Geotechnical Engineering Report
Revision 2

SH-3 St. Maries River Bridge
ITD Project No. A023(257); Key No. 23257
Shoshone County, Idaho

for
David Evans and Associates, Inc.

December 16, 2025

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GeoEngineers

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230.01 Introduction

230.01.01 OVERVIEW

This revised Geotechnical Engineering Report presents the results of our geotechnical engineering evaluation to support the design of the proposed replacement bridge structure supporting State Highway 3 (SH-3) over the St. Maries River located in Shoshone County, Idaho. This crossing is located at approximate Milepost (MP) 57.4 on SH-3, adjacent to the Cedar Creek Campground, about 3.4 miles north of the city of Clarkia and 11.1 miles southeast of the City of Santa. The replacement bridge is planned to improve safety, increase the longevity and reduce maintenance of the existing bridge. The site location is shown on the Vicinity Map, Figure A-1, the structure location is shown on the Site Plans, Figure A-2, in Appendix A.

230.01.02 PROPOSED SH-3 OVER ST. MARIES RIVER BRIDGE

The proposed SH-3 over St. Maries River bridge will consist of a single-span, steel plate girder bridge to be supported on integral abutments founded on driven steel pile foundations. The proposed roadway consists of two, 12-foot lanes; two, 7-foot shoulders; and 3-tube curb-mounted rails. The bridge will have a total width of about 48 feet, 3 inches which includes overbuild to accommodate traffic staging and a length of 120 feet, 8 inches. New abutments will be constructed a minimum of 5 feet behind the existing abutments to provide clearance during construction. The proposed replacement bridge is planned on approximately the same alignment as the existing bridge. The location of the proposed bridge is shown in plan view in Figure A-2 and in elevation view on the Subsurface Profile, Figure A-3, in Appendix A.

TABLE 1. PRELIMINARY SH-3 OVER ST. MARIES RIVER BRIDGE DESIGN INFORMATION

BRIDGE ELEMENT	SH-3 OVER ST. MARIES RIVER BRIDGE
Number of Spans	1
Approximate Total Bridge Length	120 feet, 8 inches
Approximate Bridge Width	48 feet, 3 inches
Abutment Type	Integral Abutment
Approach Slabs	20 feet-long
Approximate Bottom of Abutment Pile Cap Elevation	±2,764.5 feet (Abutments 1 and 2)
Preliminary Foundation Loads per abutment	1,391 kips (Service) 1,019 kips (Strength)

230.01.03 PROPOSED WING WALLS

The bridge abutments and wing walls will be constructed with cast-in-place concrete and will be supported by the bridge foundations. No separate wing wall foundations are planned.

230.02 Field Exploration and Laboratory Testing

230.02.01 BORINGS

We drilled one boring (GEI-1-24) at the south abutment of the existing bridge and one boring (GEI-2-24) at the north abutment of the existing bridge. We also installed one piezometer (GEI-3-24) off the northbound shoulder near the north abutment. We also collected bulk streambed sediment samples for laboratory testing to aid in the evaluation of scour.

Drilling was accomplished with a truck-mounted CME 75 hollow-stem auger drill rig. GeoEngineers' personnel maintained logs of the borings and visually classified soils encountered according to the Unified Soil Classification System (USCS) as described in the Idaho Transportation Department (ITD) Materials Manual.

The locations and elevations of the borings were surveyed by HMH Engineering. A summary of the boring locations, depths and elevations is presented in Table 2. The boring locations are also shown in Figure A-2. A key to the exploration logs is presented in Figure B-1, an explanation of bedrock terms is presented in Figure B-2 and the explorations logs are presented in Figures B-3 through B-5 in Appendix B. Appendix D includes photographs of rock core collected in our borings (Figures D-1 and D-2) and streambed sediment sample locations (Figures D-3 and D-4).

TABLE 2. BORING LOCATION, DEPTH AND ELEVATION SUMMARY

STRUCTURE	BORING ID	SURFACE ELEVATION, WGS84 (FEET)	DEPTH (FEET)	SURVEYED LOCATIONS			
				GEOGRAPHIC COORDINATE SYSTEM NAD83		PROJECT COORDINATES CF = 1.0001480766	
				LATITUDE	LONGITUDE	NORTHING	EASTING
SH-3 over St. Maries River Bridge	GEI-1-24	2,774.8	60.5	47.051034	-116.287691	1963253.87	2490621.12
	GEI-2-24	2,774.2	64	47.051296	-116.287638	1963382.71	2490660.44
	GEI-3-24	2,772.2	18.5	47.051344	-116.287597	1963390.58	2490674.50

230.02.02 FIELD TESTS

Samples of soil encountered in the borings were obtained using a standard split-spoon sampler (standard penetration test [SPT], ASTM International [ASTM] D1586) and rock core samples were obtained by HQ rock coring. Soil samples obtained with the split-barrel samplers are disturbed but can be used for visual classification and index testing. The N-values recorded from the SPT give an indication of the in-place relative density of the cohesionless soil encountered at the site. However, SPT values for gravel soil can be somewhat unrepresentative—because of particle size—and should be considered with engineering judgment and experience. The results of the field tests are presented on the boring logs in Appendix B. Soils were visually classified in the field per ASTM D2488, Description and Identification of Soils (Visual-Manual Procedure).

230.02.03 GEOPHYSICAL EXPLORATION

Geophysical explorations were not completed for this structure.

230.02.04 LABORATORY TESTS

Representative samples from the borings were tested in the laboratory to evaluate the pertinent properties of the soils. The test procedures were performed in general accordance with the applicable ASTM procedures identified below ("general accordance" means that certain local and common descriptive practices and methodologies have been followed):

- Classification of Soils for Engineering Purposes—ASTM D2487
- Water (Moisture) Content—ASTM D2216
- Particle-Size Analysis of Soils—ASTM D422
- Percent Passing the U.S. No. 200 Sieve—ASTM D1140
- Atterberg Limits—ASTM D4318
- pH—ASTM D4972
- Resistivity—American Association of State Highway and Transportation Officials (AASHTO) T288
- Soluble Sulfates—U.S. Environmental Protection Agency (EPA) 300
- Soluble Chlorides—EPA 300

Laboratory test results are presented in Appendix C and on the boring logs in Appendix B at the respective sample depths.

230.03 Surface Conditions

Vegetation near the existing bridge includes trees, small shrubs and grass. Approximately 70 feet north of the existing north abutment is a set of inactive railroad tracks. West of the bridge is private property, and the Cedar Creek campground is located to the southwest. The existing roadway crossing the bridge consists of two lanes of asphalt. An electrical pedestal is located on the east side of SH-3 north of the St. Maries River, and overhead railroad crossing warning lights and cross arms are located on each side of the St. Maries railroad. The existing ground surface ranges between approximate elevations of 2772 to 2775 feet.

230.04 Subsurface Conditions

230.04.01 GENERAL

The Geologic Map of the St. Maries 30 x 60 Minute Quadrangle (Lewis, R. S., et. al., 2000) maps the surficial deposits in the area as "***Qal*** – Alluvial deposits" from the Holocene era. These deposits include stratified and laterally discontinuous beds of pebbles, cobbles, sand and silt. The soil encountered in our exploration was consistent with the mapped alluvial deposits. Lewis maps the bedrock in the area to be "***Yqw*** – Quartzite of the Wallace Formation" of the Middle Proterozoic era. This quartzite includes fine-grained white quartzite, biotite quartzite, granofels and calc-silicate horizons. The Idaho Geological Survey maps the bedrock in the area to be "***Ymil*** – Lower Missoula Group" from the Mesoproterozoic era. This includes calcareous siltite, dolomitic siltite, quartzite and minor argillite. The rock encountered in the borings was consistent with the mapped rocks.

230.04.02 SOIL

In general, the subsurface profile encountered at the boring locations consisted of existing roadway fill soils overlying alluvium, overlying bedrock. These soils units are described in further detail below. Approximately 3.5 inches of asphalt pavement was encountered in borings GEI-1-24 and GEI-2-24, followed by approximately 6 inches of base course consisting of silty fine to coarse sand with occasional gravel (SM). No asphalt pavement or base course was observed in boring GEI-3-24.

- **Existing Fill.** An existing fill soil unit extended to depths ranging from 1 to 15 feet (Elevations 2775 to 2760 feet) in each of the borings. The fill generally consists of medium stiff sandy clay and clayey sand (CL, SC).
- **Alluvium.** An alluvium unit was observed in each of the borings. The unit extended from depths of about 12.5 to 20 feet (Elevations 2761.5 to 2754 feet) near Abutment 1 and from depths of about 12.5 to 18.5 feet (Elevations 2761.5 to 2755.5 feet) near Abutment 2. This alluvium generally consists of loose to dense sand and gravel with variable silt (SM and SP-SM, GP-GM). Boring GEI-3-24 was terminated in this unit.
- **Weathered Bedrock.** A weathered bedrock unit consisting of decomposed quartzite was encountered below the alluvium layers. Weathered bedrock was encountered at a depth of 20 to 45 feet (Elevations 2754 to 2729 feet) near Abutment 1 in GEI-1-24 and at depth of 17 feet to 44 feet (Elevations 2757 to 2730 feet) near Abutment 2 in GEI-2-24. The decomposed quartzite reworks to very dense silty sand with varying amounts of gravel (SM).
- **Intact Bedrock.** An intact bedrock unit consisting of quartzite was encountered at a depth of 45 to 60.5 feet (Elevations 2729 to 2713.5 feet) near Abutment 1 in GEI-1-24 and at depth of 44 feet to 64 feet (Elevations 2730 to 2710 feet) near Abutment 2 in GEI-2-24. The quartzite ranged from medium hard (R3) to hard (R4) and the rock quality designation (RQD) ranged from very poor to excellent.

We developed a generalized subsurface profile below the proposed bridge which represents our interpretation of the subsurface conditions encountered in the borings. The subsurface profile is presented in Figure A-3.

230.04.03 GROUNDWATER CONDITIONS

Groundwater depths were recorded in borings at the time of exploration and, where encountered, are described on each exploration log. A summary of groundwater conditions observed at the time of drilling is provided in Table 3 below.

Depth to groundwater will vary depending on many variables such as seasonal precipitation, land use and local irrigation practices. Groundwater levels should be expected to fluctuate. Based on the groundwater readings taken at the site and the anticipated variability, our design considers a groundwater level of Elevation 2764 feet, equal to 3 feet above the highest measured groundwater elevation.

TABLE 3. APPROXIMATE GROUNDWATER LEVELS

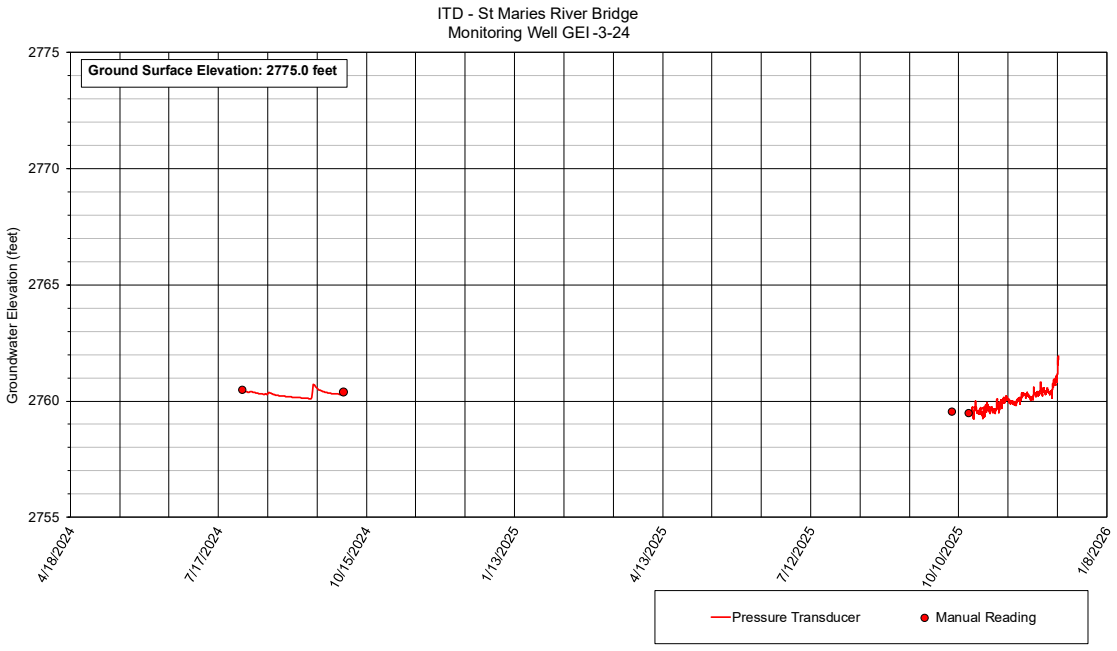
STRUCTURE	EXPLORATION ID	EXPLORATION DATE	APPROX. DEPTH TO GROUNDWATER (FEET)	APPROX. GROUNDWATER ELEVATION (FEET)
St. Maries River Bridge	GEI-1-24	July 9,2024	15[1]	2759[1]
	GEI-2-24	July 10, 2024	14[1]	2760[1]
	GEI-3-24	July 11, 2024	10.2 – 13.0[2]	2759.2 – 2762.0[2]

Notes:

[1] Approximate groundwater measured at time of drilling;

[2] Approximate groundwater measured in piezometer between July 31 to October 1, 2024 and October 16 to December 9, 2025.

Groundwater levels are currently being monitored using a data logger installed within a flush-mounted piezometer at GEI-3-24. Groundwater levels were recorded between July 31, 2024 to October 1, 2024 and October 16, 2025 to December 9, 2025. There was a recording error during the winter, spring and summer of 2025 that caused a loss in data. After a discussion with ITD, GeoEngineers will continue monitoring groundwater through spring/early summer of 2026 to attempt to capture typical seasonal high groundwater levels during spring runoff. The measured groundwater depth has varied between 10.2 and 13.0 feet below the existing ground surface (Elevation 2759.2 and 2762.0 feet, respectively) during the period of observation. The GEI-3-24 groundwater monitoring plot is shown below. The pressure transducer was temporarily removed from the monitoring well between October 1 and 9 of 2024 for maintenance. Recordings were not collected between October 2024 and October 2025. Additional groundwater data will be provided to the team as design progresses.



Groundwater conditions at the site are likely to fluctuate on a number of factors, including precipitation, St. Maries River level, precipitation/runoff, and other forms or natural and artificial groundwater recharge. We anticipate groundwater elevation at the site may fluctuate seasonally on the order of 2 to 3 feet. Groundwater elevations are likely highest during late spring and early summer months during peak runoff season.

230.04.04 FROST DEPTH

The design frost depth for this project is 30 inches based on Shoshone County building code ordinance design criteria.

230.05 Conclusions and Recommendations

230.05.01 GENERAL

Based on the results of our geotechnical engineering evaluation, it is our opinion that construction of the proposed bridge structure and wing walls are feasible from a geotechnical engineering standpoint. We understand the bridge and walls will be designed using Load and Resistance Factor Design (LRFD) methodology. Some key geotechnical considerations for the project include:

- Based on discussions with the bridge designers and for project consistency, we recommend the bridge be supported on driven HP 14x117 steel piles.
- Very dense soil was encountered in the borings in the weathered bedrock unit. Pre-drilling to the highest pile tip elevation should be planned to facilitate pile driving.
- Our analysis indicates an adequate global slope stability factor of safety below the proposed bridge.
- Seismic hazards such as ground rupture, liquefaction or lateral spreading are negligible, and are not expected to impact the foundation design.
- Based on groundwater measurements in the project vicinity and our experience, we anticipate that groundwater elevations could fluctuate 2 to 3 feet or more seasonally, and likely will be highest during late spring or early summer months. The Contractor should be prepared to encounter water in site excavations and be prepared to develop and execute dewatering plans or cofferdams to facilitate installation of scour countermeasures associated with the project.
- Wing walls will be structurally connected to the bridge abutments and will not be supported on independent foundations.

The key geotechnical information provided above is a summary for introductory purposes only. Recommended design foundation capacities, anticipated performance and construction considerations are presented in the following sections. If there are any revisions to the nature, design or location of the proposed structure relative to the current concept, the recommendations presented above and in the following sections should be reviewed by GeoEngineers and modified, as necessary.

230.05.02 FOUNDATIONS

Overview

Based on discussions with the bridge designers and the results of our analyses, we recommend the bridge be supported on driven HP 14x117 steel piles (ASTM A572 Grade 50, fy = 50 kips per square inch [ksi]).

Soil Parameters

Estimated soil properties for design of the bridge are presented in Table 4. These parameters were determined based on the results of our field exploration program and correlations to field and laboratory data. The design contact elevations of the soil units are presented in Table 5 below.

TABLE 4. ESTIMATED SOIL PARAMETERS FOR DESIGN

PARAMETER	EXISTING FILL	ALLUVIUM[2]	DECOMPOSED BEDROCK	INTACT BEDROCK
Effective Unit Weight (pcf)[1]	115 (above GW) 53 (below GW)	120 (above GW) 58 (below GW)	68 (below GW)	78 (below GW)
Friction Angle, ϕ	33°	35°	38°	—
LPILE Soil Model	Stiff Clay with and w/o Free Water (Reese)	API Sand	API Sand	(Not included in model)
Modulus of lateral soil reaction, K (pci)[1]	—	140 (above GW) 85 (below GW)	125 (below GW)	—

Notes:

[1] pcf = pounds per cubic foot;; pci = pounds per cubic inch;

[2] Use a design groundwater elevation equal to 3 feet above the highest groundwater encountered (Elevations 2762 and 2763 feet) in the explorations at each structure. Groundwater elevations are based on the groundwater data in Table 3 of this report.

GW = groundwater

TABLE 5. DESIGN SOIL CONTACT ELEVATIONS

ABUTMENT	CONTACT ELEVATION (FEET)			
	EXISTING FILL	ALLUVIUM	WEATHERED BEDROCK	INTACT BEDROCK
1 (South)	Ground Surface to 2761.5	2761.5 to 2754	2754 to 2729	2729 to 2713.5
2 (North)	Ground Surface to 2761.5	2761.5 to 2756	2756 to 2730	2730 to 2710

Driven Piles

GENERAL

Due to the presence of dense to very dense weathered bedrock, we recommend pre-drilling in accordance with Section 520 of the ITD Standard Specifications to the highest pile tip elevation. Our anticipated pile schedule is presented in Table 6 below. This schedule was developed based on information provided by the structural engineer and our axial and lateral analysis results. The pile lengths are estimated assuming a nominal pile capacity of 700 kips achieved during initial drive. The pile schedule should be updated accordingly if higher or lower pile capacities are necessary. This table should be updated by the structural engineer after final pile cap elevations and design loads are determined.

TABLE 6. ANTICIPATED PILE SCHEDULE

BRIDGE	ABUTMENT	PILE TYPE	APPROX. PILE CUT-OFF ELEVATION (FEET)	ESTIMATED PILE TIP ELEVATION (FEET)	HIGHEST PILE TIP ELEVATION (FEET)	ESTIMATED PILE LENGTH (FEET)	NOMINAL PILE CAPACITY (KIPS)
SH-3 over St. Maries River	1 (South)	HP 14x117	2766	2730	2735	36	700

BRIDGE	ABUTMENT	PILE TYPE	APPROX. PILE CUT-OFF ELEVATION (FEET)	ESTIMATED PILE TIP ELEVATION (FEET)	HIGHEST PILE TIP ELEVATION (FEET)	ESTIMATED PILE LENGTH (FEET)	NOMINAL PILE CAPACITY (KIPS)
SH-3 over St. Maries River	2 (North)	HP 14x117	2766	2730	2735	36	700

AXIAL RESISTANCE OF DRIVEN PILES

We estimated axial resistance of HP 14x117 piles using procedures in Section 10.7.3.8.6 of the AASHTO LRFD Bridge Design Specifications and our experience driving piles tipped into bedrock.

