Instrumentation
A Pile Driving Analyzer® was employed for recording, processing, and displaying the dynamic data. An instrumented subassembly, inserted at the top of the drill rod string below the hammer and anvil system and above the drill rods, recorded force and acceleration data. The subassembly was instrumented with two foil strain gages in a full bridge circuit and two piezoresistive accelerometers attached on diametrically opposite sides of the subassembly. Data sampling frequency was 50.0 kHz.

The 8G utilizes a digital system, and with the employed sampling frequency of 50.0 kHz, the signal conditioning conforms to ASTM D4633. Results for the maximum hammer operating rate, rod top force and velocity, and transferred energy are provided in Appendix B and summarized in Tables 1A through 1G. Discussions on the test method and its limitations can be found in Appendix A.

MEASUREMENTS AND CALCULATIONS

The primary objective of testing was the measurement of the energy transmitted from the hammer impact through the anvil into the instrumented subassembly and drill rods. Strain transducers and accelerometers were employed for the calculation of the transferred energy using force, F(t) and velocity v(t), records as follows:

$$EMX = \int_b^a F(t)v(t)dt$$

where time "b" is to the beginning of the energy transfer and time "a" is to the time at which the energy transfer reaches a maximum. Force is calculated as the product of the measured strain, elastic modulus and cross-sectional area, and measured acceleration is integrated to velocity.

Integrated over the complete impact event and calculated from measured force and velocity, the energy transferred to the top of the drill rod was calculated as a function of time. The maximum transferred energy (i.e., EMX or also referred to as EFV) is used as an indicator of the energy content of the event. The described method is the only theoretically correct method of measuring energy transfer and automatically corrects for rod non-uniformities such as connector masses or loose joints. The EF2 method results included in Appendix B are inherently incorrect and included in the appendices for reference alone.

TEST RESULTS

Result Discussion

Dynamic data was evaluated for the hammer operating rate, rod top force and velocity, and transferred energy. Appendix B provides the evaluated quantities for blows making up the SPT N-value, with their averages and standard deviation, plotted and printed as a function of depth for the monitored sequences of the standard penetration tests. Measurements collected for all but one sample (i.e., CME 75 Serial Number 225107 at 30.0 feet) are presented herein.

The plots in Appendix B include:

- FMX – the maximum measured rod top force
- VMX – the maximum measured rod top velocity
- BPM – the hammer operating rate in blows per minute
- BLC – the equivalent penetration resistance or count of impacts per each 6 inches set
- EFV – the maximum calculated energy (EMX) transferred to the rod top
- EF2 – the maximum of the integral of the square of force, theoretically incorrect energy transfer calculation

The corresponding tables also include:

- ETR – ratio of transferred energy (EFV) to the maximum theoretical potential energy
- CSX – the maximum measured rod top compressive stress, averaged over the cross-sectional area

The maximum theoretical potential energy is the product of the standard 140-pound hammer impact mass dropped the standard 30 inches.

A representative plot of force and normalized velocity versus time for a typical blow from each presented data set is provided in Appendix C to demonstrate the data quality.

Summary of Results

I. Seven automatic hammers were monitored during standard penetration tests conducted on May 17, 2020. The average energy transfer ratios calculated with the EFV method for the monitored sequences are tabulated below together with the corresponding average hammer operating rate(s).

Rig Number (Reference)	Energy Transfer Ratio percent	Operating Rate bpm
128 (A)	88	53
113 (B)	88	56
109 (C)	92	55
120 (D)	91	58
97 (E)	88	48
118 (F)	91	55
67 (G)	92	59

II. The uncorrected N-values encountered during monitored sequences ranged from 6 blows (i.e., CME 75 Serial Number 225107 at 30.0 feet) to refusal conditions.

III. To convert the uncorrected N-values for the employed hammer and penetrometer system and operators, the Schmertman correction for adjustment to 60 percent transfer efficiency is

$$N_{60} = \left(\frac{e_m}{60}\right) N_m$$

where N_{60} is the corrected hammer N-value, e_m is the percent energy transfer efficiency (i.e., e_m = 100*ETR) and N_m is the measured SPT N-value. N_{60} values for all measurements and monitored depths are presented in Tables 1A through 1G. The measured overall energy transfer ratios tabulated above for the respective drill rigs produce an N_{60} equivalent of roughly $1.5N_m$. Further corrections due to overburden stresses in the soil have not been considered herein but may be made prior to use of the N-values for design purposes.

We appreciate the opportunity to be of assistance to you on this project. Please contact our offices should you have any questions regarding the contents of this report, or if we may be of further service.

Respectfully,
GRL ENGINEERS, INC.

Camilo Alvarez, MSCE, P.E.
Arizona
Senior Engineer

Anna M. Klesney, MSCE, E.I.T.
Project Engineer

APPENDIX B

LABORATORY TEST RESULTS

APPENDIX B

LABORATORY TEST RESULTS

B.1 INTRODUCTION

Laboratory test were completed on select soil and rock samples recovered for 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 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 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 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 the corrosivity of soils. Geotechnical engineering property test results are presented in the attached laboratory reports. The following section describes the test procedures for 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.

B.4 GEOTECHNICAL ENGINEERING PROPERTY TESTS FOR ROCK

Geotechnical engineering property testing for rock included the uniaxial compression test. Geotechnical engineering property test results are presented in the attached laboratory reports. The following section describes the test procedures for rock.

Uniaxial Compression Test

The uniaxial compression test of select rock samples was performed in general accordance with ASTM D 2938. In this test, cylindrical rock specimens are tested in compression without lateral confinement. The results of these tests are presented as graphs showing stress versus strain and are used in a variety of geotechnical engineering analyses, including slope stability, foundation design, and earth retention design.

Central Federal Lands Highway Division Laboratory

U.S. Department of Transportation
Federal Highway Administration

An AASHTO and ISO Accredited Laboratory

Report of Soil or Aggregate Tests

AASHTO ACCREDITED

Project: Arizona FS 24(1) Buffalo Crossing Bridge

Submitted By: Devin Dixon

Date Reported: 12/23/2020

Sample Number	Lab Number		20-1735-SB	20-1737-SB	20-1736-SB	20-1738-SB
	Hole Number		BH20-01	BH20-01	BH20-02	BH20-02
	Field Number		GRAB-01	GRAB-02	GRAB-01	SPT-01
Sample Location	Station or Location		Buffalo Crossing Bridge	Buffalo Crossing Bridge	Buffalo Crossing Bridge Abutment	Buffalo Crossing Bridge Abutment
	Depth	Feet	0-2	4	0-2	11-12.5
AASHTO T 11 & T 27 Washed Sieve % Passing	3"	75.0 mm	100			
	1 1/2"	37.5 mm	96		100	
	1"	25.0 mm	91	100	97	100
	3/4"	19.0 mm	84	98	89	94
	1/2"	12.5 mm	78	95	77	84
	3/8"	9.5 mm	75	95	69	78
	#4	4.75 mm	69	92	54	64
	#10	2.00 mm	59	85	39	63
	#16	1.18 mm	55	80	34	63
	#40	425 µm	47	69	28	62
	#50	300 µm				
	#100	150 µm	36	41	21	60
	#200	75 µm	28	28	16	60
AASHTO T 255	Moisture, %		10.6	17.7	3.2	19.2
AASHTO T 89 & T 90	Liquid Limit		38	31	NV	28
	Plasticity Index		14	13	NP	3
Soil Classification	AASHTO M 145		A-2-6 (1)	A-2-6 (0)	A-1-b (0)	A-4 (0)
	ASTM D 2487		SC	SC	GM	ML
AASHTO T 190	R - Value					
AASHTO T 288	Min. Resistivity, ohm x cm			2,790		
AASHTO T 289	pH			8.4		
AASHTO Method	Optimum Moisture, %					
	Maximum Dry Density, pcf					

Distribution:	Num. / Project File
Geotechnical	Devin Dixon
Const Ops Engineer	Kalynn Scott
Project Manager	Michael Daigler
Technical Services	Gary Strike

Remarks: Sulfate and Chloride test results will follow.
BH 20-01 GRAB-01 and GRAB-02 were combined in order to perform AASHTO T 288 and AASHTO T289.

