GeoProfessional Innovation

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Mr. Scott Linja, P.E.
Keller Associates, Inc.
733 5th St., Ste. A
Clarkston, WA 99403

RE: **Geotechnical Engineering Report**
Gun Club Road, Lapwai Road to Stewart Ave
Project No. AO23(881)
Key No. 23881
Nez Perce County, Idaho

Greetings, Scott:

GeoProfessional Innovation Corporation (GPI) prepared this Geotechnical Engineering Report referencing the Idaho Transportation Department (ITD) Materials Manual Section 230 for the Gun Club Road, Lapwai Road to Stewart Avenue project located along the eastern extents of Lewiston in Nez Perce County, Idaho. These improvements are intended to provide enhanced roadway safety for local patrons and commuters traversing this key travel-way. The purpose of our evaluation was to explore the subsurface conditions along the proposed project alignment and provide geotechnical recommendations to assist design and construction. We accomplished the scope of work outlined in our initial proposal dated April 18, 2023 and subsequent Scope Addendum dated March 4, 2025.

Herein, we summarize our field evaluation, laboratory tests results, and engineering analysis, as well as our geotechnical findings, opinions, and recommendations. Review and implement our geotechnical recommendations in their entirety. Portions or individual sections of our report cannot be relied upon without the supporting text in other pertinent sections.

We appreciate the opportunity to continue our working relationship with Keller on this project. Please do not hesitate to contact us if you have any questions or comments.

Sincerely,
GPI

Noah Hattrup

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Staff Engineer

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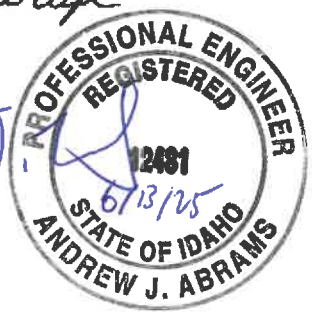


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PROJECT: Gun Club Road, Lapwai Road to Stewart Avenue
COUNTY: Nez Perce County
KEY NO.: 23881

230.00 GEOTECHNICAL ENGINEERING REPORT

230.01 Introduction

This Geotechnical Engineering Report provides design team members and construction personnel with recommendations for use in design and construction of road widening and safety improvements along Gun Club Road between Lapwai Road (Lindsay Creek Road) and Stewart Avenue. The improvements are part of a partnered effort between Nez Perce County (County) and the Local Highway Technical Assistance Council (LHTAC).

Geotechnical recommendations specifically addressing pavement improvements for Gun Club Road are provided in a separate *Roadway Materials Report* referencing the Idaho Transportation Department (ITD) *Materials Manual, Section 240*.

Design plans prepared by Keller Associates, Inc. (Keller) show the planned alignment grades are expected to remain relatively consistent with existing grades along the alignment. The project will include primarily shoulder widening along the east side of Gun Club Road, with new asphalt pavement over widened areas. Other improvements will include new striping, upgraded signage, and guardrails.

The existing Gun Club Road alignment transitions from cut to fill between the western and eastern edges of pavement. The eastern side of the road is situated atop fill embankments along the edge of the drainage east of the road. Existing embankment slopes appear to be relatively steep, about 1.5H:1V (horizontal to vertical) in certain locations. Vertical elevation changes between the existing Gun Club Road surface and the creek bottom to the east range from approximately 10 feet near the north end of the alignment near Lapwai Road, to more than 100 feet at the south end of the alignment near the Stewart Avenue intersection. The western side of the road is situated primarily over cuts into native soil and bedrock. Existing cut slopes along the western extent of the road alignment are relatively steep along much of the alignment, ranging from 2H:1V to near vertical in some locations.

Grade changes associated with road widening will be accomplished along the eastern side of the entire alignment, requiring fill embankments and geosynthetic-reinforced, wire-faced retaining walls to facilitate grade changes adjacent to the widened roadway. Keller's design drawings show the planned retaining walls will range from approximately 2 feet to 6 feet in exposed height, with embedment ranging from 2 to 4 feet below finished grades. Retaining walls will be constructed entirely in cuts below existing grade and will bear on subgrades comprising bedrock or recompacted native soil. Outside of planned retaining wall alignments, widening will be facilitated by fill embankments less than 2-feet tall (vertical) with finished slope inclinations up to 2H:1V.

The purpose of this report is to provide geologic conditions and geotechnical recommendations to aid Keller, the County, and LHTAC with assessing roadway design and construction, as well as to provide geotechnical information for contractor-supplied design of the planned retaining walls.