The estimated pile tip elevations to achieve a nominal pile capacity are presented in Table 6 above. Pre-drilling is recommended to the highest pile tip elevation since refusal could occur before the piles achieve the highest pile tip elevation. We expect the pile to achieve tip resistance on rock at approximately Elevation 2730 feet or before. We recommend the piles be fitted with rock points per section 505.03D of the ITD Standard Specifications.

The nominal axial resistance should be multiplied by the appropriate resistance factor as presented in Table 7 to estimate the factored resistance and pile lengths for various limit states.

Actual pile capacity should be determined in the field during driving based on bearing capacity curves developed using wave equation procedures. Given the variabilities with driving piles, we recommend that test piles be driven as described in the “Resistance Factors for Driven Piles” section below in this report.

Downdrag loads from static settlement are anticipated to be negligible.

UPLIFT RESISTANCE OF DRIVEN PILE

We understand that design uplift loads on piles will be low. We have provided a graph of estimated nominal uplift resistance vs. pile embedment depth in Figure E-1. The nominal uplift resistance should be multiplied by the appropriate resistance factor as presented in Table 7 to estimate the factored resistance, and pile lengths for various limit states.

PRELIMINARY DRIVABILITY ANALYSES

The Contractor will be responsible for selecting the appropriate pile driving hammer. Care should be taken when selecting a pile driving hammer such that the driving stresses do not damage the piles during installation. For planning and estimating purposes, we recommend a hammer with a rated energy between 60 to 75 foot-kip be used. Depending on the size of the selected hammer, it may be necessary to use the hammer’s lower fuel settings to keep pile stresses within acceptable limits. Refusal of piles on cobbles, boulders or in the weathered bedrock unit could occur during pile driving. We recommend pre-drilling to the highest pile tip elevation to ensure adequate pile embedment for lateral fixity (as discussed in a following section of this report). Pile refusal is expected on bedrock within about 5 feet of the highest pile tip elevation. These recommendations are provided for planning and cost estimating purposes only. The Contractor should ultimately be responsible for selecting the appropriate pile driving hammer to achieve the design depths without damaging the piles.

SETTLEMENT OF DRIVEN PILES

Based on the results of our analyses and our experience, we estimate that settlement of piles should be less than about ½ inch.

RESISTANCE FACTORS FOR DRIVEN PILES

We recommend the resistance factors presented in Table 7 below be used for the design of driven piles.

TABLE 7. RESISTANCE FACTORS—DRIVEN PILES

LIMIT STATE	RESISTANCE FACTOR		
	BEARING RESISTANCE	UPLIFT	LATERAL
Strength	0.50[1]	0.35/0.5[2]	1.0
Service	1.0	NA	1.0
Extreme	1.0	0.8	1.0

Notes:

[1] The bearing resistance value of 0.50 is based on the assumption that wave equation analysis, without pile dynamic measurement is performed on all production and test piles.

[2] Uplift resistance for single pile, and for pile group, respectively.

Wave equation analysis, without pile dynamic measurement must be performed to develop driving criteria required to confirm nominal resistance for all production and test piles. Final production pile driving criteria should be established by ITD or can be established by GeoEngineers upon request. Refer to Section 6.5.4.2 of the AASHTO LRFD Bridge Design Specifications for strength limit state resistance factors for pile drivability analyses. Severe driving conditions should be assumed when selecting resistance factors.

We recommend two test piles be completed, one at each abutment of the proposed bridge structure. Successful test piles may be used as production piles, provided the piles meet production pile driving criteria.

LATERAL RESISTANCE OF DRIVEN PILES

Response of piles required to resist lateral loads was evaluated using p-y iterative methods using the computer program LPILE (Ensoft). We used LPILE 2022, Version 12.07 to estimate pile response to lateral loads. The items below summarize our input parameters and assumptions. If any of these assumptions are not correct, we should be requested to revise our analysis or the LPILE analysis should be completed by the bridge engineer using the input soil parameters recommended in this report.

- We evaluated HP 14x117, Grade 50 steel piles (ASTM A572 Grade 50) such that fy = 50 ksi.
- We modeled 14x117 HP piles using the “Strong Axis” and “Weak Axis” options in LPILE. We modeled a single pile with loading perpendicular to the pile. P-multipliers were not used. For other pile/loading configurations, additional analyses should be completed to assess group effects using p-multipliers presented in Table 10.7.2.4.1 of the AASHTO LRFD Bridge Design Specifications. The computer program GROUP developed by Ensoft can also evaluate group effects of piles. Alternatively, the lateral load (P) values shown in the lateral resistance graphs should be multiplied by an appropriate P-multiplier for closely spaced piles as required in section 10.7.2.4 of the AASHTO LRFD Bridge Design Specifications.

- We used the input parameters presented in Tables 4 and 5 in LPILE for site soil conditions.
- We estimated pile response to lateral loading by setting the following boundary conditions at the pile head: lateral deflection ranging from 0.25 inches to 2 inches, and a pile slope of zero at the pile top (bottom of pile cap), to model fixed-head conditions.
- Vertical loads were modeled as 172 kips.
- Graphical output from our LPILE analyses is presented in Figures E-2 and E-3 in Appendix E. The structural engineer should assess pile stresses based on the results of lateral and axial loading conditions to confirm adequate pile loading and spacing requirements.

MINIMUM PILE PENETRATION, PILE SPACING AND ESTIMATED PILE LENGTH

Our analyses also assessed embedment lengths required to achieve "fixity" to resist lateral loads. ITD policy is to determine fixity based on the point of zero deflection plus 5 feet. Based on the results of the LPILE analysis for the case assuming resistance from the gravel in the CMP pile sleeve, we recommend minimum embedment depth of 29 feet (below top of pile) for Abutments 1 and 2. The resulting highest pile tip elevations for these embedment depths are included in Table 6 above. Deeper embedment may be needed to resist axial or uplift loads.

We recommend a minimum pile spacing of 3 pile diameters (4 feet, 2 inches center-to-center spacing). Piles may be spaced closer if needed; however, piles spaced closer than 2.5 pile diameters require group effects be considered for evaluation of vertical loads and may be difficult to construct. In addition, if piles are spaced closer than 5 pile diameters (6 feet, 8 inches), p-multipliers should be applied as necessary for evaluation of lateral loads to account for pile spacing following the guidelines in Section 10.7.2.4 of the AASHTO LRFD Bridge Design Specifications.

SEISMIC CONSIDERATIONS FOR PILES

As discussed in "Section 230.05.08, Seismic Design" of this report, the risk of liquefaction at the project site is low. Therefore, we anticipate that liquefaction will not affect the foundation capacities of the proposed structures for design earthquake with a site factored peak ground acceleration (As) of 0.146 g (representing an approximate 975-year design level earthquake or a 7 percent probability of exceedance in a 75-year period).

Global Slope Stability

We evaluated external global stability at the bridge abutments with the computer program SLOPE/W (GeoStudio) using limit equilibrium and Spencer's method of analysis. The soil parameters presented in Table 4 were used in the analyses. We included a traffic surcharge of 260 psf (static) and 130 psf (seismic) as a vertical surcharge pressure. For seismic conditions, we used a vertical seismic coefficient of zero and a horizontal seismic coefficient of 0.073g (equal to half of As) and pseudo-static limit equilibrium methods. Per ITD Materials Manual Section 640.04, we targeted a minimum static factor of safety of 1.5 and a minimum seismic factor of safety of 1.1.

The results of our global slope stability analyses are presented in Table 8. Our results indicate that the proposed abutment slopes meet minimum required safety factors. Illustrations showing the assumed slope configuration and critical failure surfaces are presented in Appendix F as Figures F-1 through F-4.

GeoEngineers should be consulted to update the global stability analyses if the proposed abutment geometries are revised.

TABLE 8. SUMMARY OF GLOBAL SLOPE STABILITY ANALYSIS RESULTS

LOCATION	FACTOR OF SAFETY	
	STATIC	SEISMIC
Abutment 1	1.59	1.38
Abutment 2	1.59	1.37

Spread Footings

Spread footings are not recommended for the support of the bridge structure.

230.05.03 LATERAL EARTH PRESSURES AND BACKFILL

Embankment fill for the bridge approaches and abutment wall backfill material used in conjunction with this project should be free from organic and deleterious materials and should conform to Granular Subbase requirements. Fill material for the abutment wall backfill should be compacted to meet Class A compaction criteria. The lateral earth pressure values for fill material are summarized in Table 9 below. The at-rest (K_0) and passive (K_p) values should be utilized when designing the integral abutment for thermal expansion as described in Section 11.6.1.3 of the ITD Bridge Design LRFD Manual.

TABLE 9. LATERAL EARTH PRESSURES—EXISTING AND PROPOSED EMBANKMENT FILL

PARAMETER	VALUE [1]	
	HORIZONTAL BACKSLOPE	2H:1V BACKSLOPE [2]
Backfill Unit Weight (γ)	130 pcf	130 pcf
Backfill Soil Friction Angle ($\emptyset$)	34°	34°
Soil-Wall Interface Friction Angle (δ) (Active Earth Pressure Only, Rankine methods used for At-Rest and Passive resistance)	11°	11°
Active Earth Pressure Coefficient (K_a)	0.26	0.40
At-Rest Earth Pressure Coefficient (K_o)	0.44	0.64
Passive Earth Pressure Coefficient (K_p)	3.54	10.02 (movement toward upward slope) 1.34 (movement toward downward slope)
Seismic Earth Pressure Coefficient (K_{Ae}) (Active Condition) [3]	0.30	0.52
Seismic Earth Pressure Coefficient (K_{oe}) (At-Rest Condition) [4]	0.48	0.76

Notes:

[1] All values are nominal (unfactored) and should be reduced based on anticipated wall movement per the Chart 1 below.

[2] GeoEngineers should be consulted to revise lateral earth pressure coefficient and equivalent fluid density values if back-slopes with different slope inclinations are planned.

[3] Based on a horizontal acceleration coefficient equal to half the peak ground acceleration (0.5 x A_s) and assumes that wall is allowed to move or translate during design seismic event.

[4] Based on a horizontal acceleration coefficient equal to the peak ground acceleration (As).

H:V = horizontal to vertical

Active, static and seismic earth pressure coefficients should be used for walls that are free to rotate or translate. If wall movement will be less than about 0.001H (where H is the height of the abutment wall) under static conditions, then at-rest static earth pressures should be used for abutment wall design. For the abutments, H should be determined from the bottom of the pile cap to finished grade. The passive pressure coefficients in Table 9 above should be reduced for wall movements less than about 0.05H using the graph (Chart 1) shown below. Corresponding factors should be applied (multiplied) by the lateral earth pressures presented in Table 9.

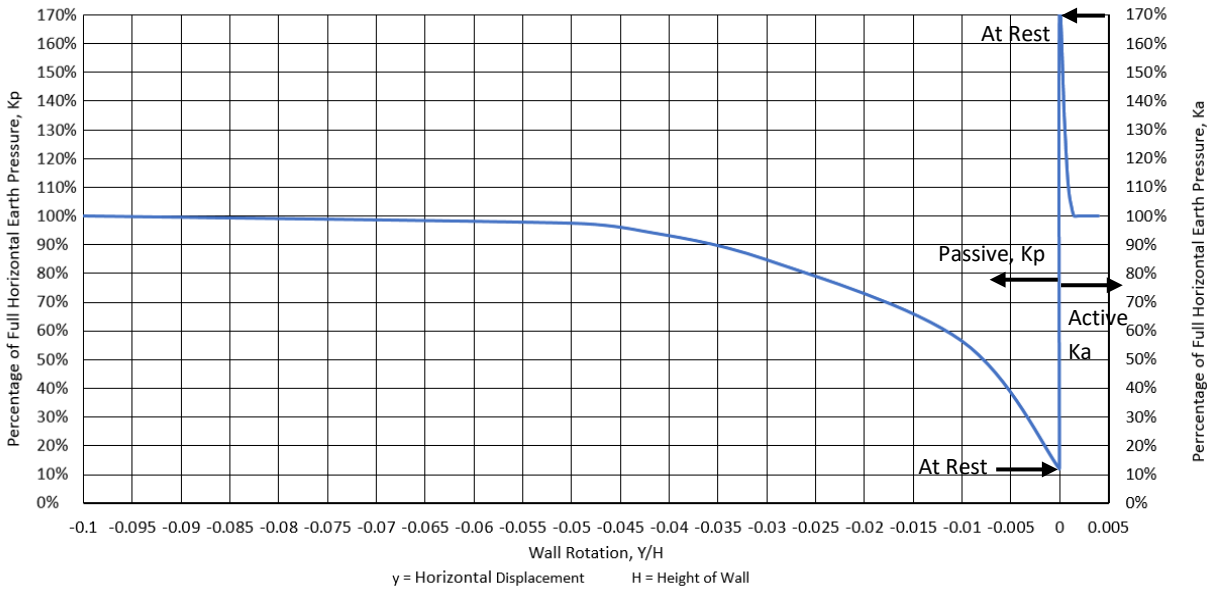


Chart 1. Recommended Lateral Earth Pressure Rotation Factors

In accordance with the AASHTO LRFD Bridge Design Specifications, Section A11.3.1, the combined effect of seismic and static lateral earth pressures can be calculated using a triangular earth pressure distribution. The effective equivalent fluid density can be calculated using the seismic earth pressure coefficient multiplied by the unit weight of the soil. The resultant force is applied at a point equal to H/3 above the bottom of the wall. This earth pressure includes both static and seismic portions. Therefore, static earth pressure should not be included separately. This seismic earth pressure is based on the Mononobe-Okabe method presented in the AASHTO LRFD Bridge Design Specifications.

The lateral earth pressures assume the backfill soils are properly drained and do not include hydrostatic pressures that can develop if groundwater or surface water accumulates behind the retaining structures.

A traffic surcharge load should be included where vehicular load is expected to act on the wall backfill within a distance of half the wall height, in accordance with the AASHTO LRFD Bridge Design Specifications, Section 3.11.6.4. We recommend that a unit weight of 130 pcf and the appropriate (active or at-rest, depending on stiffness of the wall) lateral earth pressure coefficient presented in Table 9 be used to calculate the live load traffic surcharge (LS). Additional surcharge pressures resulting from loads applied at the ground surface should also be included in the design of the bridge and retaining walls. Such earth pressures may be generated by surface surcharge loads, point loads, line loads, and strip loads. Lateral surcharge pressures for uniform surface surcharge loads may be calculated by multiplying the uniform

surcharge by the active or at-rest lateral earth coefficient presented in Table 9. GeoEngineers should be consulted if other surface surcharge loads adjacent to the bridge abutment and walls are expected.

The backfill should be compacted to Class A standards. Overcompaction of abutment or retaining walls adjacent to structures should be avoided because increased compactive effort will result in lateral pressures higher than those provided in this report. Heavy compaction equipment or other construction loads should not be allowed within one-half the height of walls. Hand-operated or lightweight compaction equipment, such as a vibrating plate, should be utilized within 3 feet in accordance with ITD Standard Specifications Section 210.03 and the fill lift thickness should be reduced to a maximum loose lift thickness of 6 inches.

If necessary, the lateral earth pressures could be reduced behind the abutment wall by constructing a wrapped-face geosynthetic wall and providing at least 6 inches of separation between the backfill and abutment. This option would only be feasible if a structural approach slab is provided to span the abutment/backfill transition.

230.05.04 ANCHORS

Permanent ground anchors are not anticipated for the proposed bridge.

230.05.05 DRAINAGE

Ground Surface

Drainage measures should be implemented to collect and divert surface runoff from the roadway to reduce potential saturation of the backfill material and sheet flow of surface water over retaining wall faces.

230.05.06 EMBANKMENTS

New embankments or widening of existing embankments should be sloped at 2H:1V (horizontal to vertical) or flatter. It might be possible to use steeper slopes by placing higher strength embankment material; using geogrid reinforcement; and/or providing slope armoring. GeoEngineers should be consulted if slopes steeper than 2H:1V are planned. The soil supporting the embankments should be stripped of all organic and loose material which is estimated to be less than 6 inches thick.

Per ITD Standard Specifications, Section 205.03, the upper 8 inches of subgrade should be scarified, moisture-conditioned and compacted to meet Class A standards. If exposed soil cannot be compacted to meet Class A standards, the material should be overexcavated and replaced with properly compacted Granular Subbase. Fill placed against existing embankments with slopes steeper than 3H:1V should be keyed into the existing embankments in accordance with Section 205.03.F of the ITD Standard Specifications. Settlement and stability of the proposed approach embankments will be evaluated in the roadway materials report for this project which will be prepared under separate cover.

Abutment and Wing Walls

We recommend backfill placed against the abutment and wing walls consist of Granular Subbase underneath the approach slab to facilitate drainage.

230.05.07 EROSION CONTROL

Surface runoff from the roadway, including the roadway across the proposed bridge structure, should be collected and discharged off the abutment embankments and walls to reduce potential embankment erosion. Discharge areas should be protected with riprap or spalls underlain by a riprap/erosion control geotextile. Appropriate temporary and permanent erosion control should be provided for embankment slopes. Slopes should be protected with rock mulch, rock armoring, seeding/revegetating, or slope concrete paving. Rock armoring, if used, should consist of rock cap or hand-placed riprap.

230.05.08 SEISMIC DESIGN

Ground Motion Parameters

In accordance with Table 3.10.6-1 of the AASHTO LRFD Bridge Design Specifications, the site is considered a Seismic Performance Zone 2. Section 630.04 of the ITD Materials Manual presents maps providing design acceleration coefficients, which have been adapted from the Seismic Hazard Maps presented in the AASHTO LRFD Bridge Design Specifications. The acceleration coefficients for design are based on the expected ground motion at the project site that has a 7 percent probability of exceedance in a 75-year period (approximate 1,000-year return period). We recommend the seismic design parameters presented in Table 10 below.

TABLE 10. 2020 AASHTO SEISMIC PARAMETERS

AASHTO SEISMIC PARAMETER	ST. MARIES RIVER BRIDGE
Site Class	D
Seismic Performance Zone	2
Peak Ground Acceleration (PGA), bedrock Site Class B	0.091
Spectral Response Acceleration at 0.2-Second Period (S_S), bedrock Site Class B	0.206
Spectral Response Acceleration at 1-Second Period (S_1), bedrock Site Class B	0.066
Zero-period Site Factor, F_{pga}	1.6
Short Period Site Factor, F_a	1.6
Long-period Site Factor, F_v	2.4
Free-field Acceleration Coefficient (A_s), Amplified for Site Class D	0.146
Design Spectral Acceleration Coefficient at 0.2-Second Period (S_{DS}), Amplified for Site Class D	0.329
Design Spectral Acceleration Coefficient at 1.0-second Period (S_{D1}), Amplified for Site Class D	0.159

Notes:

Mapped values derived from the 2009 United States Geological Survey (USGS) online tool using coordinates of 47.051034°, -116.287691°

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as pore water pressures increase in response to strong ground shaking. The increased pore water pressure may temporarily meet or exceed soil overburden pressures to produce conditions that allow soil and water to flow, deform, or erupt from the ground surface. Ground settlement, lateral spreading and/or sand boils may

result from soil liquefaction. Structures supported on or within liquefied soils may suffer foundation settlement or lateral movement that can be damaging to the structure.