Reported By: For

Patrick Kowing
Laboratory Team Leader

Form FHWA 1702 Rev. 8/2014

Central Federal Lands Highway Division Laboratory

U.S. Department of Transportation
Federal Highway Administration

An AASHTO and ISO Accredited Laboratory

AASHTO ACCREDITED

Report of Soil or Aggregate Tests

Revised 2/18/2021

Project: Arizona FS 24(1) Buffalo Crossing Bridge

Submitted By: Devin Dixon

Date Reported: 12/23/2020

Sample Number	Lab Number		20-1735-SB	20-1737-SB	20-1736-SB	20-1738-SB	
	Hole Number		BH20-01	BH20-01	BH20-02	BH20-02	
	Field Number		GRAB-01	GRAB-02	GRAB-01	SPT-01	
Sample Location	Station or Location		Buffalo Crossing Bridge	Buffalo Crossing Bridge	Buffalo Crossing Bridge Abutment	Buffalo Crossing Bridge Abutment	
	Depth	Feet	0-2	4	0-2	11-12.5	
AASHTO T 11 & T 27 Washed Sieve % Passing	3"	75.0 mm	100				
	1 1/2"	37.5 mm	96		100		
	1"	25.0 mm	91	100	97	100	
	3/4"	19.0 mm	84	98	89	94	
	1/2"	12.5 mm	78	95	77	84	
	3/8"	9.5 mm	75	95	69	78	
	#4	4.75 mm	69	92	54	64	
	#10	2.00 mm	59	85	39	63	
	#16	1.18 mm	55	80	34	63	
	#40	425 µm	47	69	28	62	
	#50	300 µm					
	#100	150 µm	36	41	21	60	
	#200	75 µm	28	28	16	60	
AASHTO T 255	Moisture, %		10.6	17.7	3.2	19.2	
AASHTO T 89 & T 90	Liquid Limit		38	31	NV	28	
	Plasticity Index		14	13	NP	3	
Soil Classification	AASHTO M 145		A-2-6 (1)	A-2-6 (0)	A-1-b (0)	A-4 (0)	
	ASTM D 2487		SC	SC	GM	ML	
AASHTO T 190	R - Value						
AASHTO T 288	Min. Resistivity, ohm x cm			2,790			
AASHTO T 289	pH			8.4			
AASHTO Method	Optimum Moisture, %						
	Maximum Dry Density, pcf						
AASHTO T 290	Sulfate Content, ppm / %			20 / 0.002			
AASHTO T 291	Chloride Content, ppm / %			4 / 0.0004			

Distribution:	Num. / Project File
Geotechnical	Devin Dixon
Const Ops Engineer	Kalynn Scott
Project Manager	Michael Daigler
Technical Services	Gary Strike

Remarks: This report was revised to include AASHTO T 290 Sulfate Content and AASHTO T 291 Chloride Content test results. This testing was performed by Colorado Analytical, a FHWA consultant.
BH 20-01 GRAB-01 and GRAB-02 were combined in order to perform AASHTO T 288 and AASHTO T289.

Reported By:

Patrick K

Patrick Kowing
Laboratory Team Leader

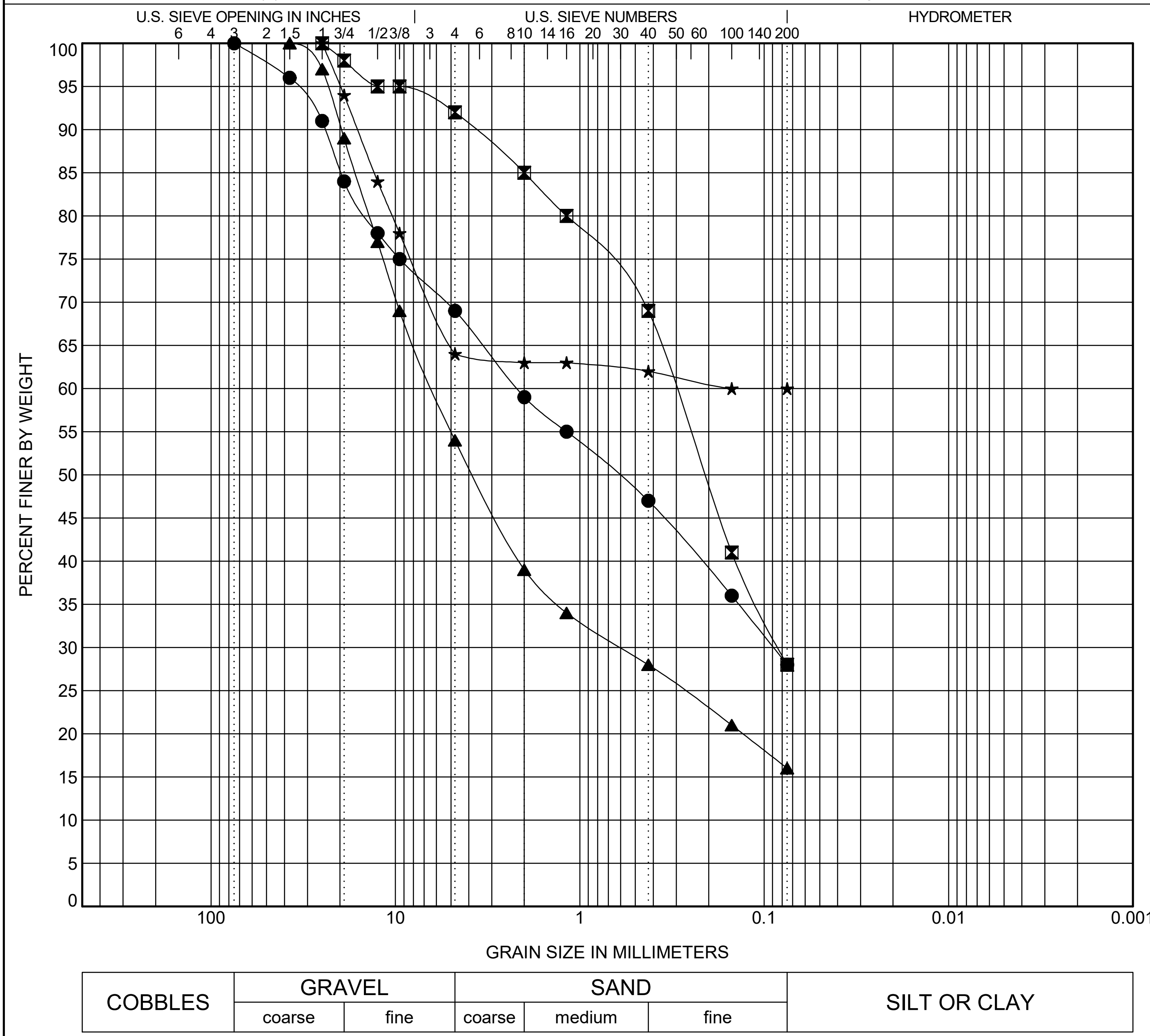
Form FHWA 1702 Rev. 8/2014

GRAIN SIZE DISTRIBUTION

U.S. Department of Transportation
Federal Highway Administration

CLIENT USDA Forest Service, Region 3 **PROJECT NAME** Buffalo Crossing Bridge

PROJECT NUMBER AZ FS 24(1) **PROJECT LOCATION** Apache-Sitgreaves National Forest, Arizona



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**BH20-01**	**0.0**	**CLAYEY SAND with GRAVEL(SC)**	**38**	**24**	**14**		
⊠	**BH20-01**	**4.0**	**CLAYEY SAND(SC)**	**31**	**18**	**13**		
▲	**BH20-02**	**0.0**	**SILTY GRAVEL with SAND(GM)**	**NV**		**NP**		
★	**BH20-02**	**11.0**	**GRAVELLY SILT(ML)**	**28**	**25**	**3**		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH20-01**	**0.0**	**75**	**2.181**	**0.089**		**31.0**	**41.0**	**28.0**	
⊠	**BH20-01**	**4.0**	**25**	**0.304**	**0.083**		**8.0**	**64.0**	**28.0**	
▲	**BH20-02**	**0.0**	**37.5**	**6.268**	**0.597**		**46.0**	**38.0**	**16.0**	
★	**BH20-02**	**11.0**	**25**	**0.075**			**36.0**	**4.0**	**60.0**	