230.02 Field Exploration and Laboratory Testing

The subsurface conditions presented in this report are based on the results of our exploration and laboratory testing program. Field exploration was accomplished via soil borings on October 16, 2023, and test pits were accomplished on March 7, 2025. The time lapse between these 2 exploration efforts was associated with the County and LHTAC receiving cultural clearance from the Nez Perce Tribe to perform test pit exploration. Both exploration phases were observed and documented by a GPI Geotechnical Engineer. The methods and results of the field and laboratory testing programs are summarized in the following sections.

230.02.01 Borings/Test Pits

GPI observed 8 exploratory borings within the roadway prism. Borings were advanced at various locations of interest outlined by Keller along the alignment. Boring locations were modified somewhat during exploration due to traffic safety and existing utility conflicts. Borings extended 11.5 to 16.5 feet below the existing ground surface. A boring summary is provided in Table 1.

Table 1: Boring Summary

Boring Number	Approximate Latitude[1]	Approximate Longitude[1]	Total Exploration Depth (feet)	Uncontrolled Fill Depth (feet)	Bedrock Surface Depth (feet)
B-23073A-1	46.391551°	-116.980845°	6.5	N.E. [2]	N.E. [2]
B-23073A-2	46.392932°	-116.980822°	10.4	N.E.[2]	8.0
B-23073A-3	46.394924°	-116.980739°	9.5	2.5	9.0
B-23073A-4	46.398767°	-116.980774°	9.5	N.E.[2]	7.5
B-23073A-5	46.404824°	-116.980881°	11.5	5.5	9.0
B-23073A-6	46.407648°	-116.979565°	9.0	>9.0	N.E. [2]
B-23073A-7	46.409622°	-116.978354°	12.0	N.E.[2]	N.E.[2]
B-23073A-8	46.411154°	-116.977028°	11.5	2.0	N.E.[2]

1. World Geodetic System of 1984 (WGS84)
2. Not Encountered (N.E.)

GPI also observed 7 exploratory test pits along the alignment. Test pits were advanced outside the roadway prism, extending into the fill embankment on the eastern edge of the roadway. Test pits were located 5 to 15 feet laterally off the roadway shoulder to reduce damage to the existing Gun Club Road pavement section and reduce traffic impacts during our work. Test pits extended 8.0 to 14.0 feet below the existing ground surface. A test pit summary is provided in Table 2.

Table 2: Test Pit Summary

Test Pit Number	Approximate Latitude[1]	Approximate Longitude[1]	Total Exploration Depth (feet)	Uncontrolled Fill Depth (feet)	Bedrock Surface Depth (feet)
TP-23073A-1	46.393445°	-116.980737°	13.0	7.0	12.0
TP-23073A-2	46.394914°	-116.980638°	12.0	4.5	11.0
TP-23073A-3	46.395361°	-116.980616°	12.0	2.5	11.0
TP-23073A-4	46.398795°	-116.980631°	12.0	6.0	9.5
TP-23073A-5	46.400028°	-116.980901°	14.0	5.0	N.E.[2]
TP-23073A-6	46.401113°	-116.980952°	8.5	5.0	7.5
TP-23073A-7	46.402805°	-116.980886°	8.0	4.0	7.0

1. World Geodetic System of 1984 (WGS84)
2. Not Encountered (N.E.)

During exploration operations, traffic control, including signs and flaggers, was used to close the northbound (eastern) roadway lane along Gun Club Road and facilitate exploration. Borings were completed using a trailer-mounted G2400 drill rig equipped with 6-inch hollow-stem augers. Test pits were completed using a CASE 580 backhoe equipped with a 2.0-foot-wide bucket. GPI visually classified and described the soil profiles encountered during exploration, referencing the *Unified Soil Classification System* (USCS). Appendix A provides the individual exploration logs summarizing the conditions encountered along with a USCS explanation, which should be used to interpret soil descriptions and references throughout this report. Borings were backfilled with native soil upon completion and plugged with bentonite. Test pits were loosely backfilled with site soil. Where borings penetrated the existing Gun Club Road pavement section, the surface was patched with cold asphalt.

230.02.02 Field Tests

Split-spoon soil samples were recovered in borings at approximate 2.5- to 5-foot intervals, and Standard Penetration Test (SPT) N-values were recorded for each sample referencing ASTM D1586. Exploratory boring logs present the blow counts for each 6-inch segment of the sampler. The SPT N-value blow counts are field values and have not been corrected for overburden pressure or dilation effects on the sample. GPI staff visually logged and classified the soil conditions, referencing ASTM D2488 during exploration.

230.02.03 Geophysical Exploration

No geophysical exploration was performed as part of the evaluation.

230.02.04 Laboratory Tests

Samples obtained during exploration were tested in GPI's laboratory to assess their engineering characteristics. Laboratory test results are provided in Appendix B, *Laboratory Test Results,* and on the individual exploratory boring logs in Appendix A. The laboratory tests outlined in Table 3 were accomplished referencing *American Association of State Highway and Transportation Officials* (AASHTO) procedures.