The evaluation of liquefaction potential is a complex procedure and is dependent on numerous site parameters, including soil grain size, soil density, site geometry, static stresses, and the design ground acceleration. Liquefaction potential of the site was evaluated by comparing the earthquake-induced cyclic stress ratio (CSR) to the soils cyclic resistance ratio (CRR) using methods by Boulanger and Idriss 2014.

The CSR was calculated using a free-field Acceleration Coefficient (A_S), determined from the United States Geological Survey (USGS) deaggregation for a 975-year return period event. An earthquake magnitude of 6.16 was used for our analysis which represents the magnitude of the highest contributing fault based on the USGS deaggregation plots. The CRR was calculated based on SPT blow counts encountered in the borings. The factor of safety against liquefaction is determined by dividing the CRR by the CSR.

The analysis indicates that the site soils have low risk of seismic liquefaction, and liquefaction settlement estimates were negligible for each boring evaluated. Since the site soils have a low risk of liquefaction, it is our opinion that lateral spreading risk is also low. In our opinion, seismic liquefaction and lateral spreading soil strength loss and soil movements do not need to be considered in design.

Fault Rupture

We reviewed the United States Geological Survey online Quaternary Faults database (USGS, 2014), Idaho Geological Survey's (IGS) Miocene--Quaternary Fault Map database and published maps available from IGS relative to seismic history and faulting in the project vicinity. No active faults were mapped by these sources within 30 miles of the project site.

Based on our observations and the site location with respect to the nearest known faults, it is our opinion that the probability of damaging fault rupture at the site is low and does not warrant fault trenching or specific design considerations.

230.05.09 CONSTRUCTION

Earthwork

Portions of the existing site soil contain appreciable amounts of fines (silt and clay) and are moisture sensitive. If site grading is performed during or after extended periods of rainfall or snow melt, the moisture content of the exposed soil could be significantly above optimum. This condition, if encountered, could delay grading by reducing equipment mobility and creating difficulties with respect to control of the soil moisture content during compaction. Typical remedial measures include disking and aerating the soil during dry weather; mixing the soil with dryer materials; removing and replacing the soil with an approved fill material; or mixing the soils with an approved hydrating agent such as a lime or cement product. GeoEngineers or experienced ITD personnel should be consulted before implementing remedial measures to observe the unstable subgrade condition and provide site-specific recommendations.

Driven Pile Installation

We recommend pile tip protection (point with teeth or rock point) be used from the ITD Qualified Products List (QPL) steel H-piles.

Decomposed bedrock and intact bedrock near the highest pile tip elevation was encountered in the borings; therefore, we recommend the contract require pre-drilling to the highest pile tip elevation (shown in Table 6) to reduce the risk of refusal on the decomposed or intact bedrock prior to reaching the minimum required pile embedment. We recommend the contract require the contractor to develop a work plan to keep a stable hole during pre-drilling. If temporary casing is used, select appropriate casing material and remove the temporary casing in accordance with the approved work plan, as specified in Section 520 of the ITD Standard Specifications for Highway Construction. After pre-drilling, fill the soil portions of the drilled holes with coarse aggregate for concrete, as specified in Section 520 of the ITD Standard Specifications for Highway Construction.

The estimated pile length for the abutment piles is 36 feet. Therefore, we do not anticipate pile splices will be needed. We recommend contract documents include a bid item for splices should they be required in the event they are necessary during construction. Mechanical splicers, if used, should be selected from the ITD Qualified Products List (QPL) and will need to be welded per ITD practice.

230.06 Appendices

- Appendix A—Vicinity Map, Site Plan and Subsurface Profile
- Appendix B—Exploration Logs
- Appendix C—Laboratory Test Results
- Appendix D—Photographs
- Appendix E—Axial and Lateral Pile Capacity Plots
- Appendix F—Global Stability
- Appendix G—Foundation Investigation Plats

230.07 Foundation Investigation Plat

A preliminary Foundation Investigation Plat is presented in is presented in Appendix G. Contact GeoEngineers to update the sheets for Preliminary, Final and PS&E submittals.

230.08 Current Specifications and Minimum Testing Requirements

Our geotechnical exploration program, analysis and recommendations are based on the following design standards:

- American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Ninth Edition, 2020.
- Idaho Transportation Department (ITD) Materials Manual, October 2020.
- ITD Bridge Design LRFD Manual, June 2022.
- ITD Standard Specifications for Highway Construction, 2023
- ITD Quality Assurance Manual, November 2019

If these change before the project is advertised, a review is required to ensure necessary changes to the report are made.

230.09 Special Provision Items

No geotechnical special provisions are anticipated at this time.

230.10 References

American Association of State Highway and Transportation Officials . 2020. Load and Resistance Factor Design Bridge Design Specifications. 9th Edition.

Lewis, R.S., Link, P.K., Standford, L.R., and Long, S.P. 2012. Geologic Map of Idaho. Idaho Geological Survey. Moscow-Boise-Pocatello. Scale 1:750,000.

Lewis, R.S., Burmester, R.F., Kauffman, J.D., and Frost, T.P. 2000. Geologic Map of the St. Maries 30 x 60 Minute Quadrangle, Idaho. Idaho Geological Survey. Scale 1:100,000.

Boulanger, R.W. and Idriss, I.M., 2014. CPT and SPT Based Liquefaction Triggering Procedures, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, Report No. UCD/CGM-14/01, April 2014.

Ensoft, APILE 2014 Version 2014.6.5

Ensoft. LPILE 2019 Version 11.02.

Geo-Slope International. GeoStudio 2016, Version 8.16.1.

Holtz, R.D. and W.D. Kovacs. 1981. An Introduction to Geotechnical Engineering.

Idaho Geologic Survey. 2004. Idaho Miocene – Quaternary Fault Database. Accessed October 10, 2024. Idaho Geological Survey WebMap (idahogeology.org).

Idaho Transportation Department. 2018. Standard Specifications for Highway Construction. Including 2019, 2020, and 2021 Supplementals.

Idaho Transportation Department. 2020. Materials Manual. October 2020.

Idaho Transportation Department, 2021. ITD Load Resistance Factor Design (LRFD) Bridge Manual.

U.S. Department of Transportation, Federal Highways Administration. 2006. “Design and Construction of Driven Pile Foundations.”

United States Geological Survey, Quaternary Faults Database. Accessed October 7, 2024. U.S. Quaternary Faults (arcgis.com)

United States Geological Survey U.S. Seismic Design Maps Online Tool: https://earthquake.usgs.gov/ws/designmaps/. Accessed October 7, 2024.

230.11 Report Considerations

We have prepared this report for ITD to design the SH-3 St. Maries River Bridge project in Shoshone County, Idaho. This report is not intended for use by others and the information contained herein is not applicable to other sites. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

If project conditions change from those assumed in this report, we recommend that we be retained to review our design, conclusions and recommendations and to determine whether they are still appropriate, or to provide supplemental information, as appropriate. When the design has been finalized, we recommend that the project design and specifications be reviewed by our firm to verify that our recommendations have been interpreted and implemented as intended.

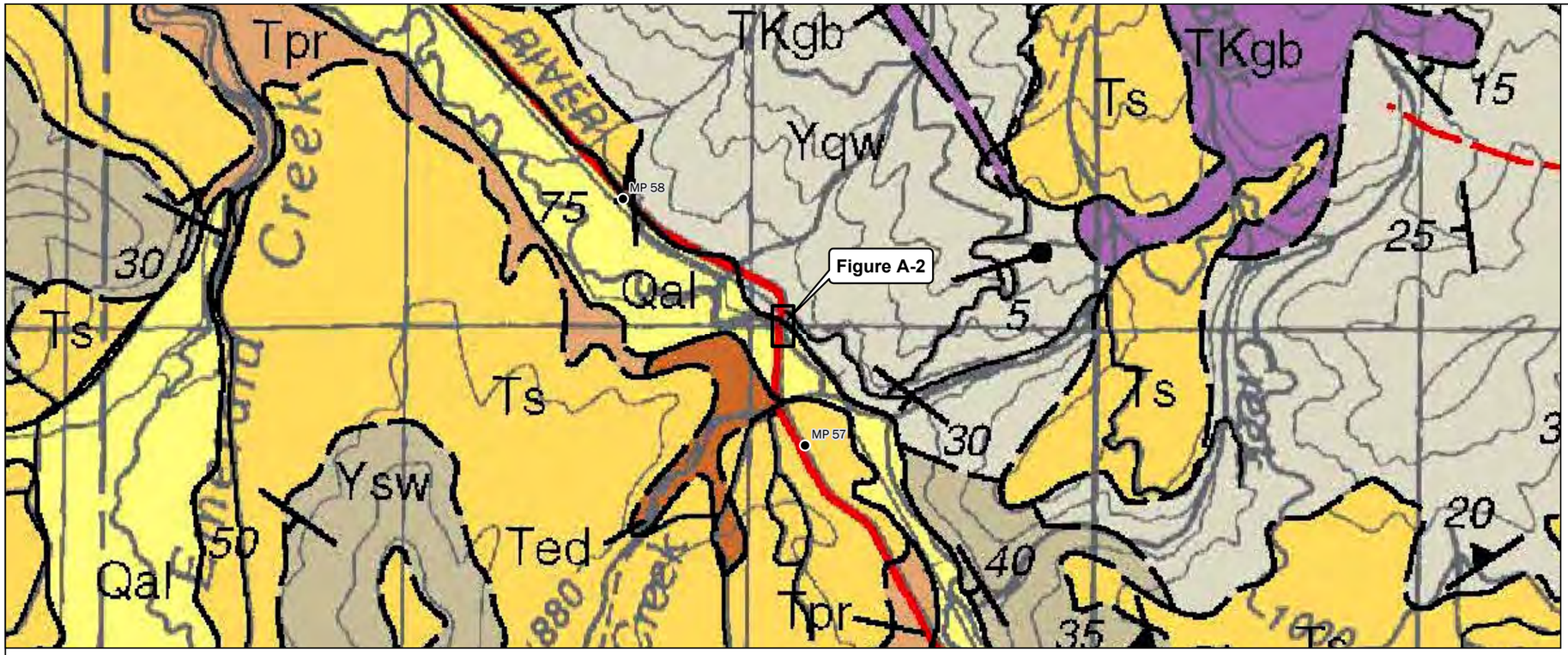
The scope of our services does not include services related to construction safety precautions and our recommendations are not intended to direct the Contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

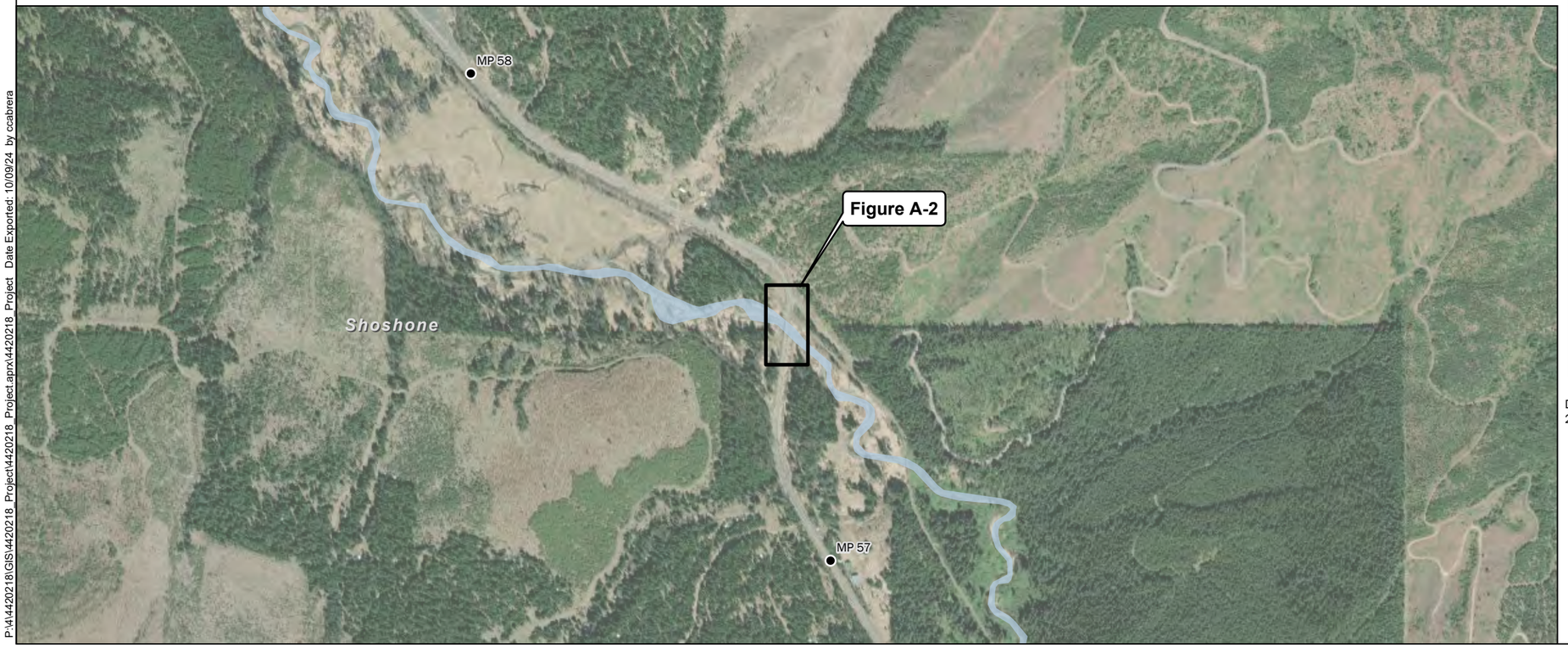
Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Appendices

Appendix A

Vicinity Map, Site Plan and Subsurface Profile





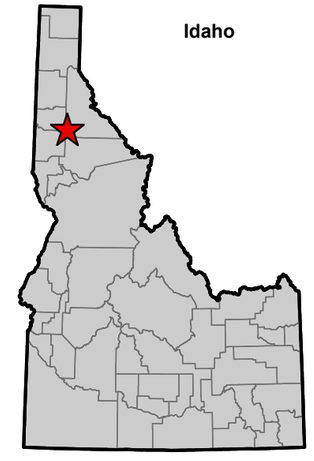
Legend

Project Area

Geologic Unit

Qal: Alluvial deposits (Holocene)
Ts: Sediment (Oligocene? and Miocene)
Tpr: Wanapum Formation - Priest Rapids Member (Miocene)
Ted: Eckler Mountain Member - Basalt of Dodge (Miocene)
TKgb: Gabbro (Tertiary or Cretaceous)
Ysw: Schist and phyllite of the Wallace Formation (Middle Proterozoic)
Yqw: Quartzite of the Wallace Formation (Middle Proterozoic)

N
W E
S



Data Source: Geologic map of the St. Maries 30 x 60 minute quadrangle, Idaho, GM-29, 2000, Reed S. Lewis, Russell F. Burmester, John D. Kauffman, and Thomas P. Frost.

Vicinity and Geology Map

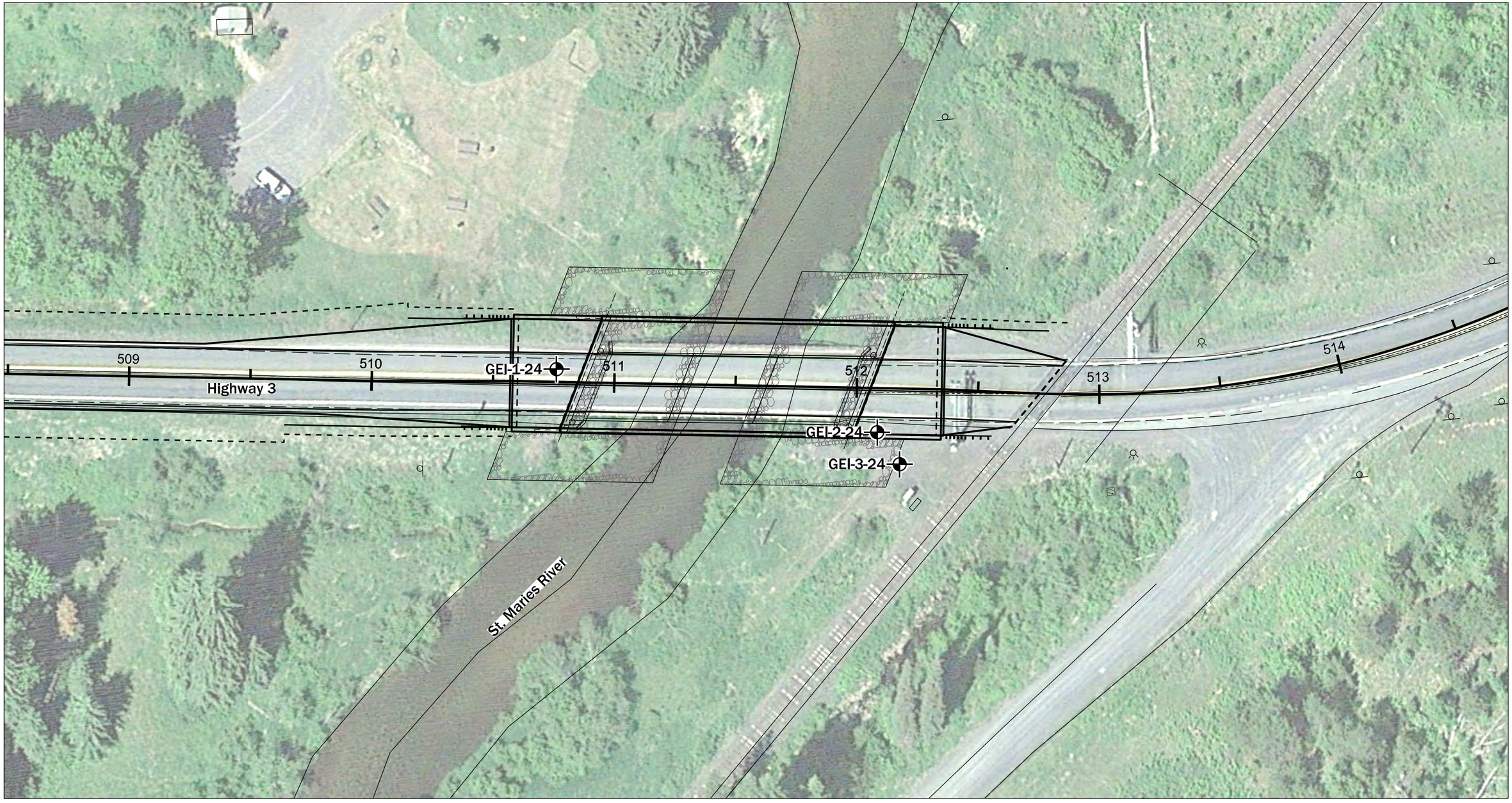
SH-3 St Maries River Bridge
Project No. A023(257); Key No. 23257
Shoshone County, Idaho

GeoEngineers

Figure A-1

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Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Aerial from Google Earth Pro dated 7/18/2018
Background data downloaded from Project Wise on 11/30/2020

Projection: Project

Legend

GEI-1-24 Boring by GeoEngineers, Inc., 2024

N

40 0 40
Feet

Site Plan

SH-3, St. Maries River Bridge
Project No. A023(257); Key No. 23257
Shoshone County, Idaho

GeoEngineers

Figure A-2

P:\A\4420218\CAD\00\Geotech\442021800_F02-F03_Site Plan and Profile.dgn

South

North

ELEVATION (FEET)

Abut. 1

Abut. 2

GEI-1-24
5' L

GEI-2-24
18' R

GEI-3-24
31' R

Finished Grade

Existing Fill

Alluvium

Weathered Bedrock

Intact Bedrock (Quartzite)

Wing Wall (Typ.)

Q_{50}

Existing Grade

Contraction Scour Depth

Steel Piles (Typ.)