U.S. Department of Transportation
Federal Highway Administration

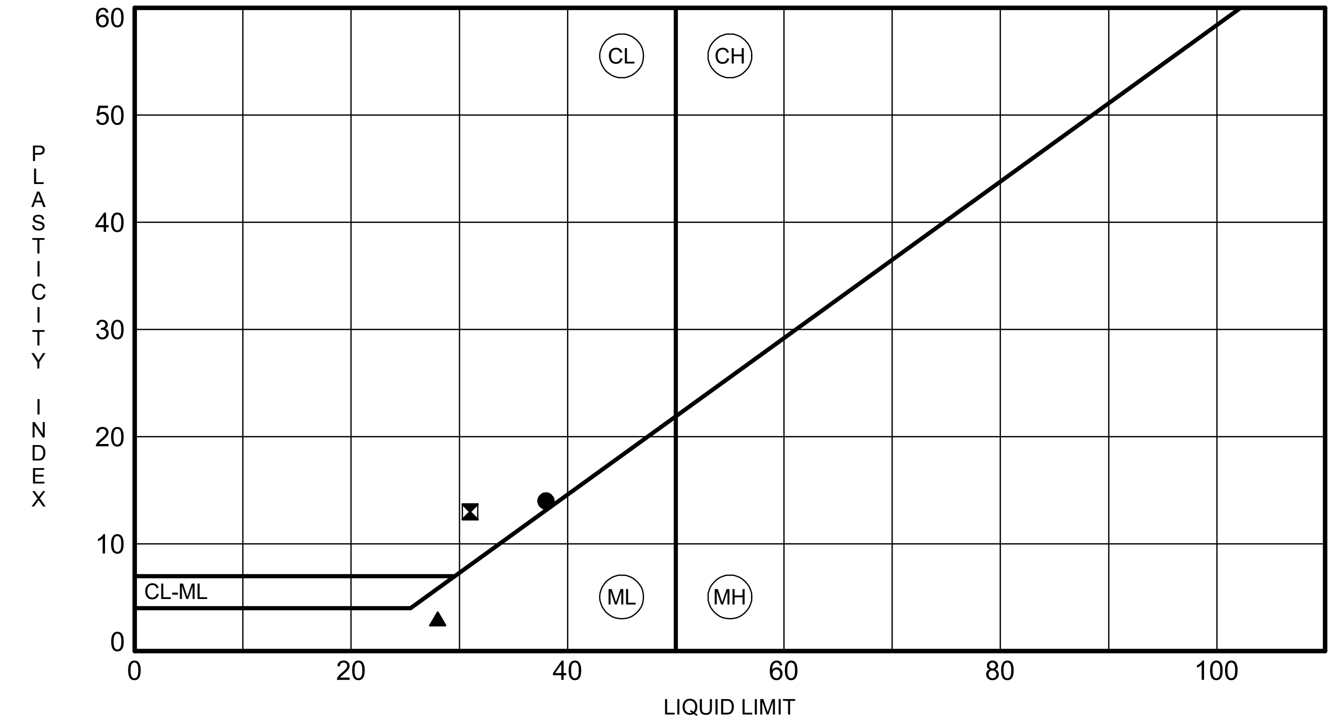
ATTERBERG LIMITS' RESULTS

CLIENT USDA Forest Service, Region 3

PROJECT NAME Buffalo Crossing Bridge

PROJECT NUMBER AZ FS 24(1)

PROJECT LOCATION Apache-Sitgreaves National Forest, Arizona



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	**BH20-01**	**0.0**	**38**	**24**	**14**	**28**	**CLAYEY SAND with GRAVEL(SC)**
⌧	**BH20-01**	**4.0**	**31**	**18**	**13**	**28**	**CLAYEY SAND(SC)**
▲	**BH20-02**	**11.0**	**28**	**25**	**3**	**60**	**GRAVELLY SILT(ML)**

U.S. Department of Transportation
Federal Highway Administration

Central Federal Lands Highway Division Laboratory

An AASHTO and ISO Accredited Laboratory

Report of Miscellaneous Tests

AASHTO ACCREDITED

Project: Arizona FS 24(1) Buffalo Crossing Bridge

Date Reported: 12/18/2020

Lab Number: See Below

Submitted By: Devin Dixon

Material Type: Rock Core

Material Source: Bridge

Field Sample Number: See Below

Tested For: ASTM D 7012 Method C, Compressive Strength of Intact Rock Core Specimen

Laboratory Number	Hole Number	Station	Offset	Sample Depth (Feet)	Specimen Diameter (Inches)	Specimen Length (Inches)	L/D Ratio	Total Load (Lbf)	Compressive Strength (psi)
20-2066-C	BH20-01 #1	Not Furnished	Not Furnished	11-16	2.37	5.76	2.4	8,940	2,030
20-2067-C	BH20-01 #2	Not Furnished	Not Furnished	21	2.39	5.62	2.4	8,320	1,850
20-2068-C	BH20-01 #3	Not Furnished	Not Furnished	26	2.38	5.95	2.5	2.930	*660
20-2069-C	BH20-02 #1	Not Furnished	Not Furnished	21	2.39	5.86	2.5	5,606	1,250
20-2070-C	BH20-02 #2	Not Furnished	Not Furnished	22	2.40	5.92	2.5	6,740	1,490

* Note: This specimen broke in less than the 2 and 15-minute requirement. ASTM D 7012 Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures, 9.4.1.1 which states "The axial load shall be applied continuously and without shock until the load becomes constant, is reduced, or a predetermined amount of strain is achieved. The load shall be applied in such a manner as to produce either a stress rate between 0.5 and 1.0 MPa/s or a strain rate as constant as feasible throughout the test. The stress rate or strain rate shall not be permitted at any given time to deviate by more than 10 % from that selected. The stress rate or strain rate selected shall be that which will produce failure of a cohort test specimen in compression, in a test time between 2 and 15 min. The selected stress rate or strain rate for a given rock type shall be adhered to for all tests in a given series of investigation." Since the specimen has a considerably lower compressive strength the 2-minute specification was not met. The reported results may differ from results obtained from a test specimen that meets the requirements.

Distribution:	Num. / Project File
Geotechnical	Devin Dixon
Const Ops Engineer	Kalynn Scott
Project Manager	Michael Daigler
Technical Services	Gary Strike

Reported By:

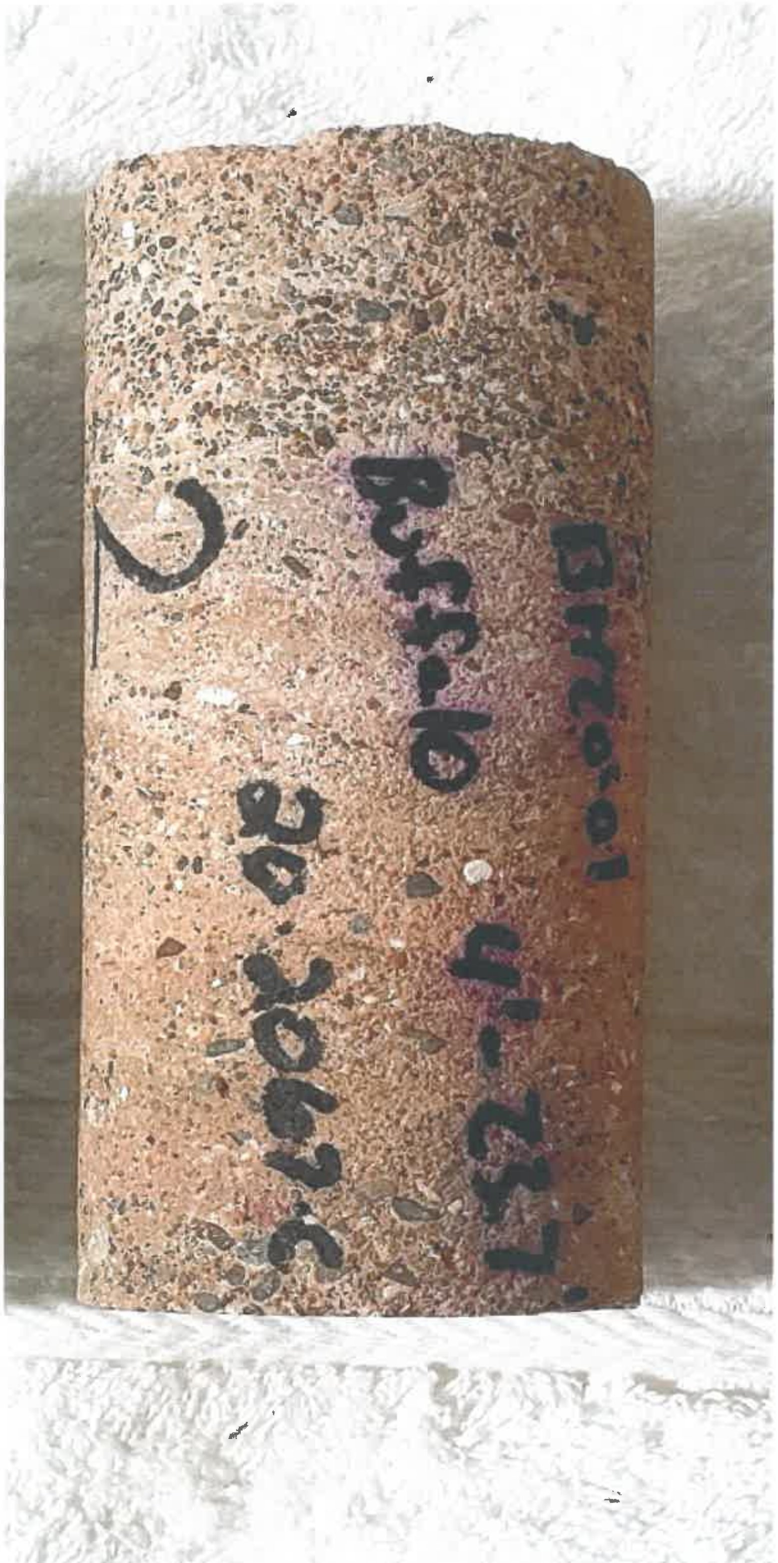
Patrick K

Patrick Kowing
Laboratory Team Leader

20-2066-C BH 20-01 #1 @ 11'-16'