Table 3: Laboratory Testing Program

Tests Performed	Purpose
Grain Size Distribution (AASHTO T 11 and T 27)	To identify the constituent particles in the soil, describing the percent by weight of gravel, sand, and fines, ultimately classifying soil type per ASTM D2487.
Atterberg Limits (AASHTO T 89 and T90)	To determine the soil plastic limit, plasticity index, and liquid limit for soil type classification.
Moisture-Density Relationship Curve (AASHTO T 180)	To establish a maximum dry density and optimum moisture content.
Moisture Content Determination (AASHTO T 255)	To determine the soil's in-place moisture content.
Direct Shear (AASHTO T 236)	To estimate the soil's cohesion and angle of internal friction.
Soil pH, Electrical Resistivity & Sulfate Content	To estimate the soil's corrosion potential and impact on buried metal or concrete elements.

230.03 Surface Conditions

The existing Gun Club Road alignment is a 2-lane road that is paved with asphalt. The paved surface is approximately 26 feet wide, comprising 2, 13-foot-wide travel lanes. The roadway grade slopes downward to the north at approximately 9%. Roadway surface elevation at the south end of the alignment is approximately 1,215 feet above mean sea level (AMSL). At the north end of the alignment, the roadway surface elevation is approximately 910 feet AMSL. A relatively shallow drainage ditch exists on the western edge of the road alignment. Along the eastern side of the road, the ground surface slopes downward relatively steeply, up to about 1.5H:1V, into a large drainage pathway east of Gun Club Road. The roadway shoulders are covered by a mixture of grass, weeds, brush, and some trees in isolated locations.

230.04 Subsurface Conditions

Borings were performed within the existing Gun Club Road pavement prism. No vegetation or organic material was encountered at the ground surface in our boring locations. All borings encountered asphalt at the existing roadway surface, ranging from 2.2 to 5.0 inches thick, underlain by 4.0 to 10.5 inches of silty gravel base course. In borings B-23073A-6 and -7, asphalt pavement was underlain by 2.0 to 3.0 inches of bituminous surface treatment (BST).

All test pit explorations were performed approximately 5 to 15 feet east of the existing Gun Club Road pavement prism. Topsoil was encountered at the ground surface in TP-23073A-4, -6, and -7, comprising silt that was brown, loose, and moist, extending 1.2 to 4.0 feet below the ground surface. Trace vegetation and organics were encountered from 0.2 to 1.0 feet below the ground surface in all test pits.

Beneath asphalt pavement sections and surficial topsoil, we encountered the following primary subsurface units during our exploration:

Uncontrolled Fill: Uncontrolled fill was encountered in most of our explorations, extending 2.5 to more than 9.0 feet below the ground surface. The fill constitutes heterogeneous mixtures of silt, sand, gravel, and cobbles with isolated boulders. In some locations, the fill included various debris, such as metal, plastic, glass, wood, and other organic material. Fill was described as brown to gray, loose to dense, and moist in the locations explored. Due to the varying composition, consistency, and debris present within the fill, we consider it to be uncontrolled and not suitable to support new retaining walls or embankments.

Loess: Beneath asphalt, topsoil and uncontrolled fill, our exploration encountered loess soil comprising silt with varying sand content, that was described as tan to brown, soft to stiff and moist. Loess extended to the termination depth of some explorations, or to the basalt bedrock interface in others at 6.5 to 14.0 feet below the ground surface.

Alluvium: North of the Lone Pine Lane intersection, our explorations encountered alluvial soil beneath surficial asphalt, topsoil and fill. Alluvium comprised silty sand that was brown loose to medium dense and moist. This soil extended to the termination depth of borings B23073A-7 and -8 at 11.5 to 12.0 feet below the ground surface.

Basalt Bedrock: Basalt bedrock was encountered in several borings and test pits between 7.5 to 12.0 feet below the existing ground surface. Bedrock was described as dark grey with orange staining, moderately to highly fractured, moderately to highly weathered, and hard. Our drilling exploration equipment (Trailer-mounted G-2400) was able to penetrate basalt bedrock 0.5 to 2.5 feet in boring locations. Test pit excavation equipment (Case 580 backhoe) was able to penetrate bedrock 1.0 to 2.5 feet where encountered.

Consistent, static groundwater was not encountered during exploration and is not expected within the upper 100 feet below ground surface, referencing Idaho Department of Water Resources (IDWR) online well reports within 1 mile of the project site. Isolated, perched water tables may be encountered during construction as seeps or springs at any time of the year, depending on changes in local and regional precipitation and runoff. Specifically, we expect water may be encountered in site excavations at the boundary between differing subsurface units, such as the boundary between fill and native soil, or soil and bedrock.