GP-GM

Quartzite

2800
2790
2780
2770
2760
2750
2740
2730
2720
2710
2700
2690
2680
2670
2660
2650

510+00
510+20
510+40
510+60
510+80
511+00
511+20
511+40
511+60
511+80
512+00
512+20
512+40
512+60
512+80

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Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Legend

- Inferred Soil Contact
- SM — Soil Classification
- Groundwater Measured in Piezometer
- Groundwater Measured at Time of Drilling
- 20 — Blow Count

20 0 20
Feet

Subsurface Profile

SH-3, St. Maries River Bridge
Project No. A023(257); Key No. 23257
Shoshone County, Idaho

GeoEngineers

Figure A-3

Appendix B
Exploration Logs

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% RETAINED ON NO. 200 SIEVE	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

Modified California Sampler (6-inch sleeve) or Dames & Moore

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
UU	Unconsolidated undrained triaxial compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

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Figure B-1

Rev. 03/2024

Scale of Relative Rock Weathering[1]

Designation	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 inch into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

Scale of Relative Rock Hardness[1]

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocketknife. Scratched with fingernail.	100-1000 psi
Soft	R2	Can be peeled by a pocketknife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1000-4000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4000-8000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8000-16000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16000 psi

Discontinuity Spacing[1]

Description for Bedding, Foliation, or Flow Banding	Spacing	Description of Joints, Faults, or Other Fractures
Very Thick	>10 ft	Very Widely Spaced
Thick	3 ft – 10 ft	Widely Spaced
Medium	1 ft – 3 ft	Moderately Spaced
Thin	2 in – 1 ft	Closely Spaced
Very Thin	<2 in	Very Closely Spaced

Rock Quality Designation (RQD)[1, 2]

RQD (Percent)	Description of Rock Quality
0-25	Very Poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

NOTES: 1. Based on ASCE Manual on Engineering Practice No. 56,1976. 2. RQD is a modified core recovery measurement which expresses the number of hard and sound rock pieces of 4" or more in size as a percentage of the total length of core run.

Explanation of Bedrock Terms

GeoEngineers

Figure B-2

Drilled: Start 7/8/2024 End 7/9/2024	Total Depth (ft) 60.5	Logged By OLK Checked By ASC	Driller GeoEngineers, Inc.	Drilling Method Hollow-stem Auger	
Surface Elevation (ft) 2774.77 Vertical Datum NAVD88		Hammer Data Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment Truck mounted CME-75	
Latitude 47.051034 Longitude -116.287691		System Datum Decimal Degrees WGS84 (feet)		See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered Soil (in)/Rock (%)	Blow Counts (N-value)	Sample/Run Testing	RQD %	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0						AC	Approximately 3½ inches of asphalt concrete pavement			
							SM	Approximately 6 inches of silty fine to coarse sand with occasional gravel (base)			
							SC	Dark gray clayey fine to coarse sand with gravel (medium dense, moist) (fill)			
		10	4-5-5 (10)	1 SA					13	47	
2770	5	13	5-6-5 (11)	2							Drilling became soft at approximately 6½ feet
		12	2-2-4 (6)	3 %F			CL	Dark gray lean clay with sand and occasional gravel (medium stiff, moist) (fill)	18	55	
2765	10	16	1-2-2 (4)	4							
		14	1-2-13 (15)	5 %F			SM	Gray silty fine to coarse sand (medium dense, moist to wet) (alluvium)	21	18	
2760	15	16	10-6-8 (14)	6 SA			SP-SM	Brown fine to coarse sand with silt and gravel (medium dense, wet) (alluvium)	12	11	Groundwater encountered at approximately 15 feet
2755	20	8	14-26-28 (54)	7 %F			SM	Decomposed Quartzite; reworks to light brown silty fine sand with trace gravel (dense to very dense, moist) (weathered bedrock)	10	15	
2750	25	18	16-16-22 (38)	8 %F					17	38	
2745	30										

Note: See Figure B-1 for explanation of symbols; Figure B-2 for explanation of bedrock terms.
Coordinates Data Source: Horizontal approximated based on survey from HMH Engineering. Vertical approximated based on survey from HMH Engineering.

Date:1/10/25 Path:P:\4\4420218\GINT\442021800.GPJ DBLibrary:LIbrary:GEOENGINEERS_DF_STD_ITD.GLB/GEI8_GEOTECH_SOIL_ROCK_%F_NO_GW

Log of Boring GEI-1-24

GeoEngineers

Project: KN: 23257 - St. Maries River Bridge
Project Location: Shoshone County, Idaho
Project Number: 4420-218-00

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered Soil (in)/Rock (%)	FIELD DATA: Blow Counts (N-value)	FIELD DATA: Sample/Run Testing	FIELD DATA: RQD %	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	30	18	18-24-30 (54)	9							
2740	35	18	8-15-33 (48)	10							
2735	40	8	30-50/4" (50/4")	11							Driller noted more competent rock at approximately 42 feet
2730	45	0 71	50/0"	12 R-1	16		QTZT	Quartzite; light gray, medium grained, moderately weathered, medium hard (R3), sand in fractures, very poor RQD			Driller noted hard rock at approximately 44½ feet; switched to rock coring
		100		R-2	100		QTZT	Quartzite; light gray, fine grained, slightly weathered, hard (R4), poor to excellent RQD			
2725	50	92		R-3 UC	50						UCS = 13,540 psi
2720	55	80		R-4 UC	20			Becomes medium grained, moderately weathered			UCS = 3,626 psi
2715	60	100		R-5	50						

Date:1/10/25 Path:P:\4\4420218\GINT\442021800.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_ITD.GLB/GEI8_GEOTECH_SOIL_ROCK_%F_NO_GW

Log of Boring GEI-1-24 (continued)

GeoEngineers

Project: KN: 23257 - St. Maries River Bridge
Project Location: Shoshone County, Idaho
Project Number: 4420-218-00

Figure B-3
Sheet 2 of 2

Drilled	Start 7/9/2024	End 7/10/2024	Total Depth (ft)	64.25	Logged By	OLK	Driller	GeoEngineers, Inc.	Drilling Method	Hollow-stem Auger
					Checked By	ASC				

Surface Elevation (ft) / Vertical Datum	2774.25 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck mounted CME-75
Latitude / Longitude	47.051296 -116.287638	System Datum	Decimal Degrees WGS84 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	Interval Recovered Soil (in)/Rock (%)	Blow Counts (N-value)	Sample/Run Testing	RQD %	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0						AC	Approximately 3½ inches of asphalt concrete pavement			
							SM	Approximately 6 inches of silty fine to coarse sand with occasional gravel (base)			
							CL	Dark gray clay with sand and occasional gravel (soft to medium stiff, moist) (fill)			
2770		16	4-2-3 (5)	1 pH; Resist; CA							pH = 6.4; Resistivity = 105 ohms-cm; Soluble Sulfate = 29 ppm; Soluble Chloride = 848 ppm
	5	11	1-2-2 (4)	2 AL							AL (LL = 37; PI = 13)
2765		16	1-2-2 (4)	3 SA				Decreased sand and gravel content	24	65	
	10	13	1-1-2 (3)	4							
2760		14	2-3-9 (12)	5 %F			SP-SM	Dark gray fine to coarse sand with silt and occasional gravel (medium dense, moist to wet) (alluvium)	16	11	
	15		2-2-4 (6)	6 SA			SM	Dark gray silty fine to coarse sand with gravel (loose, wet) (alluvium)	19	15	Groundwater encountered at approximately 14 feet
2755							SM	Decomposed Quartzite; reworks to light brown silty fine sand with occasional gravel (very dense, moist) (weathered bedrock)			
	20	10	36-36-48 (84)	7 %F					9	28	
2750											
	25	9	26-50/3" (50/3")	8							
2745											
	30										

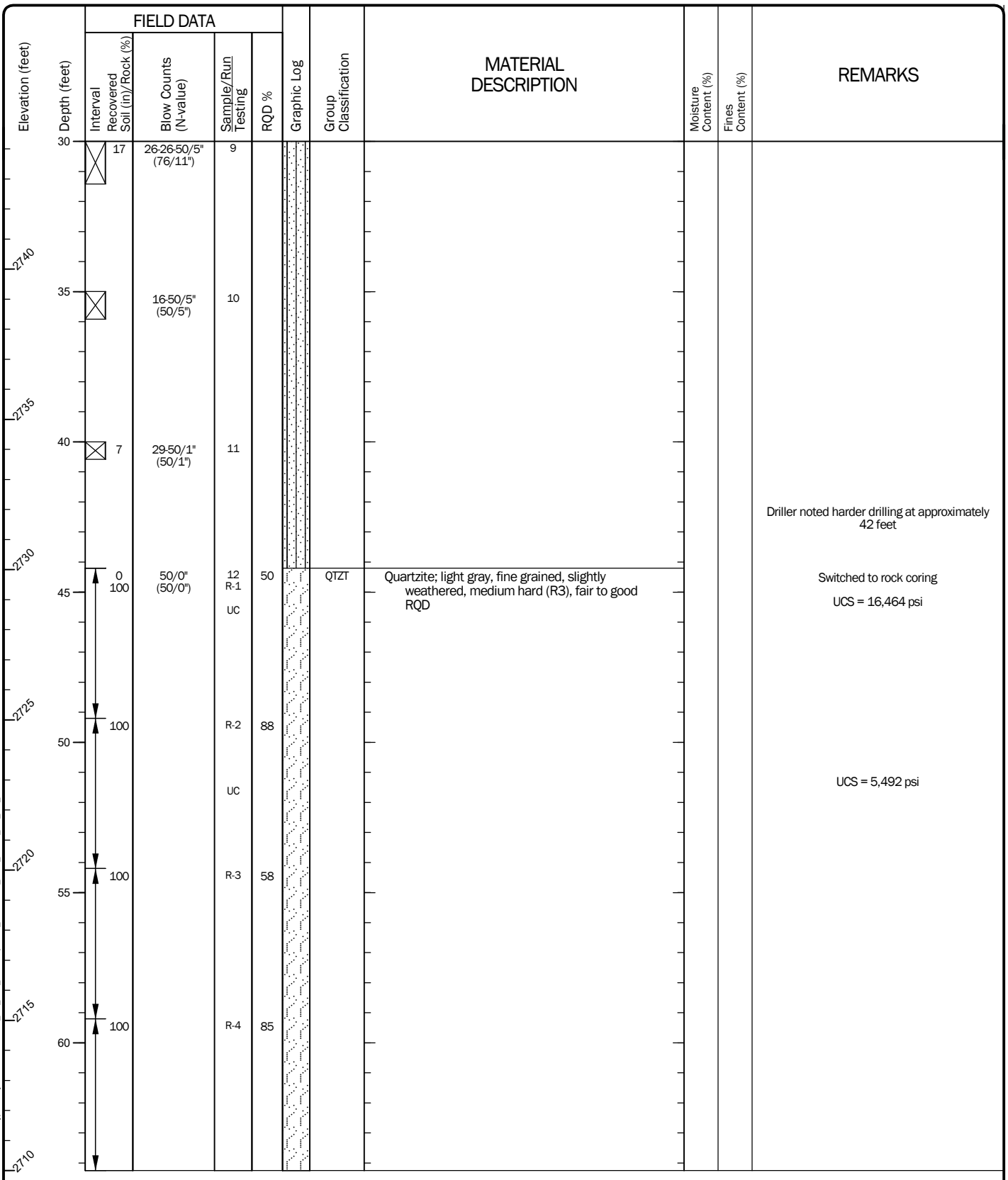
Note: See Figure B-1 for explanation of symbols; Figure B-2 for explanation of bedrock terms.
Coordinates Data Source: Horizontal approximated based on survey from HMH Engineering. Vertical approximated based on survey from HMH Engineering.

Date:1/10/25 Path:P:\4\4420218\GINT\4420218.00.GPJ DBLibrary:LLibrary:GEOENGINEERS_DF_STD_ITD.GLB/GEI8_GEOTECH_SOIL_ROCK_%F_NO_GW

Log of Boring GEI-2-24

GeoEngineers

Project: KN: 23257 - St. Maries River Bridge
Project Location: Shoshone County, Idaho
Project Number: 4420-218-00



Log of Boring GEI-2-24 (continued)

GeoEngineers

Project: KN: 23257 - St. Maries River Bridge
Project Location: Shoshone County, Idaho
Project Number: 4420-218-00

Figure B-4
Sheet 2 of 2

Date:1/10/25 Path:P:\4\4420218\GINT\442021800.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_ITD.GLB/GEI8_GEOTECH_SOIL_ROCK_%F_NO_GW

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	7/11/2024	7/11/2024	18.5	OLK	ASC	GeoEngineers, Inc.	Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck mounted CME-75
Surface Elevation (ft) Vertical Datum	2772.22 NAVD88	Top of Casing Elevation (ft)	2771.81
Latitude Longitude	47.051344 -116.287597	Horizontal Datum	Decimal Degrees WGS84 (feet)

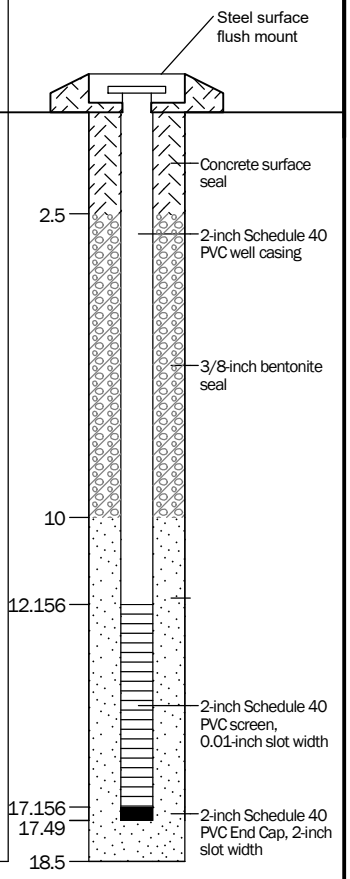
A 2-in well was installed on 7/11/2024 to a depth of 18.5 ft.

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
12/9/2025	10.22	2762.00

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
2770	0								CL	Dark gray sandy lean clay with occasional gravel (soft to medium stiff, moist) (fill)		
			14	7		1 %F					19	61
	5		10	3		2 SA					18	50
2765			18	6		3 SA				With organic matter and decreased gravel content	29	66
	10		16	6		4	▼					
2760			18	3		5						
	15		8	11		6 SA			GP-GM	Gray fine to coarse gravel with silt and sand (medium dense, wet) (alluvium)	11	8
2755			11	35		7				Becomes brown and very dense		

WELL LOG



Note: See Figure B-1 for explanation of symbols; Figure B-2 for explanation of bedrock terms.
Coordinates Data Source: Horizontal approximated based on survey from HMH Engineering. Vertical approximated based on survey from HMH Engineering.

Log of Monitoring Well GEI-3-24

GeoEngineers

Project: KN: 23257 - St. Maries River Bridge
Project Location: Shoshone County, Idaho
Project Number: 4420-218-00

Figure B-5
Sheet 1 of 1

Date:12/16/25 Path:P:\A\4420218\GINT\442021800.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

Appendix C
Laboratory Test Results

Table C-1

Summary of Laboratory Test Results

SH-3, St. Maries River Bridge

ITD Project No. A023(257); Key No. 23257

Shoshone County, Idaho

Sample Information				Water Content (%)	Soil Index Testing								Consolidation	UCS (psi)	Resilient Modulus	Idaho R-Value	Soil Corrosivity				Soil (USCS) /Rock Description
					Atterberg Limits (%)			Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Sieve Analysis											
Boring Designation	Sample Number	Sample Depth (feet)	Sample Type		Liquid Limit	Plastic Limit	Plastic Index			Percent Gravel	Percent Sand	Percent Fines					Soil pH	Soil Resistivity (ohms-cm)	Sulfates	Chlorides	
GEI-1-24	1	2.5	SPT	13						22..5	31	47									Clayey fine to coarse sand with gravel, SM
GEI-1-24	3	7.5	SPT	18								55									Lean clay with sand, CL
GEI-1-24	5	12.5	SPT	21								18									Silty fine to coarse sand, SM
GEI-1-24	6	15	SPT	12						34	55	11									Silty fine to coarse sand with silt and gravel, SP-SM
GEI-1-24	7	20	SPT	10								15							50-?		Decomposed Quartzite, silty fine sand, SM
GEI-1-24	8	25	SPT	17								38									Decomposed Quartzite, silty fine sand, SM
GEI-1-24	Run 3	49	HQ core	0				165	164					13540							Quartzite
GEI-1-24	Run 4	55	HQ core	0				163	162					3626							Quartzite
GEI-2-24	1	2.5	SPT														6.4	105	29.0	848	Clay with sand and occasional gravel, CL
GEI-2-24	2	5	SPT	22	37	24	13														Clay with sand and occasional gravel, CL
GEI-2-24	3	7.5	SPT	24								65									Clay with sand and occasional gravel, CL
GEI-2-24	5	12.5	SPT	16								11									Fine to coarse sand with occasional gravel, SP-SM
GEI-2-24	6	15	SPT	19						33	53	15									Silty fine to medium sand with gravel, SM
GEI-2-24	7	20	SPT	9								28									Silty fine sand, SM; Decompose Quartzite
GEI-2-24	Run 1	46.5	HQ core	0				166	166					16464							Quartzite
GEI-2-24	Run 2	49	HQ core	0				167	166					5492							Quartzite
GEI-3-24	1	2.5	SPT	19								61									Sandy lean clay, CL
GEI-3-24	2	5	SPT	18						13	37	50									Sandy lean clay with occasional gravel, CL
GEI-3-24	3	7.5	SPT	29								66									Sandy lean clay with occasional gravel, CL
GEI-3-24	4	10	SPT																		Sandy lean clay with occasional gravel, CL
GEI-3-24	6	15	SPT	11						47	45	8									Fine to coarse gravel with silt and sand, GP-GM