20-2066-C BH 20-01 #1 @ 11'-16'

Arizona FS 24(1) Buffalo Crossing Bridge



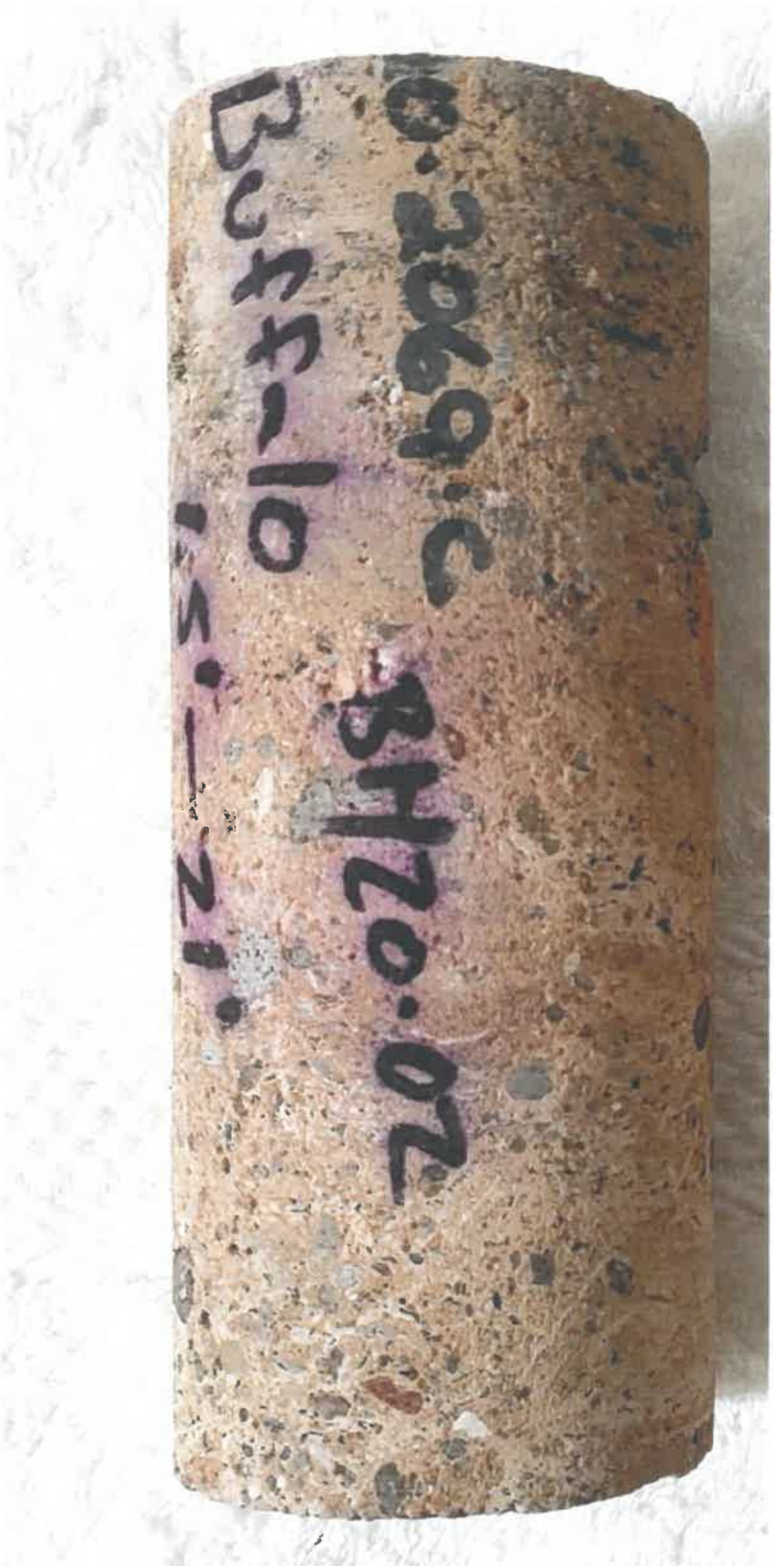
20-2067-C BH 20-01 #2 @ 21.0’

Arizona FS 24(1) Buffalo Crossing Bridge

20-2067-C BH 20-01 #2 @ 21.0’

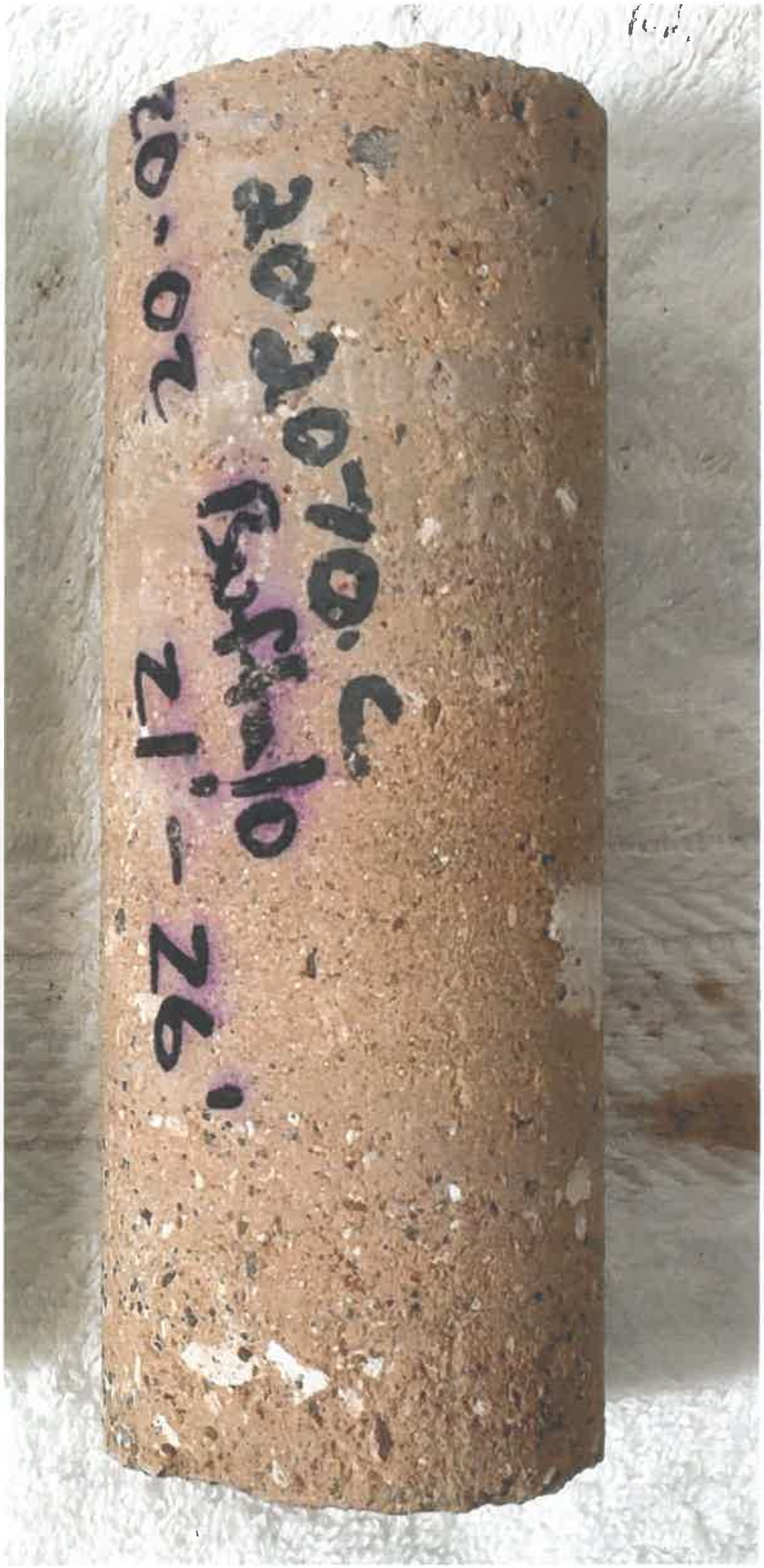
20-2068-C BH 20-01 #3 @ 26’