Logs illustrating our exploration findings are included in Appendix A. Approximate exploration locations are shown on Plates 1 through 4, *Exploration Maps*

230.05 Conclusions and Recommendations

230.05.01 General

Based on the conditions encountered during exploration and our experience with similar projects, we recommend the planned geosynthetically reinforced, wire-faced retaining walls be supported on bedrock, native soil, or structural fill placed over these materials. The following sections provide our geotechnical recommendations to assist design and construction for the planned retaining walls based on this understanding.

230.05.02 Foundations

Typical concrete foundation elements are not expected for this project. Planned retaining walls and associated reinforced fill must bear on basalt bedrock or native subgrade soil compacted to Class A requirements. If desired for constructability, the contractor may place a levelling course of crushed gravel below initial course of reinforcement or wire facing baskets. The levelling course should be up to 1-foot thick, and comprise *Aggregate for Open Graded Base – Class 1 (Rock Cap)* conforming to Section 703.08 in the latest *ITD Standards* or *Granular Subbase* conforming to Section 703.11 in the latest *ITD Standards*, referencing the latest ITD Standard Specifications.

Bearing resistance for planned walls must be evaluated by the selected contractor as part of their wall design, referencing the AASHTO Load Resistance Factor Design (LRFD) Bridge Design Specifications, Section 11, and the Federal Highway Administration (FHWA) Publication No. FHWA-NHI-10-024; *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes.*

Referencing AASHTO LRFD Bridge Design Specifications Section 11.10.2.2, we recommend the minimum embedment for the reinforced soil mass at the wall face be at least 2-feet. This is because the finished ground surface in front of the wall is expected to slope down and away from the wall at inclinations of up to 1.5H:1V.

230.05.03 Lateral Pressures and Backfill

GPI recommends wire-faced retaining walls and associated geosynthetic reinforced zones be backfilled using *Rock Cap* compacted to Class A requirements. Backfill outside the reinforced zone may comprise *Granular Subbase* compacted to Class A requirements. Backfill must be keyed into the existing soil to facilitate uniform performance. Keying recommendation are illustrated in Plate 6, *Wall Backfill Schematic* attached to this report.

Uncontrolled fill, debris, and loose soil should be removed from beneath the planned wall foundation and reinforcing elements before placing backfill. Avoid overcompaction of the backfill behind the wall facing so that the wall facing and reinforcement are not displaced or damaged. Only light hand-operated compaction equipment should be used to compact backfill within 3 feet of wall faces.

Lateral earth pressures will be realized on the reinforced wall mass from retained soil behind the structure as well as any surcharges from traffic, equipment, or material placed adjacent to the walls. Lateral earth pressures from retained soil can be estimated using the equivalent fluid weights (EFW) in Table 4. These values assume no friction between the reinforced wall mass and retained soil. Where walls are allowed to translate laterally at the top (such as flexible retaining walls), active EFWs can be used.

Table 4: Static Equivalent Fluid Weights (EFW)

Rankine Lateral Earth Pressure Case	Equivalent Fluid Weight (EFW)[1]	
	Loess Silt with Sand ***(Φ = 36.5°, γ=115.8 pcf)*** ***Dry Conditions***	**Granular Subbase** ***(Φ = 34°, γ=140 pcf)*** **Dry Conditions**
At-rest case (no wall movement)	50 pcf	65 pcf
Active case (wall movement away from soil mass)	30 pcf	40 pcf
Passive case (wall movement toward soil mass)	400[2,3] pcf	495[2,3] pcf

1. Assumes flat backfill behind wall face.
2. Assumes ¾ inch of allowable lateral movement to mobilize passive resistance.
3. Neglect passive resistance within the upper 2.0 feet below ground surface due to potential for erosion, frost action, and other disturbance in front of walls.

Lateral surcharge pressures due to traffic, equipment, slopes, storage loads, etc., have not been included in the above lateral EFW recommendations. To estimate the lateral pressures on the reinforced wall mass from traffic, equipment, stored material, and other surcharges, use the following equation, referencing Equation 3.11.6.1-1 in the *AASHTO LRFD Specifications*:

$$\Delta_p = k_s * q_s,$$

Where:

Δ_p = Horizontal earth pressure increases due to uniform surcharge pressure (pounds per square foot, psf)
k_s = Lateral earth pressure coefficient, estimated at 0.5 for surface surcharges
q_s = Uniform surface surcharge (psf)

230.05.04 Anchors

Anchors are not required for this project.