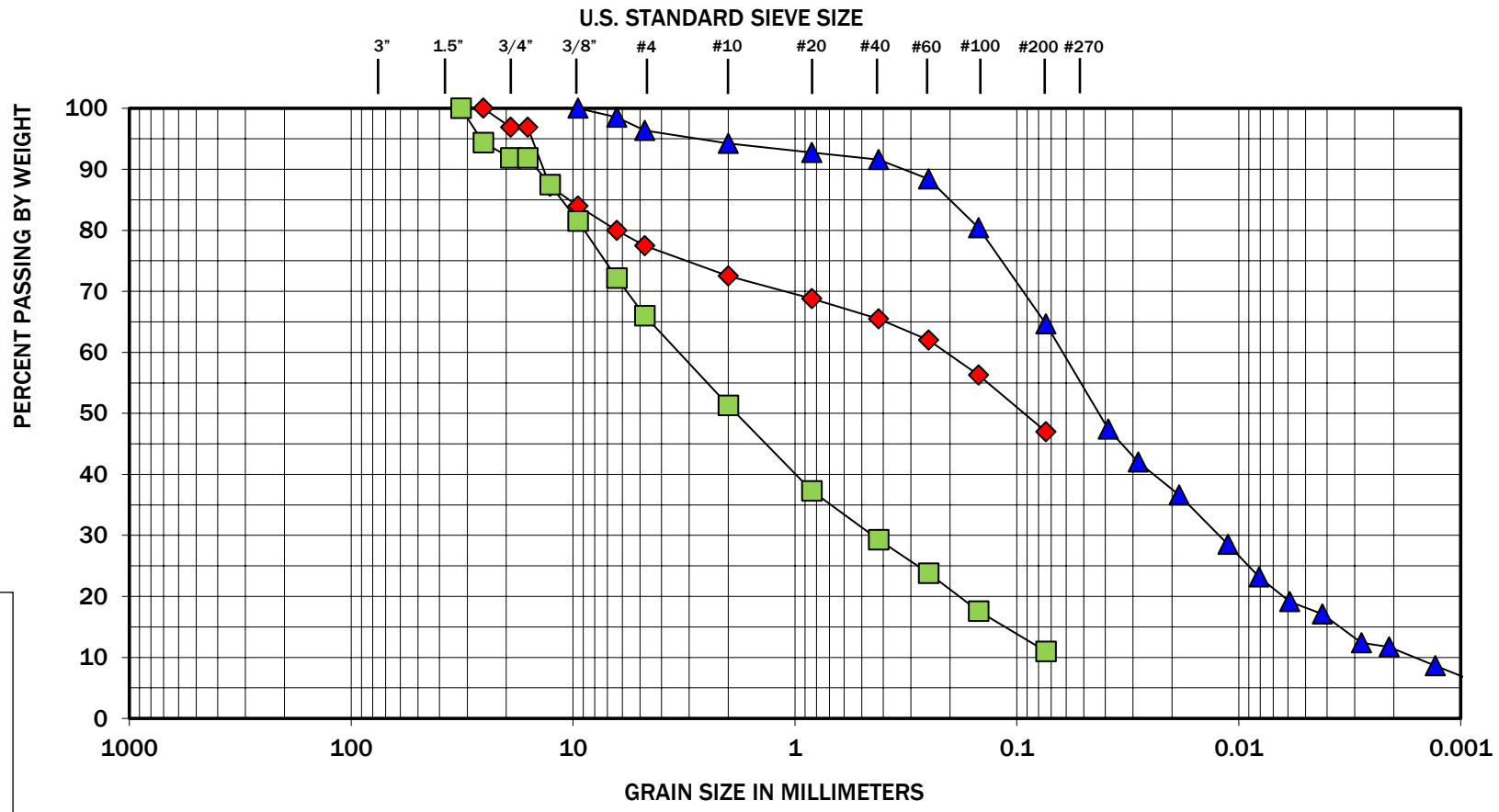
Notes:

SPT = standard penetration test

UCS = unconfined compressive strength

USCS = Unified Soil Classification System

ohms-cm = ohms-centimeter; pcf = pounds per cubic foot; psi = pounds per square inch



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	GEI-1-24	2.5 – 4.0	13	Clayey fine to coarse sand with gravel (SM)
■	GEI-1-24	15 – 16.5	12	Silty fine to coarse sand with silt and gravel (SP-SM)
▲	GEI-2-24	7.5 - 9	24	Lean clay with sand (CL)

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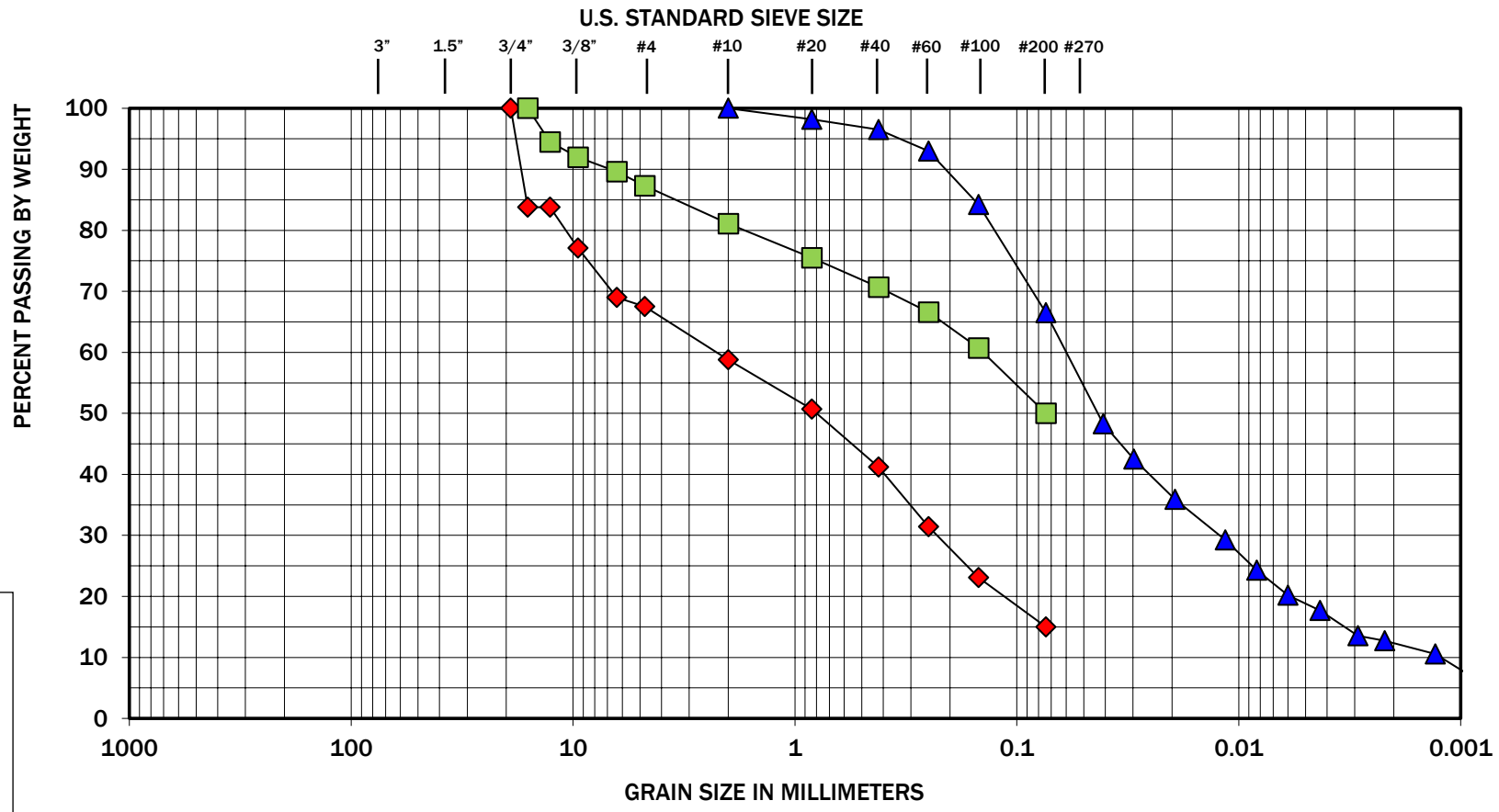
The grain size analysis results were obtained in general accordance with ASTM D 6913.

GeoEngineers

Sieve Analysis Results

SH-3 St. Maries River Bridge
ITD Project No. A023(257); Key No. 23257
Shoshone County, Idaho

Figure C-1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	GEI-2-24	15 – 16.5	19	Silty fine to medium sand with gravel (SM)
■	GEI-3-24	5 – 6.5	18	Sandy lean clay with occasional gravel (CL)
▲	GEI-3-24	7.5 – 9.0	29.33	Sandy lean clay with occasional gravel (CL)

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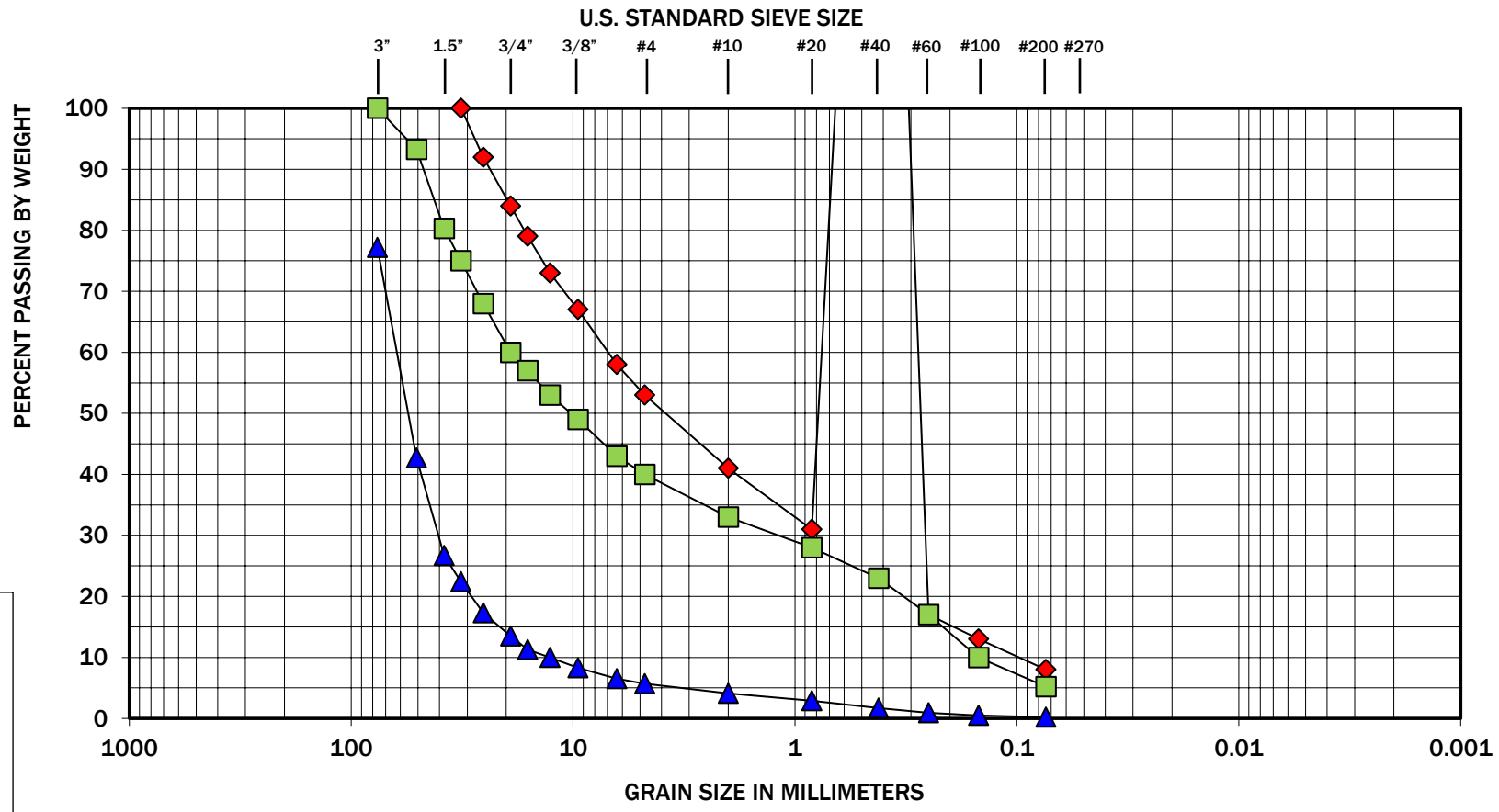
The grain size analysis results were obtained in general accordance with ASTM D 6913.

Sieve Analysis Results

SH-3 St. Maries River Bridge
ITD Project No. A023(257); Key No. 23257
Shoshone County, Idaho

GeoEngineers

Figure C-2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	GEI-3-24	15 – 16.5	11	Fine to coarse gravel with silt and sand (GP-GM)
■	GEI-Bulk 1		5.15	Poorly graded gravel with silt and sand (GP-GM)
▲	GEI-Bulk 2		3.84	Poorly graded gravel (GP)

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The grain size analysis results were obtained in general accordance with ASTM D 6913.

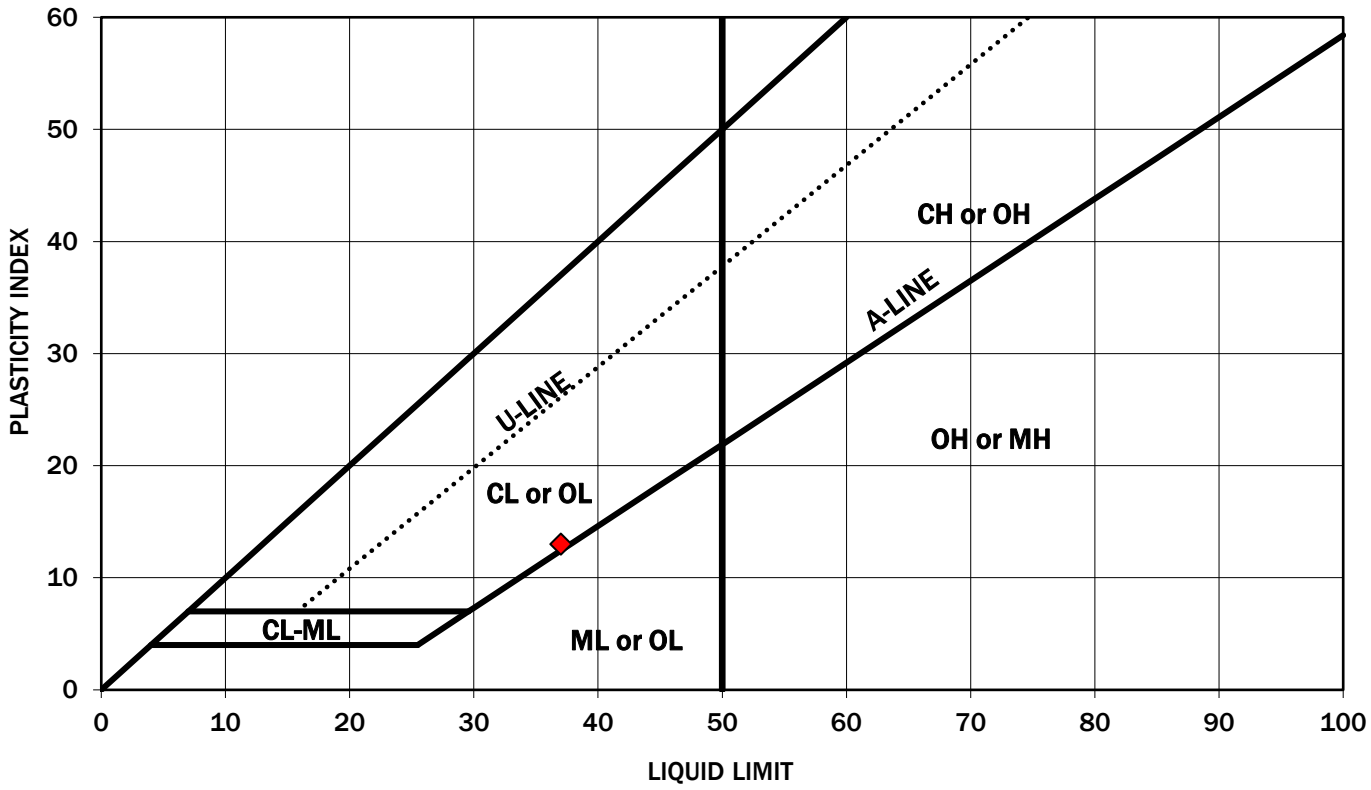
Sieve Analysis Results

SH-3 St. Maries River Bridge
ITD Project No. A023(257); Key No. 23257
Shoshone County, Idaho

GeoEngineers

Figure C-3

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	GEI-2-24	5-6.5	22	37	13	Dark brown-black gr sandy silt, moist (ML) with trace organic matter

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AASHTO ACCREDITED

Atterberg Limits Test Results

SH-3 St. Maries River Bridge
ITD Project No. A023(257); Key No. 23257
Shoshone County, Idaho

GeoEngineers

Figure C-4

ANATEK LABS

Analytical Results Report For:

GeoEngineers, Inc.

Project Number:

SH-3 St. Marries River 04420-218-00

Anatek Work Order:

WEH0494

Anatek Moscow - 1282 Alturas Drive - Moscow, ID 83843 - 208-883-2839 - moscow@anateklabs.com - FL NELAP E87893
Anatek Spokane - 504 E Sprague Ste. D - Spokane, WA 99202 - 509-838-3999 - spokane@anateklabs.com - FL NELAP E871099
Anatek Yakima - 4802 Tieton Drive - Yakima, WA 98908 - 509-225-9404 - yakima@anateklabs.com - FL NELAP E871190
Anatek Wenatchee - 3019 Gs Center Rd - Wenatchee, WA 98801 - 509-701-8362

Anatek Labs, Inc.

1282 Alturas Drive - Moscow, ID 83843 - (208) 883-2839 - email moscow@anateklabs.com
504 E Sprague Ste. D - Spokane, WA 99202 - (509) 838-3999 - email spokane@anateklabs.com

Client:	GeoEngineers, Inc.	Work Order:	WEH0494
Address:	523 E. 2nd Ave.	Project:	SH-3 St. Marries River 04420-218-00
	Spokane, WA 99202	Reported:	8/19/2024 17:21
Attn:	Alysa Churchfield		

Analytical Results Report

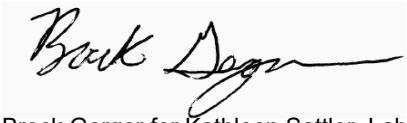
Sample Location:	GES-2-24 S-1 @ 2.5'		
Lab/Sample Number:	WEH0494-01	Collect Date:	07/09/24 00:00
Date Received:	08/09/24 15:30	Collected By:	GEI
Matrix:	Solid		

Analyte	Result	Units	PQL	Analyzed	Analyst	Method	Qualifier
Inorganics							
% Solids	84.1	%	0.100	8/12/24 11:18	ALH	SM 2540 G	*
Chloride	848	mg/kg dry	18.0	8/14/24 12:58	BDM	EPA 300.0	
pH	6.35 @ 25°C	pH Units in Water	0.986	8/12/24 11:11	ALH	EPA 9045D	
Resistivity	105	ohms-cm	1.00	8/12/24 13:20	ALH	ASTM G 57a	
Sulfate	28.6	mg/kg dry	1.80	8/14/24 12:16	BDM	EPA 300.0	

Anatek Labs, Inc.

1282 Alturas Drive - Moscow, ID 83843 - (208) 883-2839 - email moscow@anateklabs.com
504 E Sprague Ste. D - Spokane, WA 99202 - (509) 838-3999 - email spokane@anateklabs.com

Authorized Signature,



Brock Gerger for Kathleen Sattler, Lab Manager

PQL	Practical Quantitation Limit
ND	Not Detected
MCL	EPA's Maximum Contaminant Level
Dry	Sample results reported on a dry weight basis
*	Not a state-certified analyte

This report shall not be reproduced except in full, without the written approval of the laboratory
The results reported related only to the samples indicated.

Anatek Labs, Inc.

1282 Alturas Drive - Moscow, ID 83843 - (208) 883-2839 - email moscow@anateklabs.com
504 E Sprague Ste. D - Spokane, WA 99202 - (509) 838-3999 - email spokane@anateklabs.com

Certifications

Code	Description	Facility	Number
W WA DOE	Washington Department of Ecology	Anatek-Spokane, WA	C585

Anatek Labs, Inc.