20-2068-C BH 20-01 #3 @ 26’

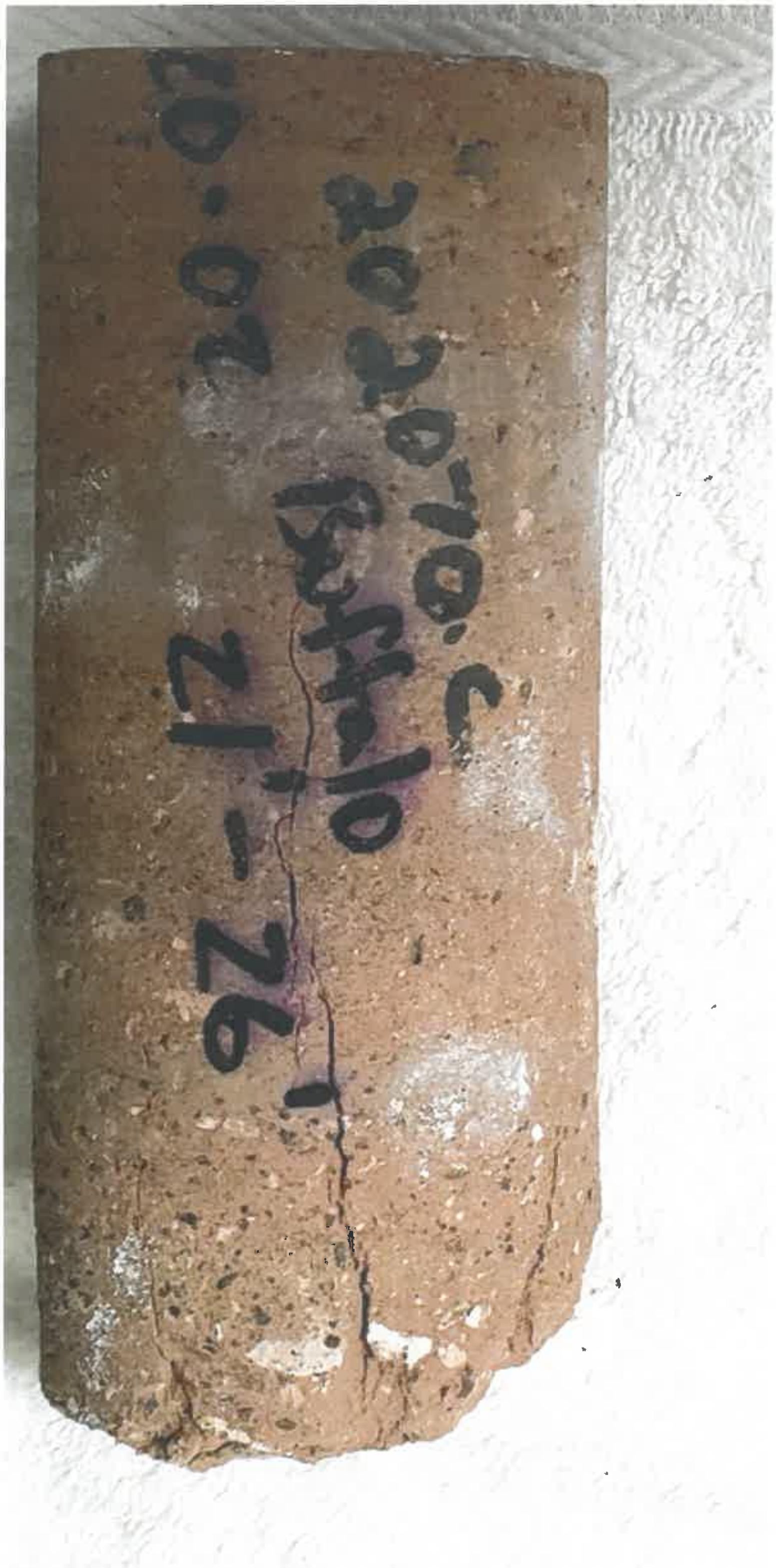


20-2069-C BH 20-02 #1 @ 21’

20-2069-C BH 20-02 #1 @ 21’



20-2070-C BH 20-02 #2 @ 22’



20-2070-C BH 20-02 #2 @ 22’

APPENDIX C

PHOTOGRAPHS

Site Reconnaissance
& Subsurface Investigation

SITE RECONNAISSANCE PHOTOS

Photo 1: South Approach (facing north)

Photo 2: North Approach (facing south)

Photo 3: Buffalo Crossing Bridge (facing northwest)

Photo 4: Rock Outcroppings in Cut Slope Northwest of Bridge Site

Photo 5: South Abutment

Photo 6: Rock Embankment at Southeast Corner of South Abutment

Photo 7: North Abutment

Photo 8: Rock Outcroppings, Embankment, & Rock Baskets Along Northeast Bank

Photo 9: Rock Baskets Near Northeast Corner of North Abutment

Photo 10: East Fork Black River (looking southwest)

Photo 11: East Fork Black River (looking northeast)

SUBSURFACE INVESTIGATION PHOTOS

Boring BH20-01 (North Abutment): Photos 12-19
&
Boring BH20-02 (South Abutment): Photos 20-28

Photo 12: Drilling Setup, Boring BH20-01

Photo 13: Hollow Stem Auger Drilling, Boring BH20-01

Photo 14: Rock Coring, Boring BH20-01

Photo 15: Core Run #1, Boring BH20-01

(Note: width of whiteboard is 8.5 inches)

Photo 16: Core Run #2, Boring BH20-01

Photo 17: Core Run #3, Boring BH20-01

Photo 18: Core Run #4, Boring BH20-01

Photo 19: Core Run #5, Boring BH20-01

Photo 20: Drilling Setup, Boring BH20-02

Photo 21: Hollow Stem Auger Drilling, Boring BH20-02

Photo 22: Rock Coring, Boring BH20-02

Photo 23: Core Run #1, Boring BH20-02

Photo 24: Core Run #2, Boring BH20-02

Photo 25: SPT-01, Boring BH20-02

Photo 26: Core Run #3, Boring BH20-02

Photo 27: Core Run #4, Boring BH20-02

Photo 28: Core Run #5, Boring BH20-02

APPENDIX D

EXCAVATION
CHARACTERISTICS OF ROCK &
RIPPABILITY CHARTS

(Provided by Various Sources)

Table D.1: Rock Hardness and Excavation Characteristics[1]

Rock Hardness Description	Identification Criteria	Unconfined Compressive Strength		Seismic Compression (P-Wave) Velocity		Excavation Characteristics
		MPa	psi	m/s	f/s	
Very Soft Rock	Material crumbles under firm blows with sharp end of geological pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3-c, thick can be broken by finger pressure.	1.7-3.0	246-435	450-1,200	1,475-3,935	Easy Ripping
Soft Rock	Can just be scraped with a knife; indentations 1-mm to 3-mm show in specimen with firm blows of the pick point; has dull sound under hammer.	3.0-10.0	435-1,450	1,200-1,500	3,935-4,920	Hard Ripping
Hard Rock	Cannot be scraped with a knife; hand specimen can be broken with a pick with a single firm blow; rock rings under hammer.	10.0-20.0	1,450-2,900	1,500-1,850	4,920-6,070	Very Hard Ripping
Very Hard Rock	Hand specimen breaks with a pick after more than one blow; rock rings under hammer.	20.0-70.0	2,900-10,150	1,850-2,150	6,070-7,050	Extremely Hard Ripping or Blasting
Extremely Hard Rock	Specimen require many blows with geological pick to break through intact material; rock rings under hammer.	> 70.0	> 10,150	> 2,150	>7,050	Blasting

[1]Table from Weaver (1975).