230.05.05 Drainage

Reinforced wall backfill must comprise *Rock Cap* conforming to Section 703.08 in the latest *ITD Standards*, compacted to Class A requirements. This open graded granular material will aid in providing drainage throughout the wall backfill and reducing the buildup of hydrostatic pressure.

Retaining walls shall be constructed with adequate drainage systems to reduce the potential for water ponding within the wall backfill or eroding soil behind, beneath or in front of planned walls. Install wall drains backfill as shown on Plate 6, *Wall Backfill Schematic*. Maintaining uniformly drained conditions is critical to long-term building performance and to help route water away footings to reduce settlement risk. We recommend installing foundation drains at the lowest bearing elevation to achieve gravity draining behind the wall around the structure's perimeter.

Where wire-faced retaining walls are planned, surface drainage features must prevent salt-laden water or de-icing chemicals from either infiltrating into the wall reinforced zones and/or sheet flowing over the wall faces. Such features should include raised curbs and drain inlets to collect and direct surface runoff away from all retaining walls.

230.05.06 Embankments and Reinforced Soil Slopes

Wire-faced retaining wall design and construction shall reference the AASHTO LRFD Bridge Design Specifications, Section 11, in conjunction with the requirements of FHWA Publication No. FHWA-NHI-10-024; *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*. The following sections delineate additional geotechnical information to aid the contractor's design for planned retaining walls.

230.05.06.01 Soil Engineering Parameters

We provide the following soil engineering parameter estimates based on our field and laboratory testing specific to the planned alignment, correlations to soil type, and our experience with various aggregate products commonly used for construction. In Table 5, we provide estimated unfactored soil-engineering parameter ranges for the anticipated foundation soil and retained soil, as well as the wall backfill material. We strongly recommend that appropriate engineering judgment be applied specific to the design application for each parameter. Independent use of these without consulting GPI regarding their application is done so at the user's own risk.

Table 5. Soil Engineering Parameters

Engineering Soil Parameters	Estimated Parameter Range		
	Foundation Soil & Retained Soil: (Loess, Silt with Sand)	Backfill Outside Reinforced Zone: (Granular Subbase)	Reinforced Wall Backfill: (Rock Cap)
In-situ Moist Unit Weight	γ_m = 110-120 pcf	N/A	N/A
In-situ Moisture Range	ω = 2-10 percent	N/A	N/A
Organic Content	O.C. = <1 percent	O.C. = <1 percent	O.C. = <1 percent
Percent Passing #200 Screen	% fines = 30 – 60%	% fines = <15%	% fines = <5%
Maximum Dry Density (Modified Proctor)	γ_{max} = 115 – 125 pcf	γ_{max} = 135 – 145 pcf	γ_{max} = 130 – 140 pcf
Optimum Moisture Range	ω_{opt} = 10 – 15%	ω_{opt} = 5 – 10%	ω_{opt} = 3 – 5%
Internal Friction Angle	Φ = 30 – 37 degrees	Φ = 34 – 38 degrees	Φ = 36 – 40 degrees
Apparent Cohesion	c = 100 – 400 psf	c = 0 psf	c = 0 psf
Sliding Friction Coefficient	fs = 0.25-0.35	fs = 0.4-0.5	fs = 0.4-0.5

Corrosion of buried metallic elements is an electrochemical process and is dependent on many factors, including the type of metal or alloy, galvanic effects, and soil properties such as resistivity, pH, and oxygen content. Generally, soil that has low resistivity and low pH is more corrosive than soil with high resistivity and high pH. Laboratory chemical test results are presented in Appendix B and summarized below:

- pH: 8.2 - 8.7
- Resistivity: 1,690 - 2,860 ohm-cm
- Sulfates: 9.0-72.9 ppm

Resistivity and pH can be related to corrosion potential as presented in Table 6, which is adapted from the FHWA *Publication No. FHWA-NHI-09-087 November 2009 – Corrosion/Degradation of Soil Reinforcements for Mechanical Stabilized Earth Walls and Reinforced Soil Slopes,* referenced in the AASHTO LRFD Specifications.

Table 6. Effect of Resistivity on Corrosion (NCHRP, 1978)

Resistivity Range (ohm-cm)	Corrosion Potential
<700	Very Corrosive
700 – 2,000	Corrosive
2,000 – 5,000	Moderately Corrosive
5,000 – 10,000	Slightly Corrosive
>10,000	Non-corrosive

Based on this reference and our laboratory test results, our opinion is the soil conditions at the site present a corrosive to moderately corrosive environment for buried metallic elements, such as wall reinforcement or utility pipes. We recommend the selected contractor consider the risk of corrosion to buried structures in their wall design and construction material selection.