1282 Alturas Drive - Moscow, ID 83843 - (208) 883-2839 - email moscow@anateklabs.com
504 E Sprague Ste. D - Spokane, WA 99202 - (509) 838-3999 - email spokane@anateklabs.com

Quality Control Data

Inorganics

Analyte	Result	Qual	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit
Batch: BEH0489 - W Wet Chem										
Calibration Check (BEH0489-CCV1)					Prepared & Analyzed: 08/12/24 11:11					
pH	6.05			pH Units in Water	6.00		101	0-200		
Batch: BEH0661 - W Ions										
Blank (BEH0661-BLK1)					Prepared & Analyzed: 08/14/24 11:35					
Sulfate	ND		1.50	mg/kg wet						
Chloride	ND		1.50	mg/kg wet						
LCS (BEH0661-BS1)					Prepared & Analyzed: 08/14/24 09:09					
Sulfate	3.95			mg/L	4.00		98.7	90-110		
Chloride	3.78			mg/L	4.00		94.5	90-110		

WEH0494

Due: 08/26/24

CHAIN OF CUSTODY RECORD

GeoEngineers
523 East Second Avenue
Spokane, Washington 99202
(509) 363-3125

DAT

PAG

LAB

LAB NO. 04420-218-10

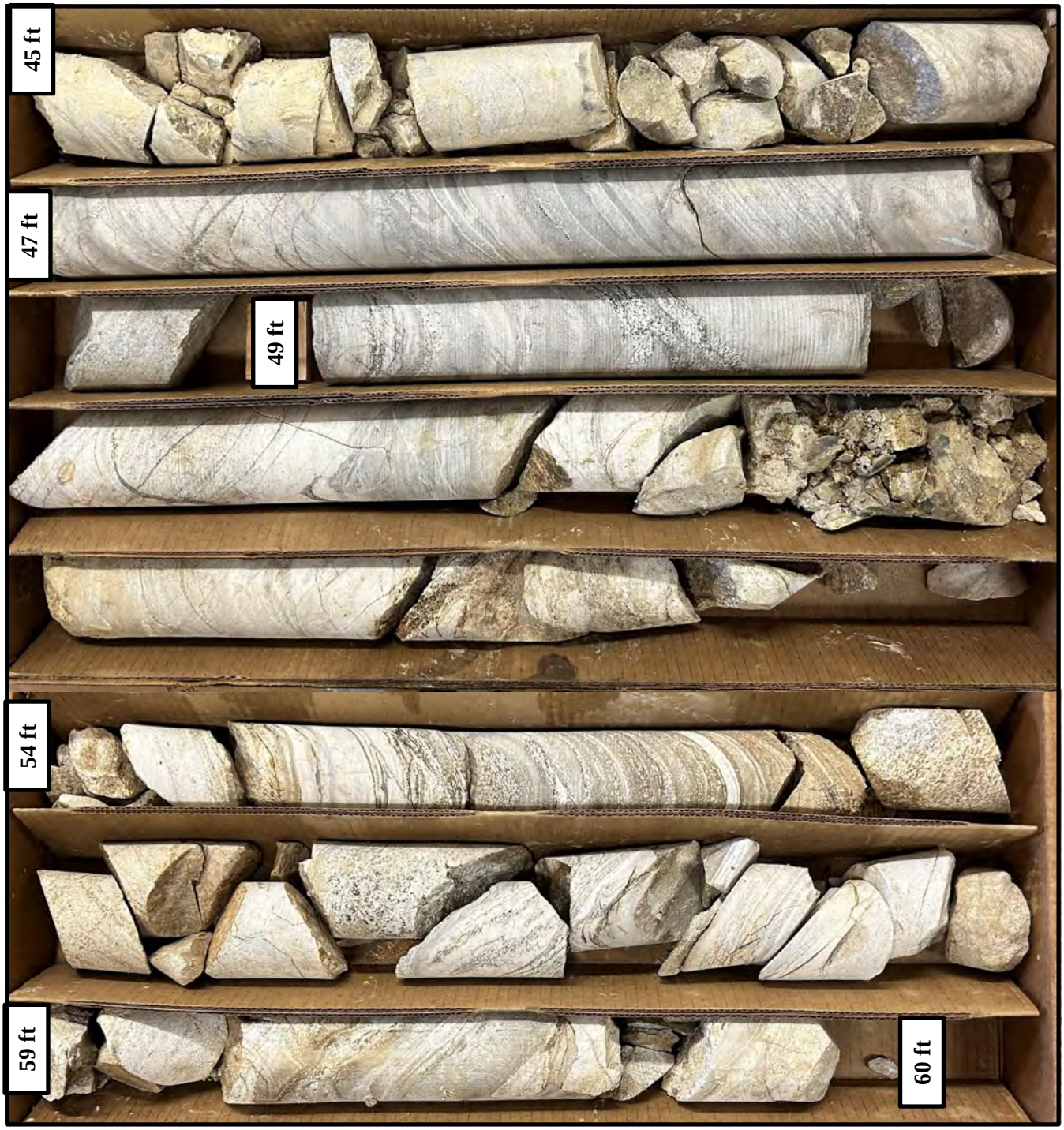
PROJECT NAME/LOCATION SH-3 St. Marries River
PROJECT NUMBER 04420-218-10
PROJECT MANAGER Alysa
SAMPLED BY GEE

SAMPLE IDENTIFICATION	SAMPLE COLLECTION DATE	TIME	MATRIX	# OF CONTAINERS	ANALYSIS REQUIRED: pH	Resistivity	Soluble sulfate	Soluble chloride					NOTES/COMMENTS (Preserved, filtered, etc.)
GEE-2-24 S-1@2.5'	7.09.24	—	soil	1	✓	✓	✓	✓					

RELINQUISHED BY	FIRM	RELINQUISHED BY	FIRM	RELINQUISHED BY	FIRM
SIGNATURE	GEE	SIGNATURE		SIGNATURE	
PRINTED NAME	Gernet Tefferi	PRINTED NAME		PRINTED NAME	
DATE	TIME	DATE	TIME	DATE	TIME
RECEIVED BY	FIRM Anatek	RECEIVED BY	FIRM	RECEIVED BY	FIRM
SIGNATURE		SIGNATURE		SIGNATURE	
PRINTED NAME	Blake Sattler	PRINTED NAME		PRINTED NAME	
DATE	TIME	DATE	TIME	DATE	TIME

ADDITIONAL COMMENTS: gtefferi@geoengineers.com

Appendix D
Photographs



Boring GEI-1-24 rock core from 45 to 60 feet.

Rock Core Photographs	
SH-3, St. Marie's River Bridge ITD Project No. A023(257); Key No. 23257 Shoshone County, Idaho	
GeoEngineers	Figure D-1

4420-218-00 Date Exported: 10/15/2024



Boring GEI-2-24 rock core from 44.2 to 64.2 feet.

Rock Core Photographs	
SH-3, St. Marie's River Bridge ITD Project No. A023(257); Key No. 23257 Shoshone County, Idaho	
GeoEngineers	Figure D-2

4420-218-00 Date Exported: 10/15/2024



Location of Stream Bed Sample 1.

Stream Bed Sample Photographs

SH-3, St. Marie's River Bridge
ITD Project No. A023(257); Key No. 23257
Shoshone County, Idaho

GeoEngineers

Figure D-3

4420-218-00 Date Exported: 10/15/2024



Location of Stream Bed Sample 2.

Stream Bed Sample Photographs

SH-3, St. Marie's River Bridge
ITD Project No. A023(257); Key No. 23257
Shoshone County, Idaho

GeoEngineers

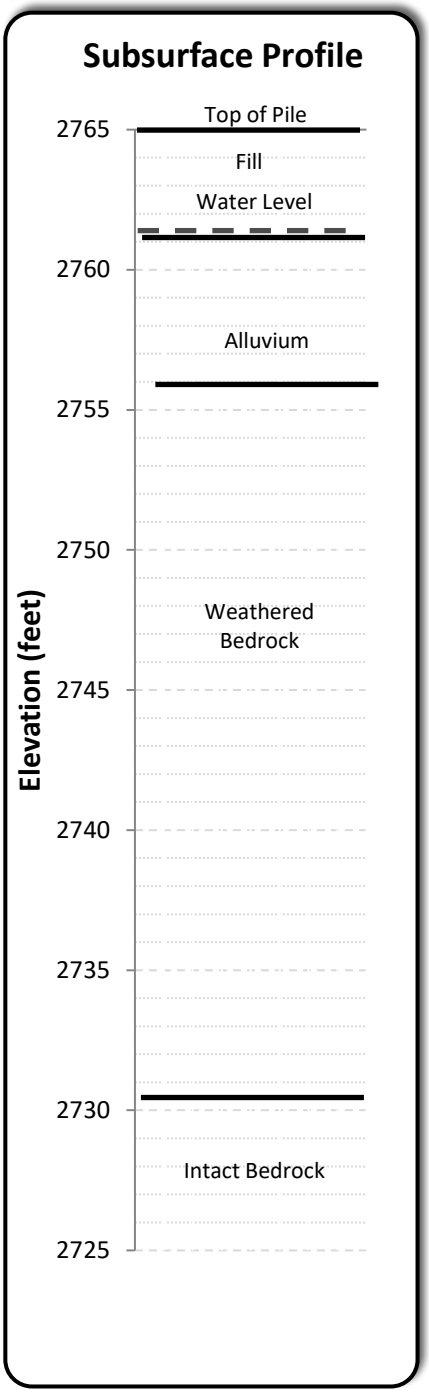
Figure D-4

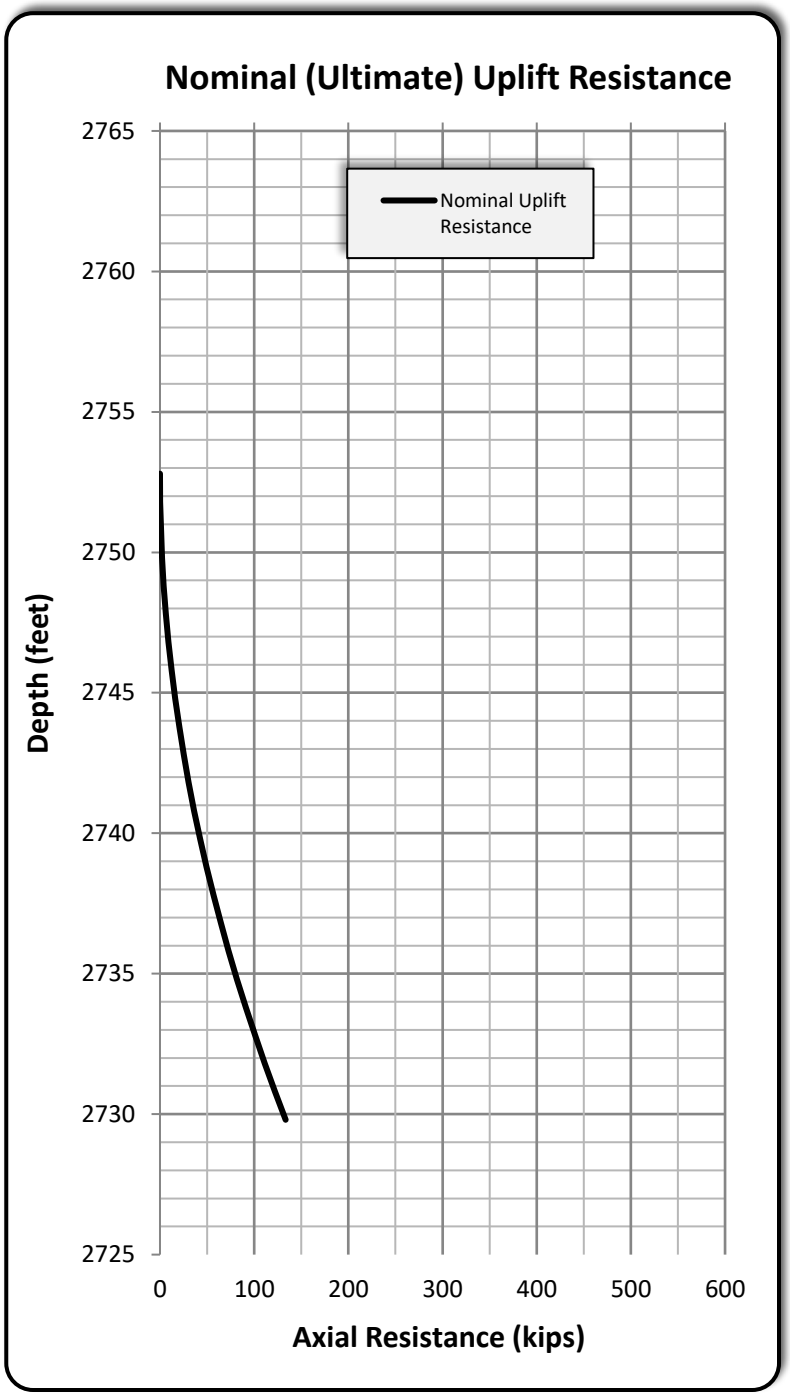
4420-218-00 Date Exported: 10/15/2024

Appendix E
Axial and Lateral Pile Capacity Plots

AASHTO DRIVEN PILE RESISTANCE

23257 – SH-3 St Maries River Bridge – Abutment 1&2 with Design Scour – Driven HP14x117 Pile





General Notes

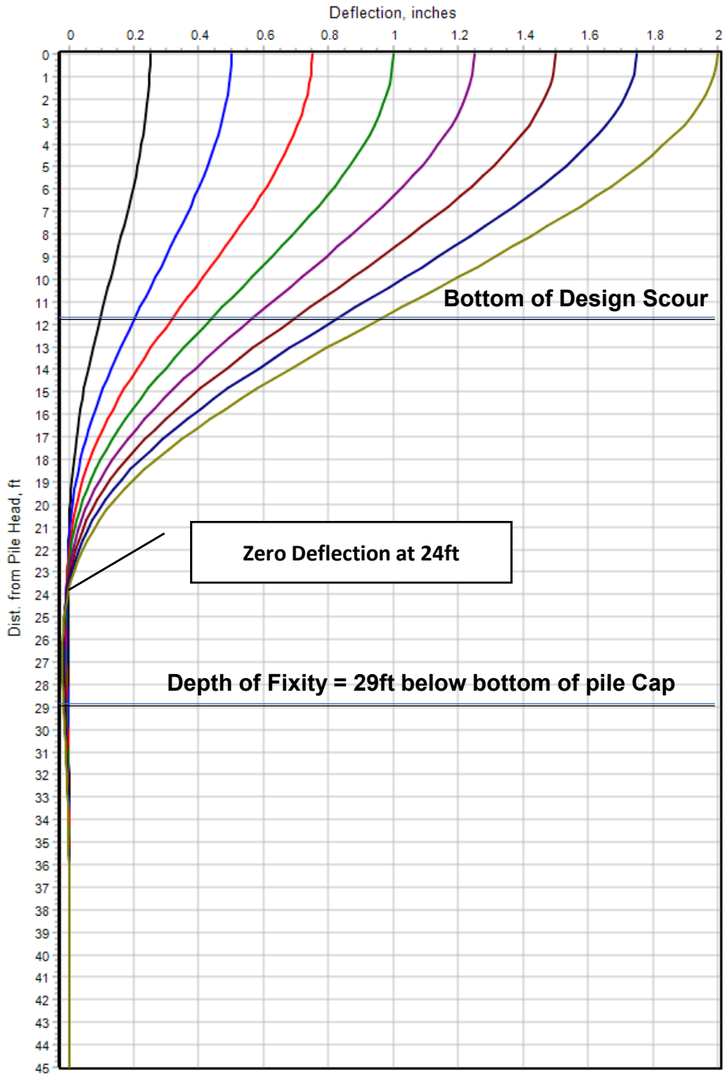
1. The pile resistances were developed in accordance with the AASTHO LRFD Bridge Design Manual using the Alpha method and the Nordland/Thurman method.
2. The plots are based on a single pile and do not consider group effects of closely spaced piles.
3. Pile settlement is estimated to be negligible under Service loading conditions.
4. The appropriate LRFD resistance factors as described in the report text should be applied to the nominal resistances in the plots. Strength limit state plots apply a resistance factor of 0.65 assuming two test piles with PDA testing and CAPWAP analysis are completed.

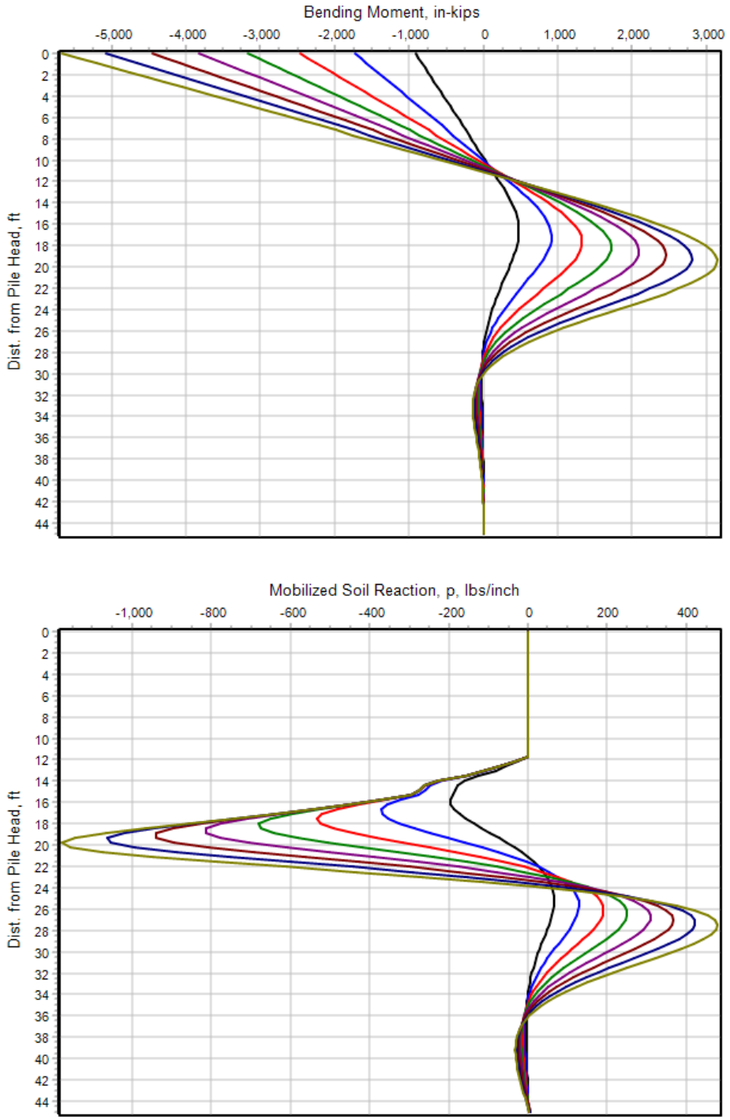
Axial Resistance
South Abutment – Driven HP14x117 Pile

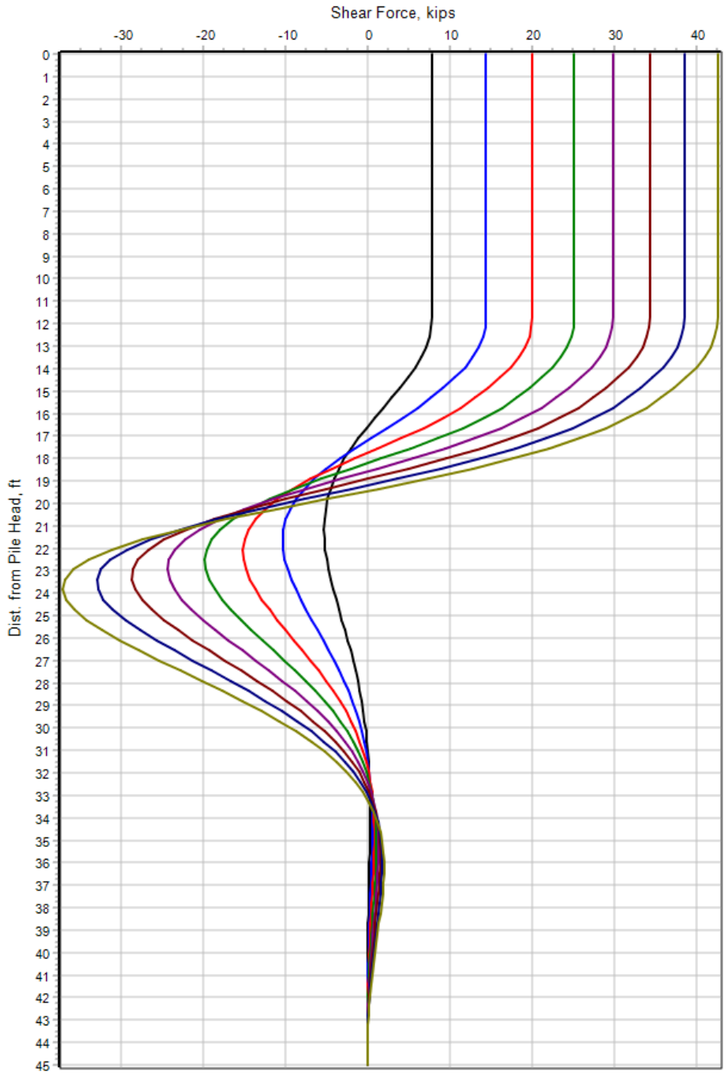
SH-3 St Maries River Bridge
ITD Project No. A023(257) (Key No. 23257)
Shoshone County, Idaho

GeoEngineers

Figure E-1







Assumptions

- HP 14x117
- Grade 50 Steel
- Fixed-head condition
- Horizontal ground surface at design scour elevation of 2753
- Single pile with no group effects or p-multipliers
- 172 Kip vertical load
- Top of pile at elevation 2764.5 ft.