Table D.2: Excavation Characteristics of Rock[2]

Classification Elements	Class I **Very hard ripping to blasting**	Class II **Hard ripping**	Class III **Easy ripping**
	Rock material requires drilling and explosives or impact procedures for excavation may classify as rock excavation (NRCS Construction Spec. 21). **Must fulfill all conditions below:**	Rock material requires ripping techniques for excavation may classify as rock excavation (NRCS Construction Spec. 21). **Must fulfill all conditions below:**	Rock material can be excavated as common material by earth-moving or ripping equipment may classify as common excavation (NRCS Construction Spec. 21). **Must fulfill all conditions below:**
Headcut erodibility index, k_h (NEH628.52)	$k_h \geq 100$	$10 < k_h < 100$	$k_h \leq 10$
Seismic velocity, approximate (ASTM D5777 and Caterpillar Handbook of Ripping, 1997)	> 2,450 m/s (> 8,000 ft/s)	2,150-2,450 m/s (7,000-8,000 ft/s)	< 2,150 m/s (< 7,000 ft/s)
Minimum equipment size (flywheel power) required to excavate rock. All machines assumed to be heavy-duty, track-type backhoes or tractors equipped with a single tine, rear-mounted ripper.	260 kW (350 hp), for $k_h < 1,000$ 375 kW (500 hp), for $k_h < 10,000$ Blasting, for $k_h > 10,000$	185 kW (250 hp)	110 kW (150 hp)

[1]The classification is a general guide and does not prescribe the actual contract payment method to be used, nor supersedes NRCS contract documents. The classification is for engineering design purposes only.

[2]Table from USDA (2012).

USE OF SEISMIC VELOCITY CHARTS[1]

The charts of ripper performance estimated by seismic wave velocities have been developed from field tests conducted in a variety of materials. Considering the extreme variations among materials and even among rocks of a specific classification, the charts must be recognized as being at best only one indicator of rippability.

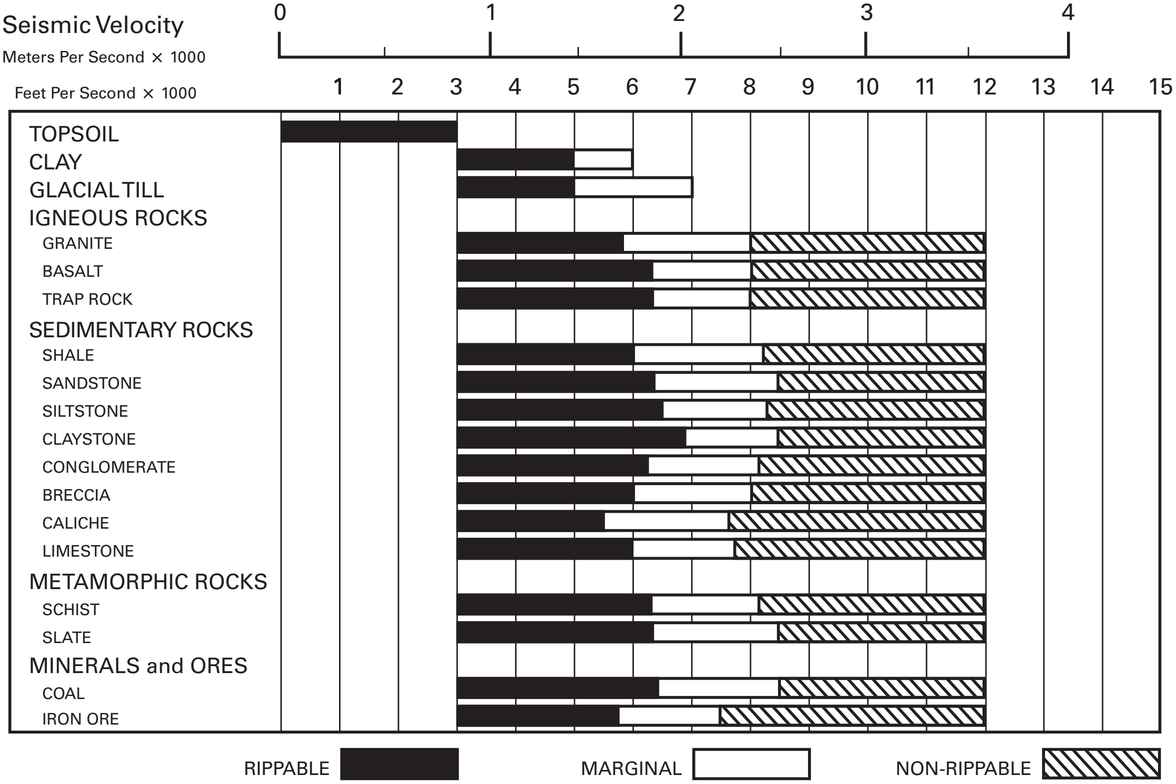
Accordingly, consider the following precautions when evaluating the feasibility of ripping a given formation:

- Tooth penetration is often the key to ripping success, regardless of seismic velocity. This is particularly true in homogeneous materials such as mudstones and claystones and the fine-grained caliches. It is also true in tightly cemented formations such as conglomerates, some glacial tills and caliches containing rock fragments.
- Low seismic velocities of sedimentaries can indicate probable rippability. However, if the fractures and bedding joints do not allow tooth penetration, the material may not be ripped effectively.
- Pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry, particularly in the caliches, conglomerates and some other rocks; but the economics should be checked carefully when considering popping in the higher grades of sandstones, limestones and granites.

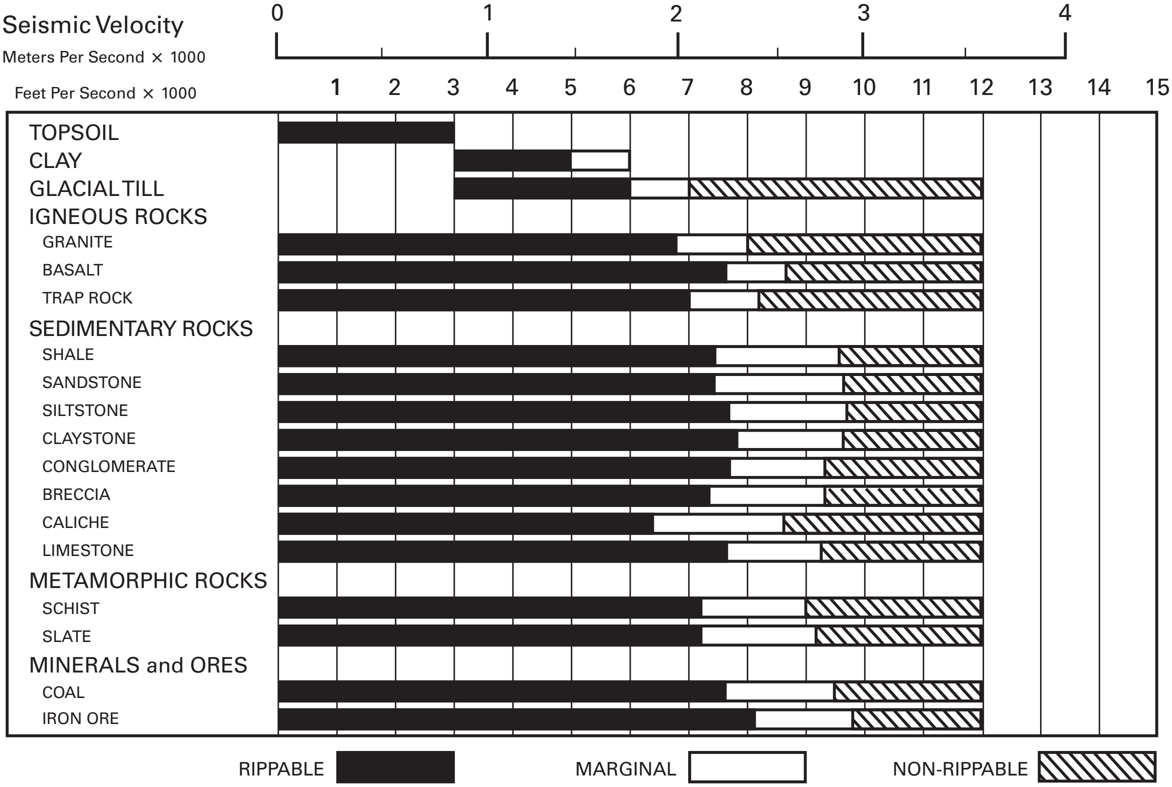
Ripping is still more art than science, and much will depend on operator skill and experience. Ripping for scraper loading may call for different techniques than if the same material is to be dozed away. Cross-ripping requires a change in approach. The number of shanks used, length and depth of shank, tooth angle, direction, throttle position all must be adjusted according to field conditions. Ripping success may well depend on the operator finding the proper combination for those conditions.