Soluble sulfates in soil, when at high enough levels, can cause decomposition of the microstructure of cement, precluding normal chemical reactions in the cement compound and reducing strength. Typically, soil with soluble sulfate levels less than 3,000 ppm are considered low risk. The total sulfate content of the on-site soil is very low indicating that it is not likely to cause degradation of regular Type I/II or 1L cement.

230.05.06.02 Wall Design Factors

The following recommended LRFD factors for contractor-supplied wall design are based primarily on the FHWA Publication No. FHWA-NHI-10-024; *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*. The material basis for our recommended LRFD factors is that wall backfill will comprise *Aggregate for Open Graded Base – Class 1 (Rock Cap)* per ITD Standards Section 703.08 (γ= 140 pcf, ϕ = 38°). Tables 7 and 8 summarize our recommended LRFD wall design factors.

Table 7. Retaining Wall Design Factors for Permanent Loads (P)

Design Aspect	Load Factor	Load Factor	
		Min	Max
EH (Horizontal earth pressure; active)	$\gamma_{P\text{-}EH}$	0.90	1.50
EV (Vertical earth pressure)			
-Retaining walls and abutments	$\gamma_{P\text{-}EV}$	1.00	1.35
-Overall stability	$\gamma_{P\text{-}EV}$	-	1.00
ES (Earth surcharge; dead load)	$\gamma_{P\text{-}ES}$	0.75	1.50
LS (Live-load surcharge; equiv. traffic load)	$\gamma_{P\text{-}LS}$	1.00	1.75
DC (Components and attachments)	$\gamma_{P\text{-}DC}$	0.90	1.25
EQ (Earthquake loading)	$\gamma_{P\text{-}EQ}$	-	1.00
For external stability analysis, use the following:			
Sliding and eccentricity: $\gamma_{P\text{-}EH}$ = 1.5	$\gamma_{P\text{-}EV}$ = 1.0	$\gamma_{P\text{-}ES}$ = 1.5	$\gamma_{P\text{-}LS}$ = 1.75 $\gamma_{P\text{-}EQ}$ = 1.0
Bearing capacity:	$\gamma_{P\text{-}EV}$ = 1.35	$\gamma_{P\text{-}ES}$ = 1.5	$\gamma_{P\text{-}LS}$ = 1.75
For internal stability analysis, use the following:			
$\gamma_{P\text{-}EV}$ = 1.35	$\gamma_{P\text{-}ES}$ = 1.5	$\gamma_{P\text{-}LS}$ = 1.75	$\gamma_{P\text{-}EQ}$ = 1.0

Table 8. Retaining Wall Design Resistance Factors (2)

Design Aspect	Resistance Factor	Resistance Factor	
		Static	Combined Static/Seismic
Reinforcement tensile strength and connection strength:			
Geogrid/geotextile	ϕ_{TC}	0.90	1.20
Metal mats/grids	ϕ_{TC}	0.65	0.85
Metal strips	ϕ_{TC}	0.75	1.00
Reinforcement pullout strength	ϕ_{PS}	0.90	1.20
Shear resistance along interfaces:			
Sliding along reinforced soil and foundation soil	$\phi\tau$	1.0	1.0
Sliding along reinforced soil and reinforcement	$\phi\tau$	1.0	1.0
Bearing capacity of shallow foundation	ϕ_f	0.65	0.90

230.05.06.03 Retaining Wall Global Stability Evaluation

Retaining wall design is to be supplied by the selected contractor with respect to internal stability (e.g., bearing capacity, sliding, and overturning resistance) and overall (global) stability. However, to aid project planning and wall feasibility evaluation, GPI accomplished preliminary global stability analysis on the planned retaining wall configurations provided by Keller. Our analysis relied upon material properties outlined in this report, and preliminary assumptions regarding wall type and reinforcement length, including:

- Walls will comprise welded wire facing with geogrid reinforced backfill.
- Reinforced wall backfill comprising *Rock Cap* compacted to Class A requirements.
- Geogrid reinforcement spaced at regular vertical intervals not more than 2 feet vertically, and extending horizontally at least 70 percent of the wall height behind the wall's facing, per AASHTO Section 11.10.2.
- 250 psf surface traffic surcharge load on pavement surface above walls and reinforced backfill.
- Final wall design will be provided by the contractor to establish internal and global stability referencing applicable AASHTO and FHWA design criteria.

We used the computer software SLOPE/W, developed by GEO-STUDIO, Inc., to estimate global stability safety factors for the planned retaining walls. Referencing AASHTO Section 11.6.2.3, we used the Service 1 Load Combination (e.g., ϕ = 1.0) applied to anticipated loads, and a 0.65 resistance factor (e.g., ϕ = 0.65) applied to resisting forces in our global stability analysis. We also performed a pseudo-static analysis to evaluate seismic acceleration impacts on wall stability by increasing resistance factors to 1.0 and adding a horizontal acceleration coefficient equal to 50 percent of the PGA (i.e., PGA = 0.087g). The resulting capacity to demand ratio (CDR) in each analysis is 1.0 or greater. This method is analogous to providing a 1.5 factor of safety (FOS) in traditional allowable stress design (ASD) methodology.