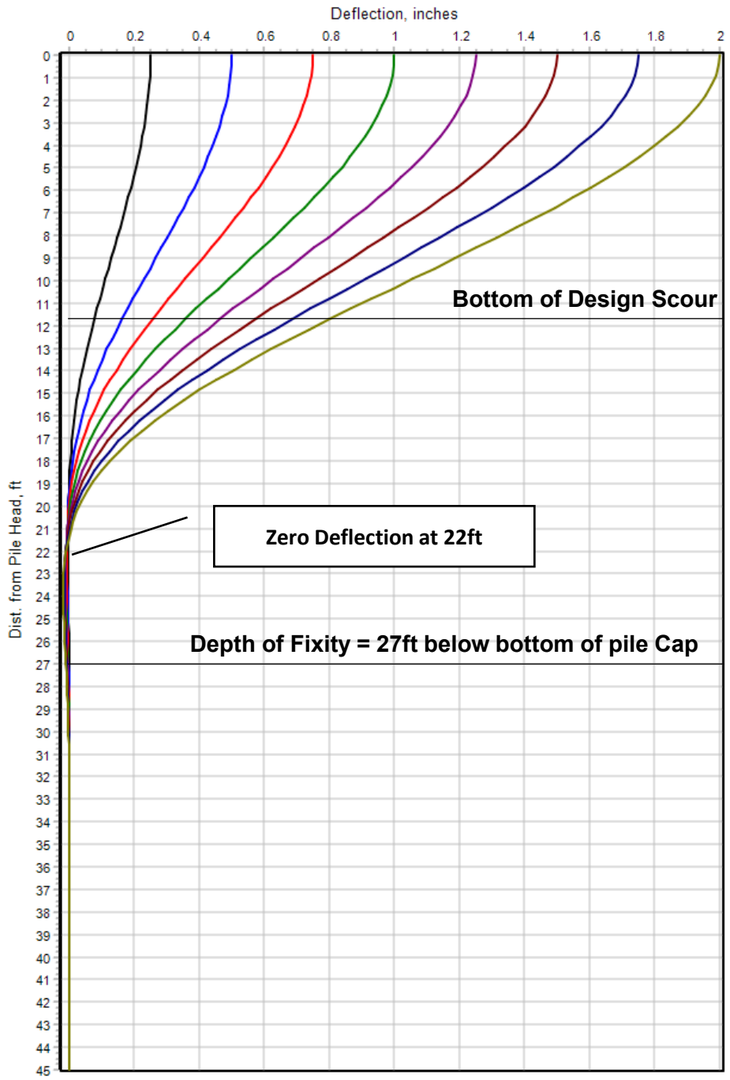
Reference: LPILE v2022.12.07 (Ensoft)

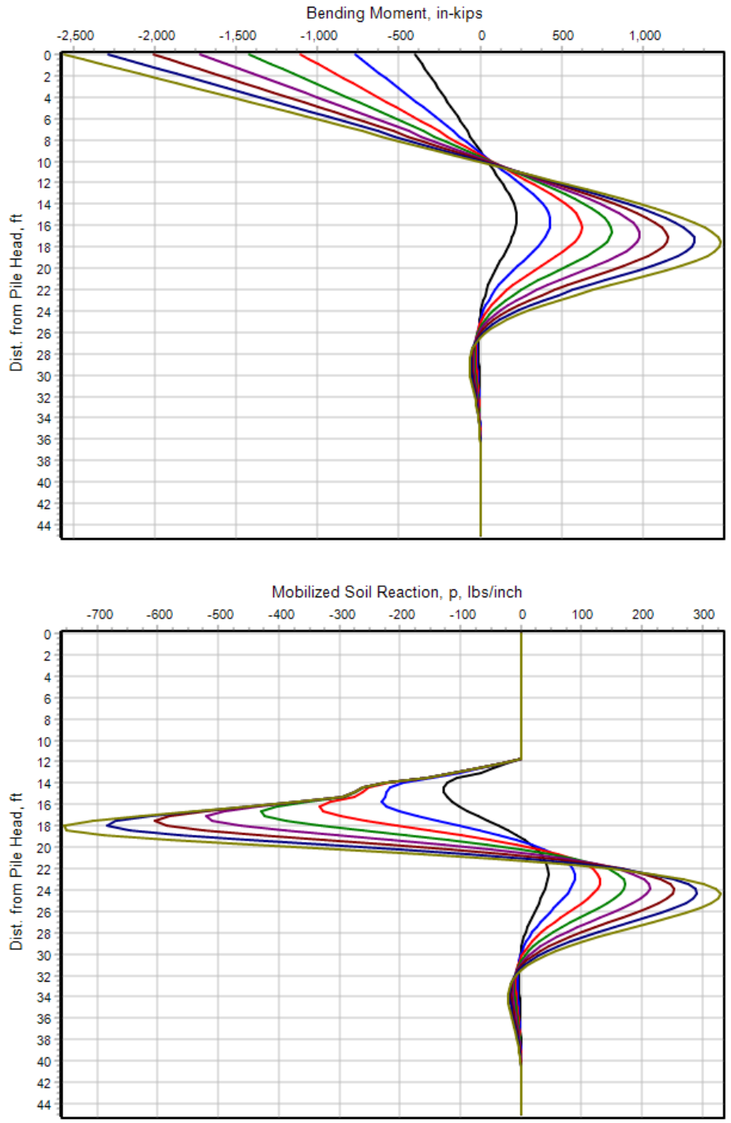
LPile Plots – St Maries
Abutments 1 and 2 – Strong Axis

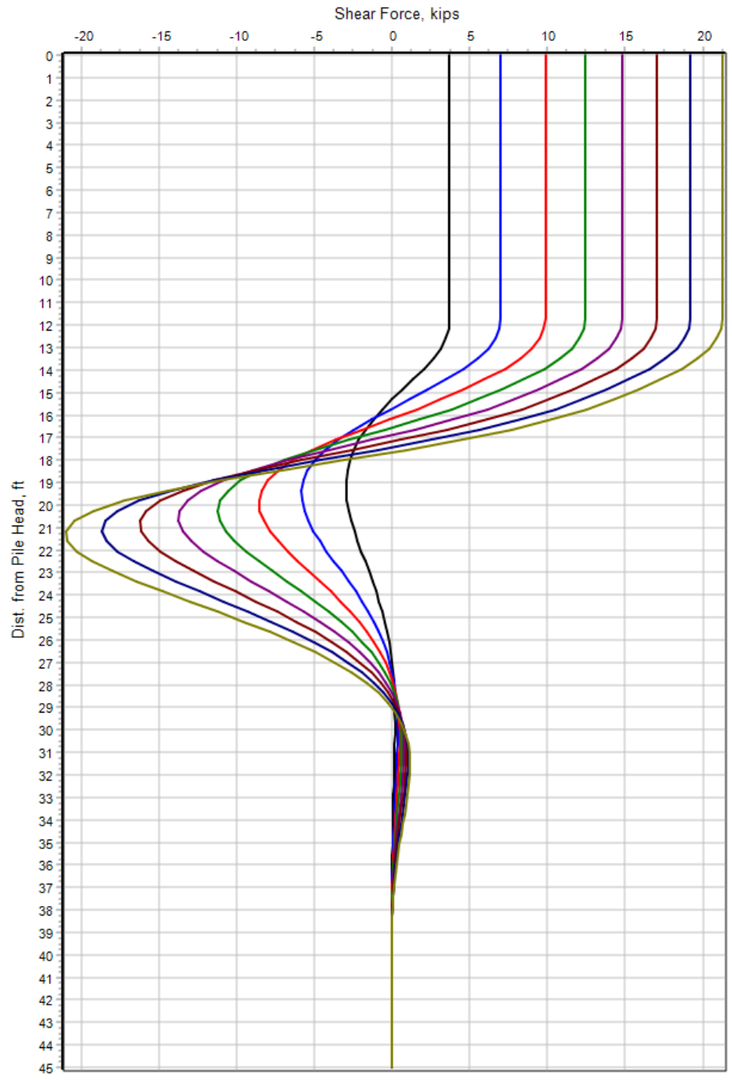
SH-3 St. Maries River Bridge
ITD Project No. A023(257); Key No. 23257
Shoshone County, Idaho

GeoEngineers

Figure E-2







Assumptions

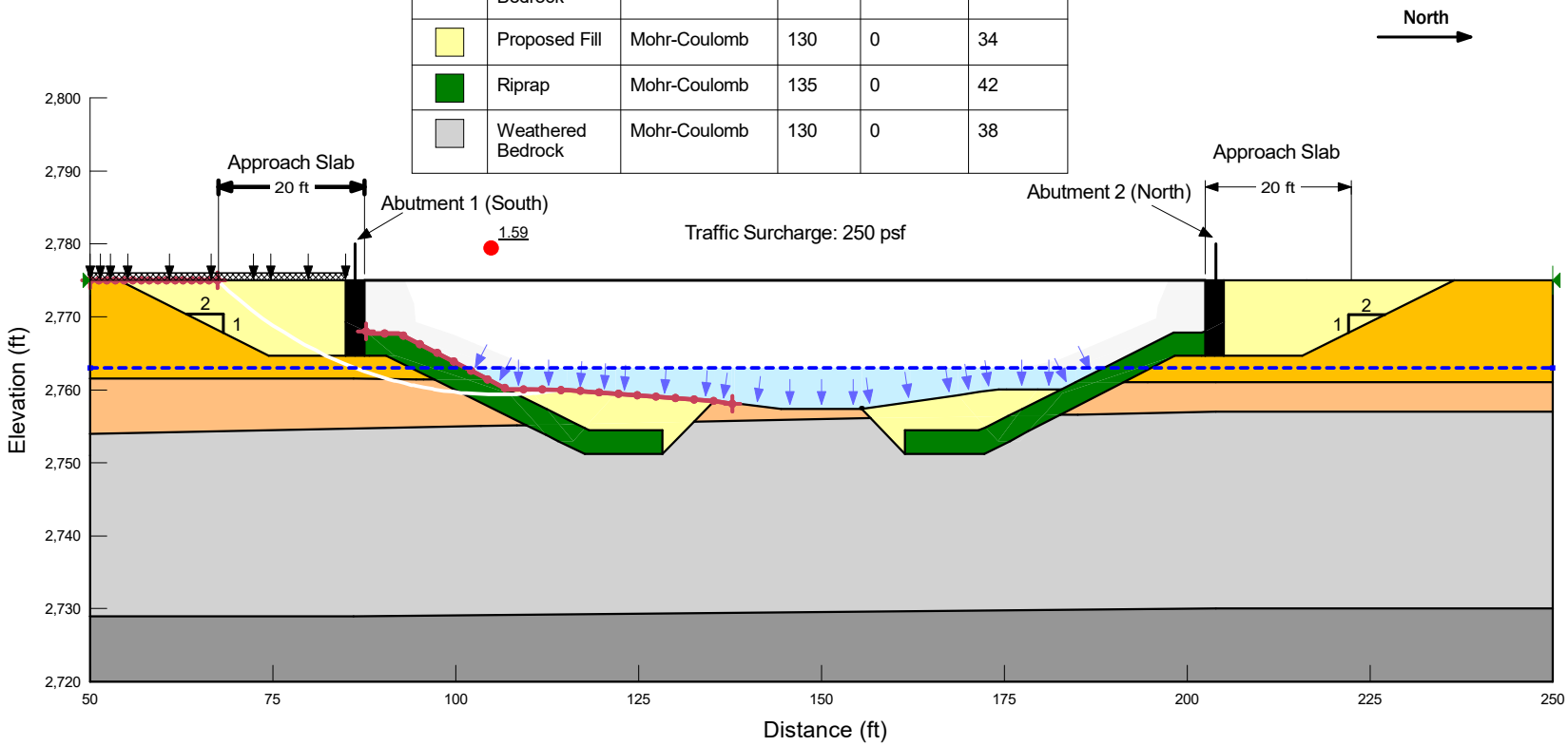
- HP 14x117
- Grade 50 Steel
- Fixed-head condition
- Horizontal ground surface at design scour elevation of 2753
- Single pile with no group effects or p-multipliers
- 172 Kip vertical load
- Top of pile at elevation 2764.5 ft.

Reference: LPILE v2022.12.07 (Ensoft)

LPile Plots – St Maries Abutments 1 and 2 – Weak Axis	
SH-3 St. Maries River Bridge ITD Project No. A023(257); Key No. 23257 Shoshone County, Idaho	
GeoEngineers	**Figure E-3**

Appendix F
Global Stability

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Alluvium	Mohr-Coulomb	120	0	35
	Concrete	High Strength	150		
	Existing Fill	Mohr-Coulomb	115	0	33
	Intact Bedrock	High Strength	140		
	Proposed Fill	Mohr-Coulomb	130	0	34
	Riprap	Mohr-Coulomb	135	0	42
	Weathered Bedrock	Mohr-Coulomb	130	0	38



Abutment 1
Static (Spencer)

St Maries River Bridge

Shoshone County, Idaho

GeoEngineers

Figure F-1

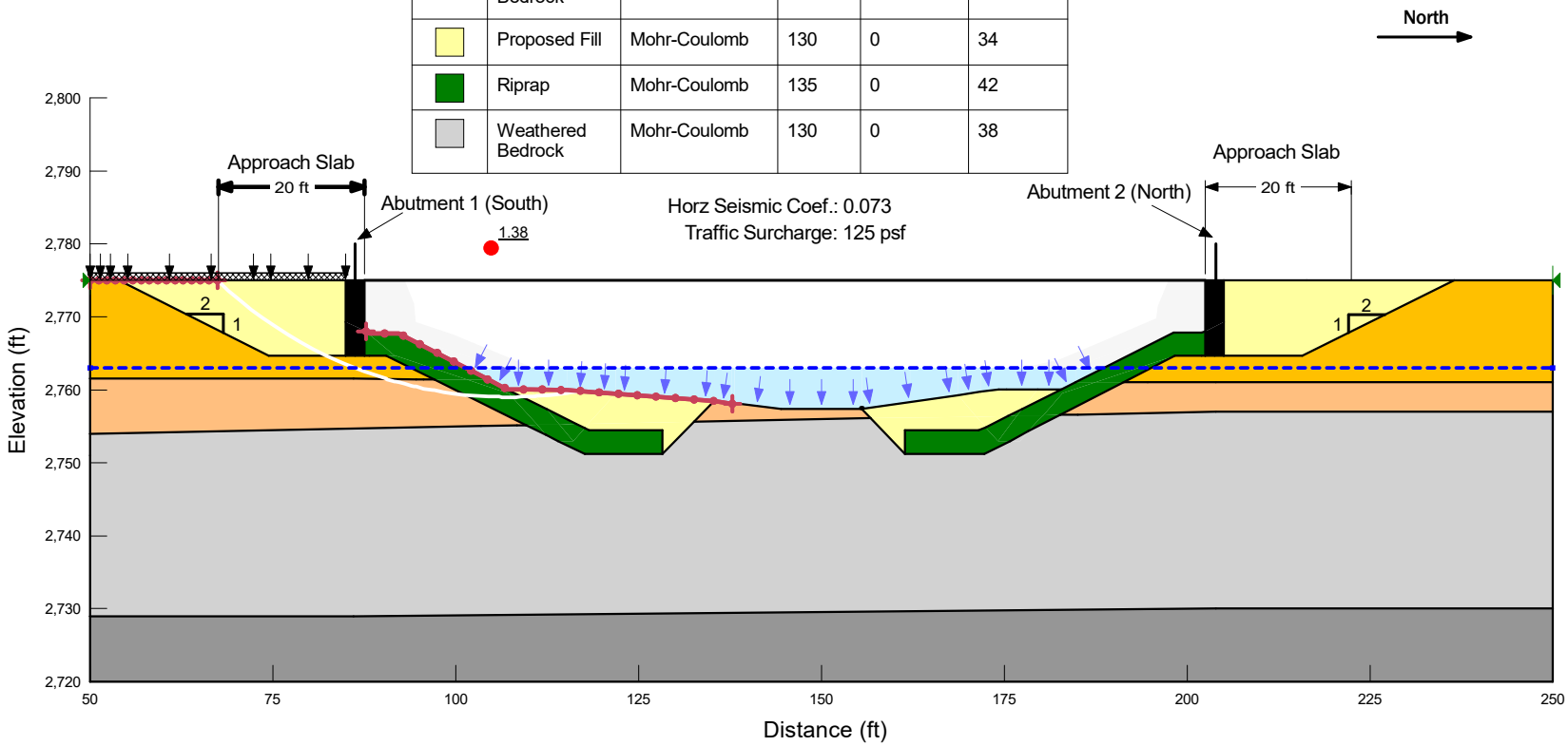
Notes:

1. Retaining Walls designed with minimum embedment and minimum CSBC as shown on drawing WADT-M-05.

Datum: NAVD88

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Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Alluvium	Mohr-Coulomb	120	0	35
	Concrete	High Strength	150		
	Existing Fill	Mohr-Coulomb	115	0	33
	Intact Bedrock	High Strength	140		
	Proposed Fill	Mohr-Coulomb	130	0	34
	Riprap	Mohr-Coulomb	135	0	42
	Weathered Bedrock	Mohr-Coulomb	130	0	38



Notes:

1. Retaining Walls designed with minimum embedment and minimum CSBC as shown on drawing WADT-M-05.

Datum: NAVD88

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Abutment 1
Seismic (Spencer)