[1]Text and the following tables from Hawthorne Cat (2018).

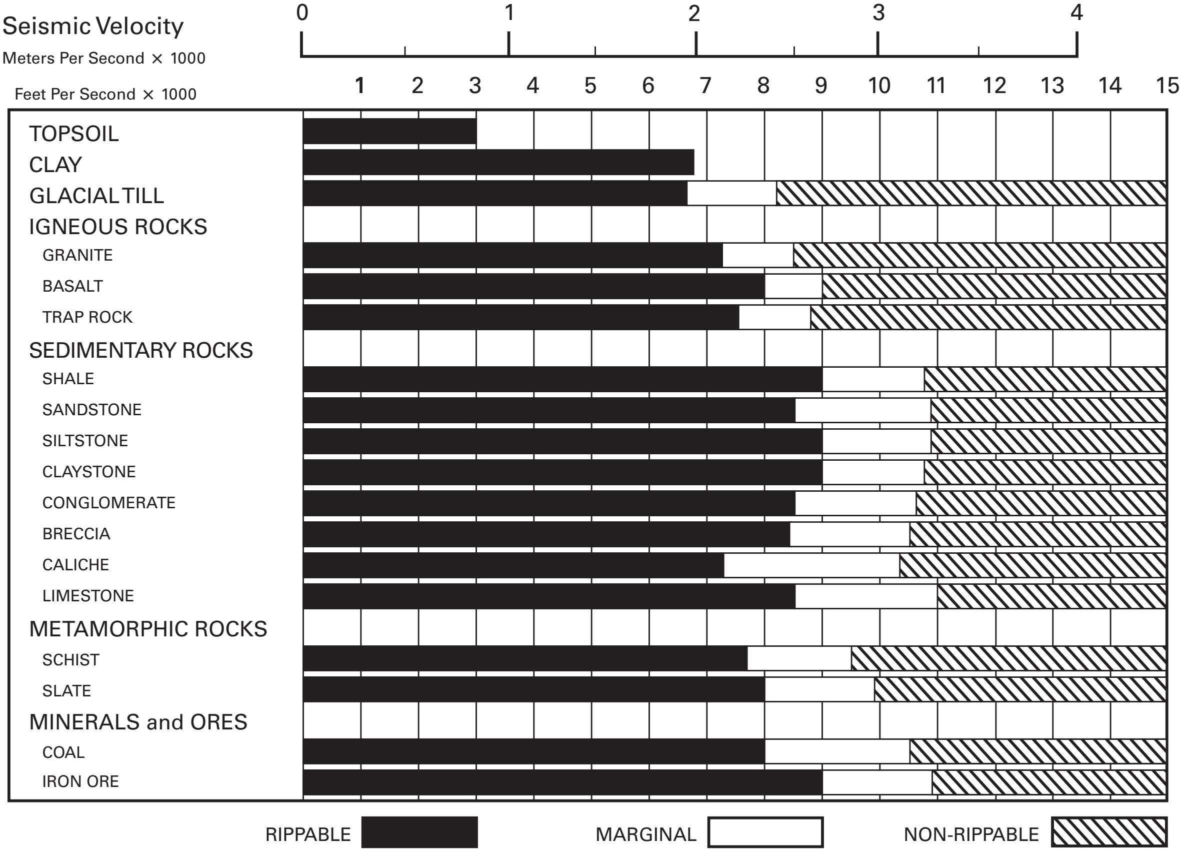
D8R/D8T
● Multi- or Single Shank No. 8 Ripper
● Estimated by Seismic Wave Velocities



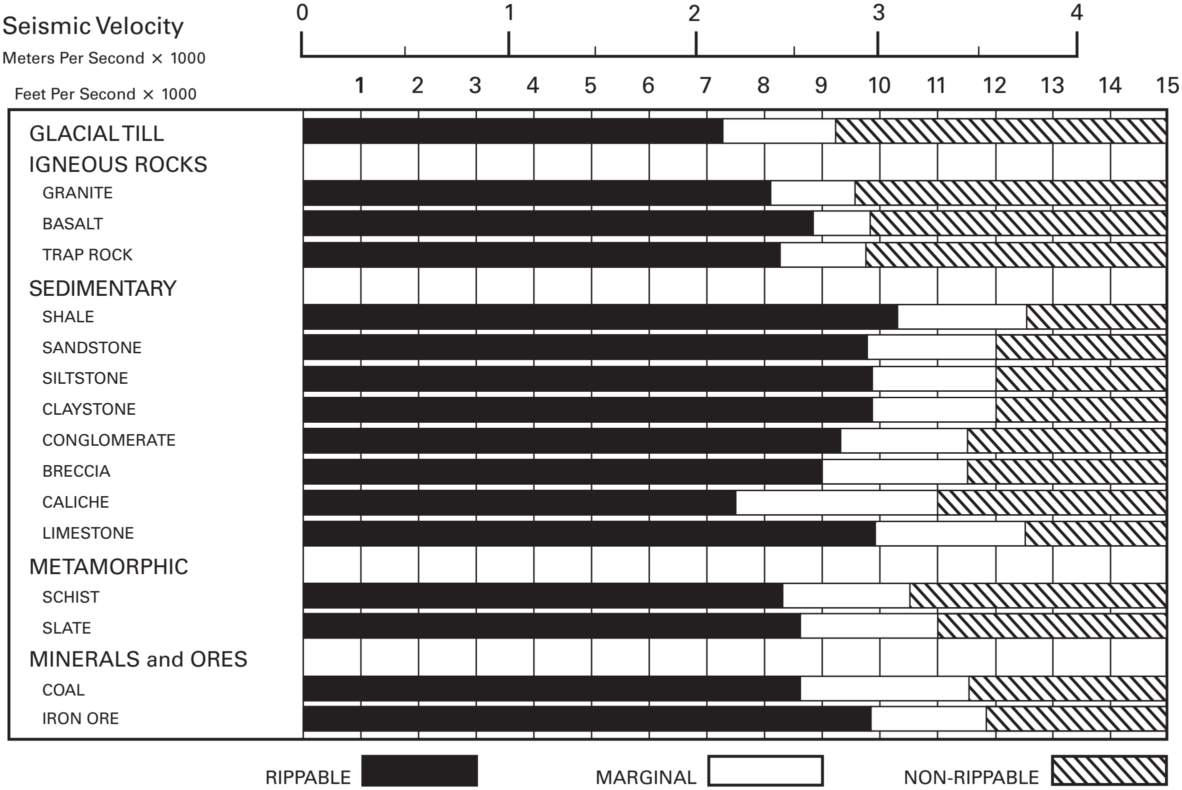
D9R/D9T
● Multi- or Single Shank No. 9 Ripper
● Estimated by Seismic Wave Velocities



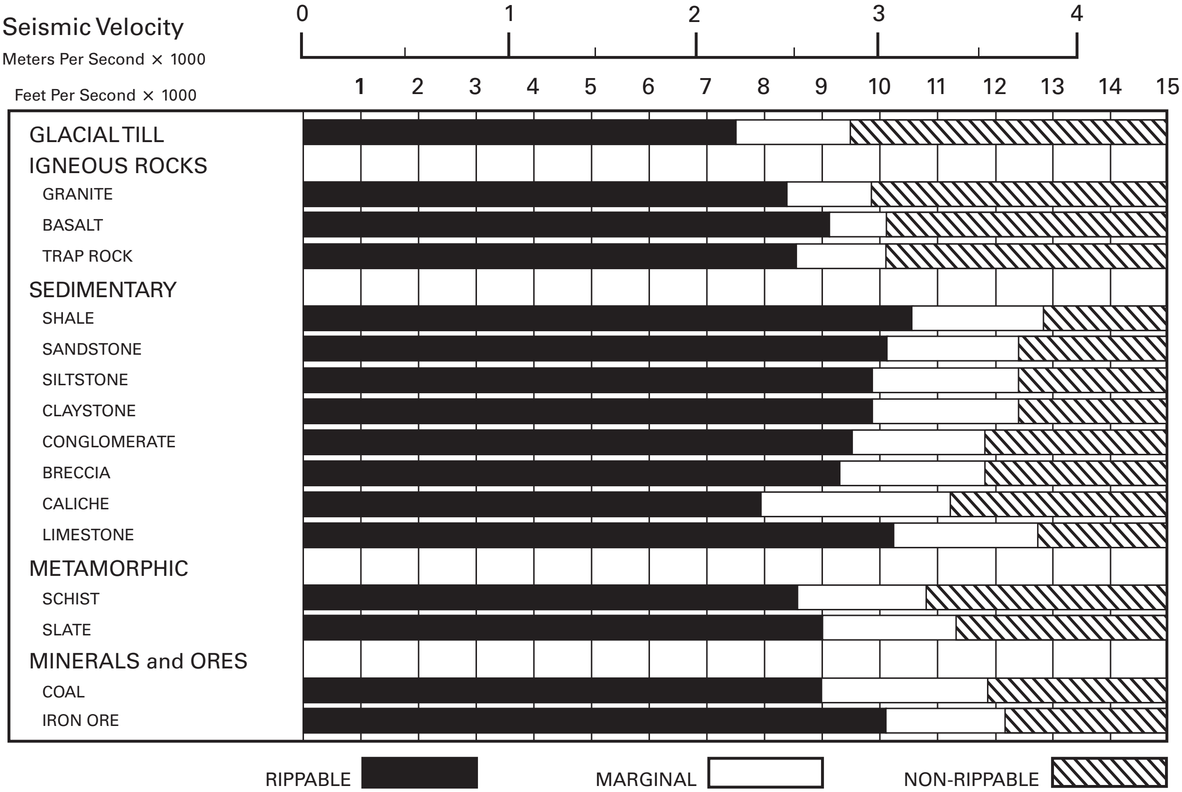
D10T2
- **Multi- or Single Shank No. 10 Ripper**
- **Estimated by Seismic Wave Velocities**