SLOPE/W searches for the potential failure circle with the lowest CDR by calculating CDRs on many different potential failure planes using various slope stability analysis methods. Our analyses focused on the tallest anticipated wall sections outlined by Keller and do not comprise a comprehensive stability analysis or design for each wall section. These design aspects must also be accomplished by the contractor referencing their selected wall system, means, methods, and manufacturer recommendations. The approximate section we analyzed is located at Station 28+25, on the east side of Gun Club Road, as this appears to be the tallest wall

section along the planned alignment. Our preliminary analyses provide predicted CDR against global slope failure at this section corresponding to deep-seated circular slope failures. Based on our analysis and the assumptions outlined herein, we anticipate planned wall configurations established by Keller will maintain CDR above 1.0, provided the contractor's design adheres to applicable AASHTO and FHWA design criteria. Our stability analysis results are illustrated in Appendix C.

230.05.06.04 Soil Slope Stability

From reviewing plan drawings provided by Keller, we do not anticipate embankment fill slopes greater than 2 vertical feet above existing grades will be constructed as part of this project outside of the planned retaining walls. We recommend slopes and embankments be designed no steeper than 2H:1V. Our opinion is slopes constructed steeper than 2H:1V will be susceptible to seasonal sloughing, erosion, and more significant instability risks during wet periods of the year.

Embankment fill placed over existing surfaces that are steeper than 5H:1V must be keyed into the existing site surface as illustrated on Plate 7, *Slope Construction Schematic*. If adequate keying into existing surfaces is not accomplished, differential settlement of the embankments can be expected.

230.05.07 Erosion Control

Areas disturbed during construction must be revegetated and protected to prevent surface erosion. Implement and maintain erosion control measures until adequate vegetative cover can be established after construction is complete.

We do not anticipate slopes will receive irrigation; however, where water is present at the slope surface due to precipitation and storm runoff, sloughing should be expected. If adequate vegetative cover cannot be established immediately after construction is complete, we recommend aggressive erosion control measures be implemented to reduce excessive erosion and violation of local, state, and federal codes. These measures can be, in part, accomplished by following an Idaho Department of Environmental Quality (IDEQ) approved Stormwater Pollution Prevention Plan (SWPPP). We strongly recommend a vegetation and maintenance program be initiated immediately after construction to reduce the potential for erosion and surface sloughing. In addition, a maintenance program allows County staff to visually observe the slope conditions at different times of the year.

County maintenance personnel must be notified that irrigation, snow removal or other water sources must not direct runoff or subsurface water to slopes. Surcharges from stockpiled snow, equipment, materials, or other items must not be allowed to access slope faces.

230.05.08 Seismic Design

We recommend a seismic site class "D" be used in design for this project. The following seismic parameters in Table 9 are recommended for design, referencing Section 630 in the 202 ITD Materials Manual.

Table 9. Seismic Response Criteria

Period (seconds)	Standard Acceleration Coefficients[1] for Site Class B (g)	Site Factor[2] for Site Class D	Design Spectral Acceleration Parameters for Site Class D (g)
0.0 (Peak)	PGA = 0.09	F_{PGA} = 1.6	A_s = 0.14
0.2 (Short)	S_S = 0.222	F_a = 1.6	$S_{DS} = S_S * F_a$ = 0.36
1.0	S_1 = 0.067	F_v = 2.4	$S_{D1} = S_1 * F_v$ = 0.16

1. Acceleration coefficients based on Figures 630.04.01.1, 630.04.01.2, & 630.04.01.3 in the *ITD Materials Manual*.
2. Site factors based on Tables 630.04.02.1 and 630.04.02.2 in the *ITD Materials Manual*.

Liquefaction is a common concern in low-density sand and non-plastic silt with a potential for saturated conditions. The liquefaction potential rapidly decreases when the soil density increases and the percentage of cohesive fine-grained soil increases, and where saturated conditions are not present. The subsurface conditions above bedrock are predominantly silt with varying sand and gravel content. Further, consistent, static groundwater is not anticipated above the bedrock surface. Therefore, in our opinion, the potential risk for liquefaction to impact the planned improvements is low.

Based on Figure 630.05.01.1 in the *ITD Materials Manual*, no active fault exists in Idaho within 100 miles of the project site. Therefore, our opinion is that the risk of impacts due to fault rupture at the project site are low.