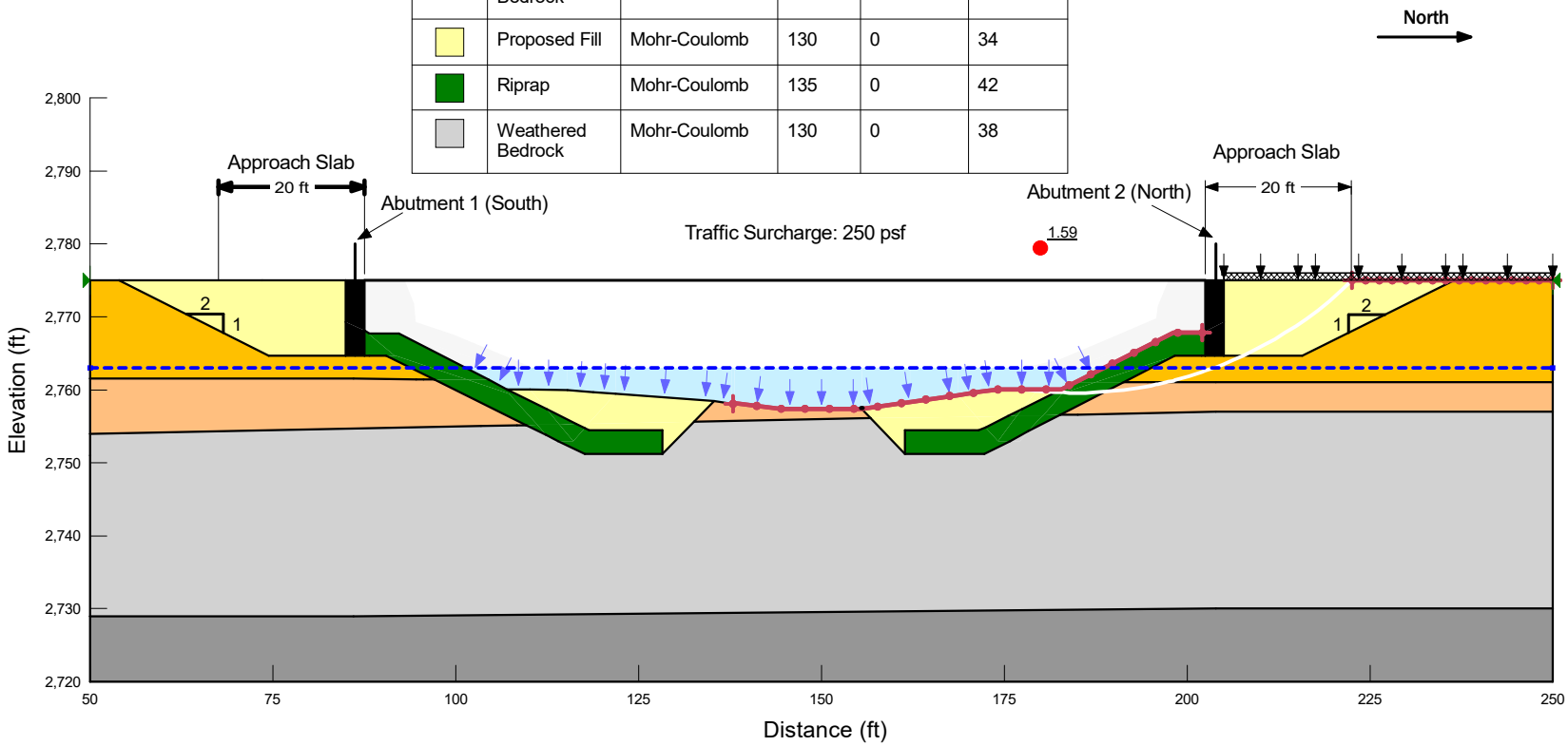
St Maries River Bridge

Shoshone County, Idaho

GeoEngineers

Figure F-2

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Alluvium	Mohr-Coulomb	120	0	35
	Concrete	High Strength	150		
	Existing Fill	Mohr-Coulomb	115	0	33
	Intact Bedrock	High Strength	140		
	Proposed Fill	Mohr-Coulomb	130	0	34
	Riprap	Mohr-Coulomb	135	0	42
	Weathered Bedrock	Mohr-Coulomb	130	0	38



Abutment 2
Static (Spencer)

St Maries River Bridge

Shoshone County, Idaho

GeoEngineers

Figure F-3

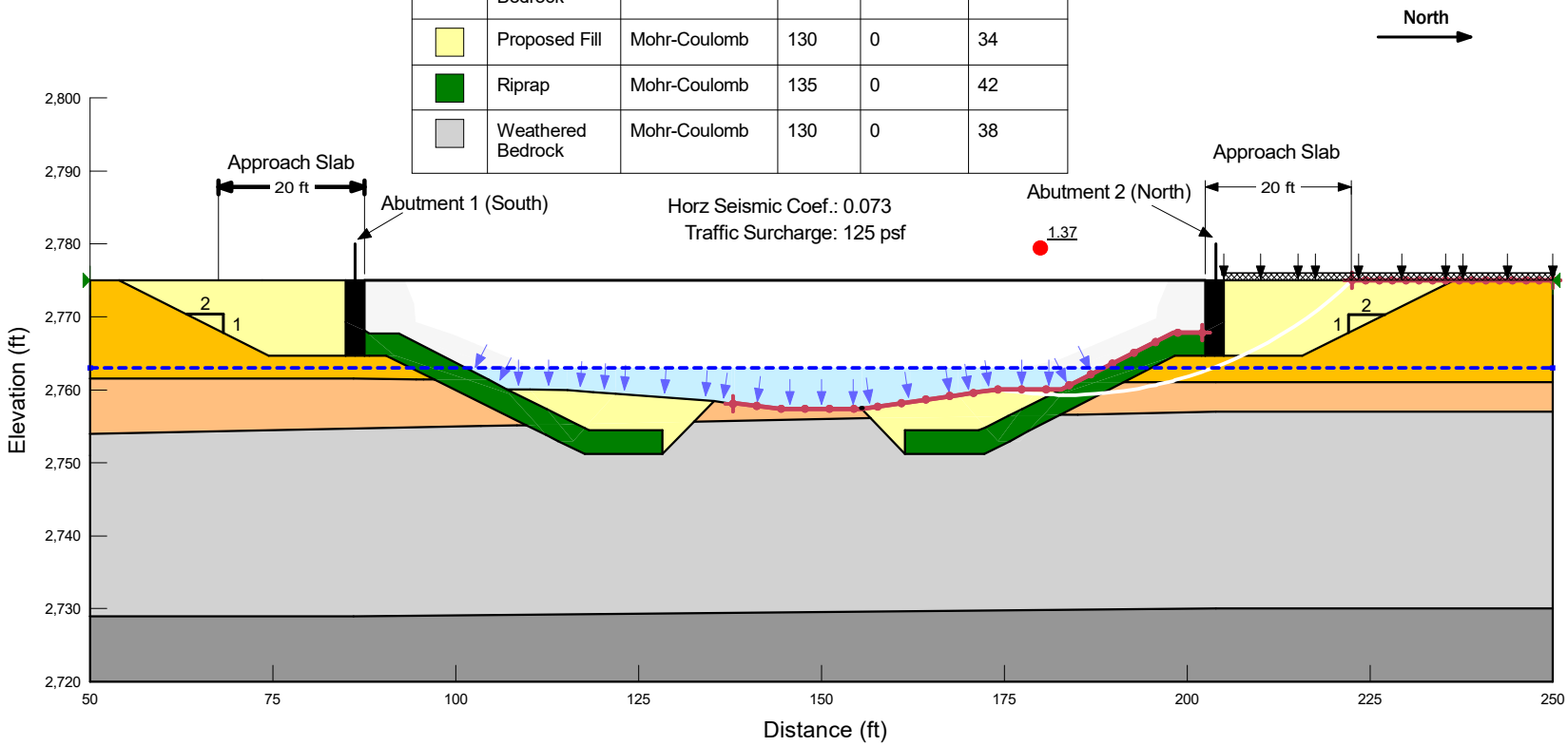
Notes:

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Datum: NAVD88

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	Riprap	Mohr-Coulomb	135	0	42
	Weathered Bedrock	Mohr-Coulomb	130	0	38



Abutment 2
Seismic (Spencer)

St Maries River Bridge

Shoshone County, Idaho

GeoEngineers

Figure F-4

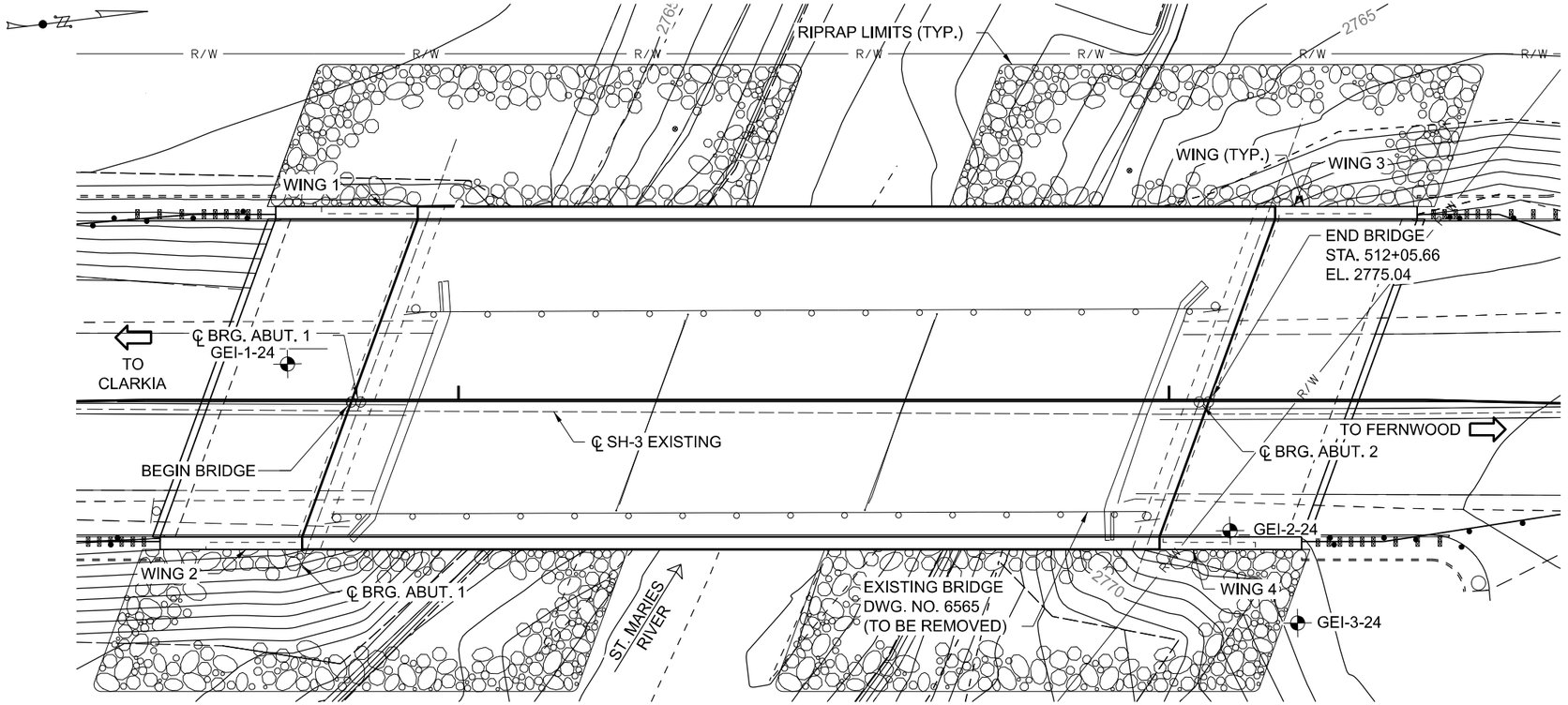
Notes:

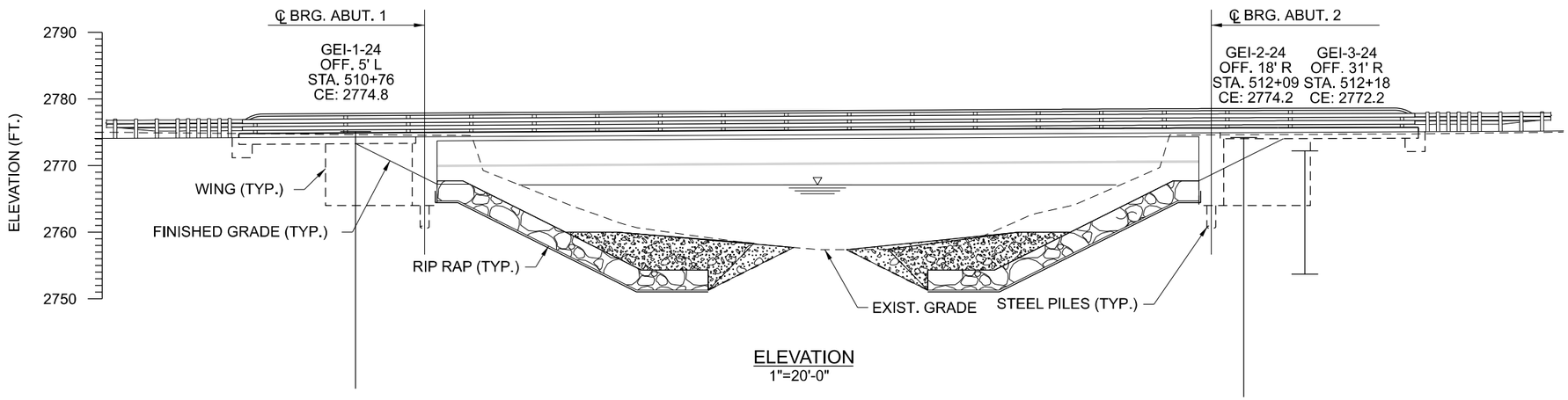
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Appendix G
Foundation Investigation Plats





ELEVATION
1"=20'-0"

REVISIONS			
NO.	DATE	BY	DESCRIPTION

DESIGNED C. LONGTON
DESIGN CHECKED M. McARTHUR
DETAILED J. FELLOWS
DWG. CHECKED M. McARTHUR
CORRECTIONS

SCALES SHOWN ARE FOR 11" X 17" PRINTS ONLY

CADD FILE NAME
23257 bdtl D01.dgn

DRAWING DATE: DECEMBER 2025

IDAHO TRANSPORTATION DEPARTMENT
YOUR Safety→YOUR Mobility→YOUR Economic Opportunity
GeoEngineers

ENGLISH

PROJECT NO.
A023(257)

FOUNDATION INVESTIGATION (SHEET 1 OF 2)

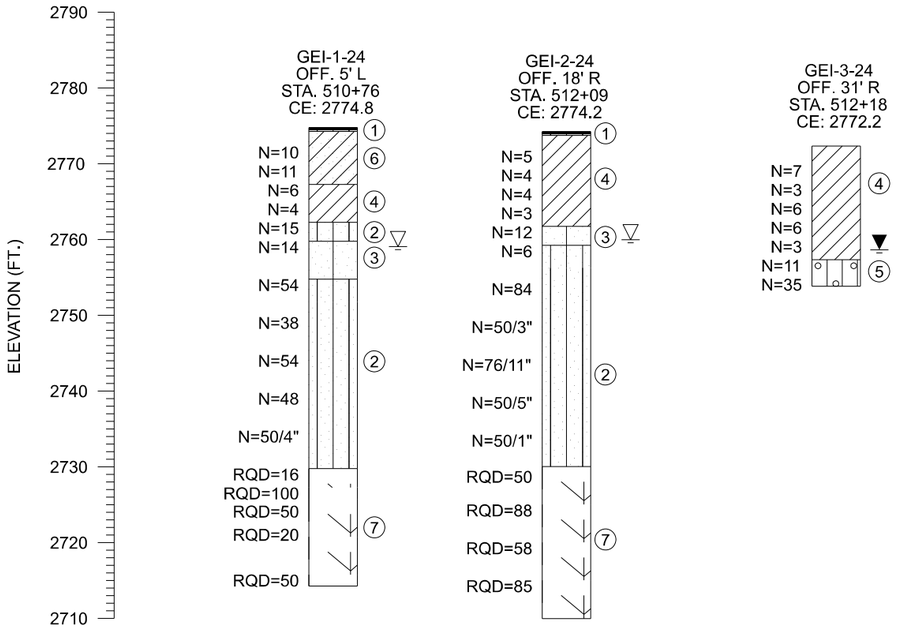
121' STEEL GIRDER BRIDGE
SH-3 OVER ST MARIES RIVER
STA 511+45.32

BRIDGE PLANS

BRIDGE KEY NO. 10091	
COUNTY SHOSHONE	KEY NO. 23257
BRIDGE DWG. NO. 19431	SHEET 5 OF 31

PROFESSIONAL ENGINEER
LICENSED
20370
12/16/25
STATE OF IDAHO
MORGAN McARTHUR

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LEGEND

- *LAYER 1 — ASPHALT CONCRETE (AC)*
- *LAYER 2 — SILTY SAND WITH GRAVEL (SM)*
- *LAYER 3 — SAND WITH SILT, GRAVEL AND OCCASIONAL COBBLES (SP-SM)*
- *LAYER 4 — CLAY WITH SAND (CL)*
- *LAYER 5 — GRAVEL WITH SILT, SAND AND OCCASIONAL COBBLES (GP-GM)*
- *LAYER 6 — CLAYEY SANDS, SAND-CLAY MIXTURES (SC)*
- *LAYER 7 — QUARTZITE*

OFF	*OFFSET FROM CENTERLINE (FEET)*
CE	*COLLAR ELEVATION (FEET)*
N	*BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SPLIT-SPOON SAMPLER A DISTANCE OF 12 INCHES*
N [1]	*SAMPLER ON APPARENT COBBLE. BLOW COUNT LIKELY OVERSTATED*
N [2]	*FIELD BLOW COUNT FOR OVERSIZED SAMPLER CONVERTED TO STANDARD SPT N-VALUE USING THE LACROIX-HORN EQUATION (ASTM SPT-523. 1973)*
▼	*GROUNDWATER MEASURED IN PIEZOMETER*
▽	*GROUNDWATER MEASURED AT TIME OF DRILLING*
①	*SOIL LATER (SEE LAYER NUMBERS TO LEFT)*

NOTES:

1. THE SUBSURFACE CONDITIONS SHOWN RESPRESENT THE APPROXIMATE DEPTHS OF CHANGES IN SOIL TYPE. THE TRANSITION BETWEEN MATERIALS MAY BE GRADUAL OR ABRUPT.

2. THIS PROFILE SHOWS SUBSURFACE CONDITIONS SHOWN AT THE SPECIFIC BORING LOCATIONS AT THE TIME THE EXPLORATIONS WERE COMPLETED. THEY MAY NOT BE REPRESENTATIVE OF SUBSURFACE SOIL AND GROUNDWATER CONDITIONS AT OTHER LOCATIONS AND TIME.

3. COBBLES ARE PRESENT AND SHOULD BE ANTICIPATED TO BE ENCOUNTERED AT THE SITE.

4. PERCHED GROUNDWATER WAS ENCOUNTERED IN ONE BORING FROM 2.5' TO 5' BELOW THE GROUND SURFACE AT THE TIME OF DRILLING. GROUNDWATER WAS NOT ENCOUNTERED IN THE OTHER EXPLORATIONS AT THE TIME OF EXPLORATION. GROUNDWATER LEVELS ARE APPROXIMATE AND SHOULD BE EXPECTED TO FLUCTUATE.

5. THE FOUNDATION INVESTIGATION REPORT, EXPLORATION LOGS, AND OTHER INFORMATION RELATED TO FOUNDATION INVESTIGATION FOR THIS PROJECT ARE AVAILABLE AT THE ITD DISTRICT MATERIALS SECTION.

P:\4 4420218\CAD\00\FIP\23257 bdtl D01.dgn

REVISIONS			
NO.	DATE	BY	DESCRIPTION
1			
2			
3			
4			
5			

DESIGNED C. LONGTON
DESIGN CHECKED M. McARTHUR
DETAILED J. FELLOWS
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ENGLISH
PROJECT NO.
A023(257)

FOUNDATION INVESTIGATION (SHEET 2 OF 2)
121' STEEL GIRDER BRIDGE
SH-3 OVER ST MARIES RIVER
STA 511+45.21

BRIDGE PLANS
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COUNTY SHOSHONE
KEY NO. 23257
BRIDGE DWG. NO. 18431
SHEET 6 OF 31

PROFESSIONAL ENGINEER
LICENSED
20370
12/16/25
STATE OF IDAHO
MORGAN McARTHUR