D11T
● Multi- or Single Shank No. 11 Ripper
● Estimated by Seismic Wave Velocities



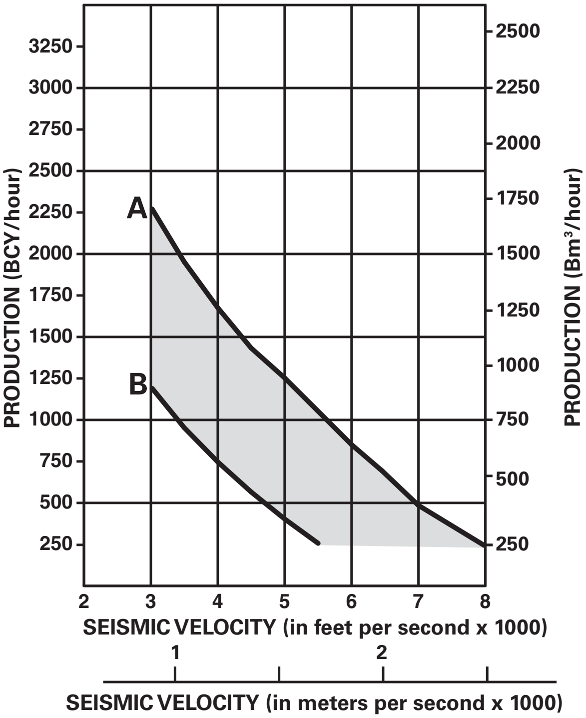
D11T CD
- **Single Shank No. 11 Ripper**
- **Estimated by Seismic Wave Velocities**



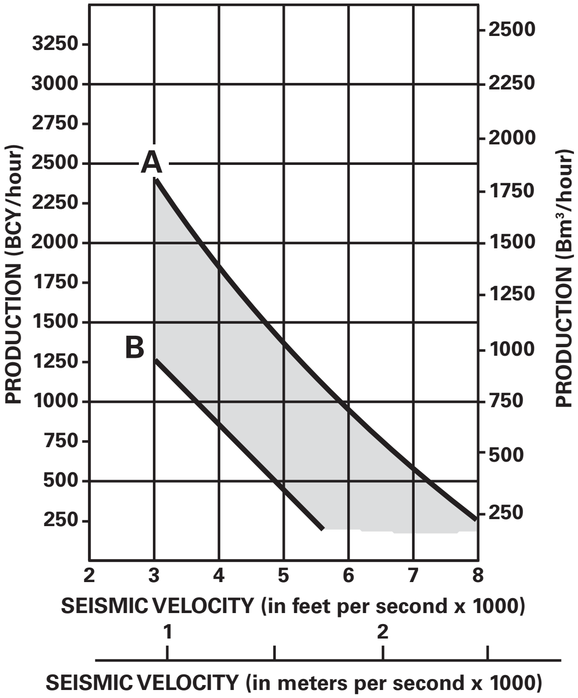
CONSIDERATIONS FOR USING PRODUCTION ESTIMATED GRAPHS:

- Machine rips full-time — no dozing.
- Power shift tractors with single shank rippers.
- 100% efficiency (60 min hour).
- Charts are for all classes of material.
- In igneous rock with seismic velocity of 8000 fps (2450 mps) or higher for the D11T, and 6000 fps (1830 mps) or higher for the D10T2, D9R/D9T and D8R/D8T, the production figures shown should be reduced by 25%.
- Upper limit of charts reflect ripping under ideal conditions only. If conditions such as thick lamination, vertical lamination or any factor which would adversely affect production are present, the lower limit should be used.

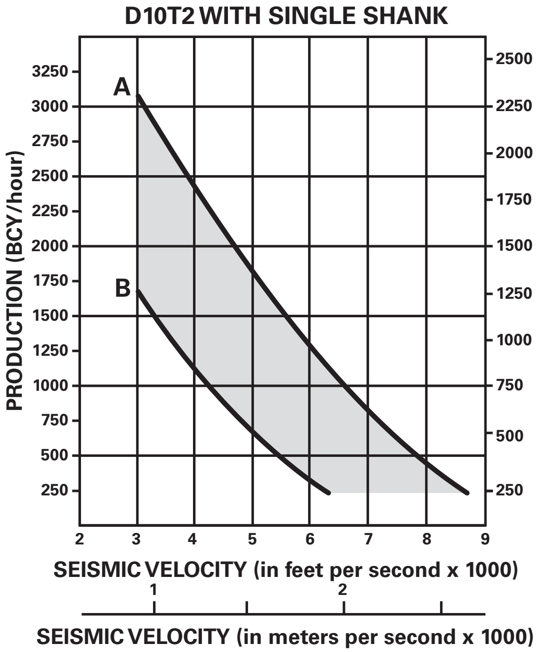
D8R/D8T WITH SINGLE SHANK

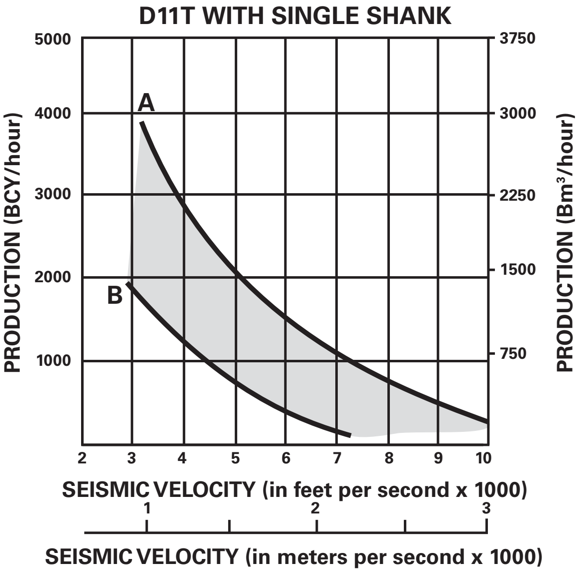


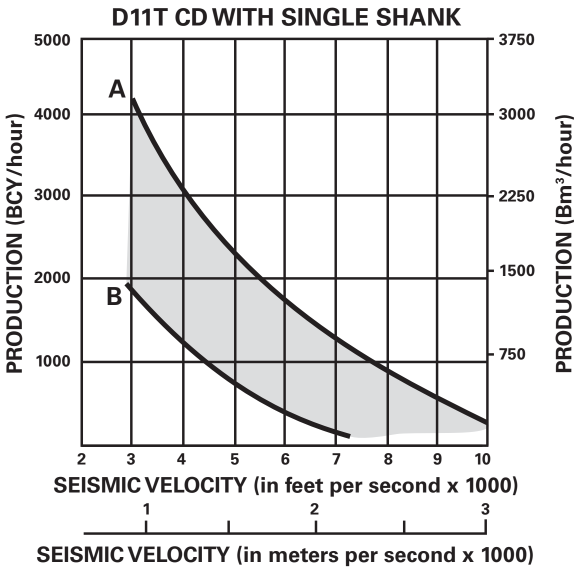
D9R/D9T WITH SINGLE SHANK



KEY
A — IDEAL
B — ADVERSE







KEY

A — IDEAL
B — ADVERSE