230.05.09 Construction

The following sections provide geotechnical recommendations to aid construction for the planned improvements.

230.05.09.01 Soil Excavation Characteristics

We anticipate site soil above bedrock may be excavated using conventional excavation techniques. Carefully plan and implement temporary excavations to be sloped, shored, or braced in accordance with the *Occupational Safety and Health Act* (OSHA) regulations and local code. The soil conditions atop bedrock we observed are classified as "C" soil according to the OSHA criteria, provided they remain dry. Class C soil must not be sloped steeper than 1.5H:1V. Flatter excavations may be required in areas due to the variable conditions exposed during construction. The contractor is ultimately responsible for excavation stability and site safety. Soil and groundwater conditions can vary and are impacted by contractor means and methods. Isolated, local flattening of slopes or other measures may be required due to surficial sloughing and erosion activity at the site.

Construction vibrations can cause excavations to slough or cave. We do not recommend stockpiling materials adjacent to or within 10 feet of excavations, which may cause a surcharge and contribute to excavation instability. Ultimately, the contractor is solely responsible for site safety and excavation configurations, factoring in water infiltration, construction access, adjacent loading, and other factors that contribute to excavation stability.

Plan excavations with water collection points and utilize conventional sumps and pumps to remove nuisance water from runoff, seeps, springs, or precipitation. If site soil excavations are not immediately backfilled, they may degrade when exposed to runoff and require over-excavation and replacement with granular fill. We recommend construction activities and excavation backfilling be performed as rapidly as possible following

excavation to reduce the potential for subgrades to degrade under construction traffic. Further, installing wall drainage systems proactively can facilitate drainage during construction and reduce over-excavation.

230.05.09.02 Bedrock Excavation

Exploration encountered basalt bedrock at depths ranging from 7.5 to 12.0 feet below the existing ground surface. Our exploration equipment was able to penetrate the bedrock surface 0.5 to 2.5 feet prior to being refused due to rock density and hardness. Competent basalt bedrock is typically classified as "Stable Rock" referencing OSHA guidelines and can be cut near vertical for temporary applications.

In our opinion, the upper 1 to 2 feet of the bedrock surface is expected to be excavatable using large conventional rock excavation techniques and equipment. Excavating bedrock beyond 1 to 2 feet will be difficult and will require large and possibly specialized excavating equipment. We expect excavation equipment will need to be equipped with ripper shanks, rock teeth, and/or hydraulic breakers. We do not anticipate blasting will be allowed for bedrock removal on this project.

The following considerations can be incorporated into the project specifications or used as a general guide to facilitate bedrock excavation requirements. Earthwork and general contractors should consider the following with respect to accomplishing rock excavation.

1. Bedrock excavation shall be performed with late-model excavation equipment; configurations equipped with short-tip-radius rock buckets, rated at not less than 150 hp net flywheel power with a bucket-curling force of no less than 35,000 lbf and stick-crowd force of not less than 23,000 lbf.

2. A minimum 9,500 ft-lb hydraulic breaker is expected to breakout competent bedrock in confined spaces when not fractured or weathered. The contractor shall maintain contingencies for mobilizing such equipment.

3. Other rock excavation methods, including predrilling, chemical agents, and others, may be considered to achieve the required excavation depths. However, potential impacts of each method must be carefully evaluated by the County, design team, and any prospective contractor.

4. For reuse as *Granular Subbase*, bedrock excavation must reduce the excavated material to a maximum 0.33-foot particle size.

As part of any bedrock excavation process, it is critical that the overall bedrock removal limits be clearly outlined to contractors such that appropriate methods are utilized to prevent fracturing at or below subgrade elevations and that rock removal extends laterally to the prescribed limits.

230.05.09.02 Uncontrolled Fill Removal

Our exploration encountered uncontrolled fill in various locations, extending to varying depths along the planned alignment. In general, it appears the existing Gun Club Road alignment was created primarily by cutting soil and bedrock from the western side of the alignment and placing it as fill on the eastern side to create a level roadbed. No documentation exists regarding this fill's placement or compaction. The fill placed contained various debris, including deleterious material such as wood and organics. Our opinion is this uncontrolled fill is not suitable to support the planned retaining walls. We recommend all uncontrolled fill be removed from beneath all planned walls and reinforced backfill along the planned alignment. Uncontrolled fill removal must expose native soil or bedrock beneath the planned walls, reinforced soil mass, and any

associated levelling course. Based on our current understanding of the planned wall alignments and configurations, we anticipate much of the uncontrolled fill will be removed as part of excavation to achieve wall subgrade and keying for backfill placement. However, some additional overexcavation will be required in locations to remove deeper uncontrolled fill deposits.

Given reinforced wall backfill and backfill behind the reinforced soil mass are specified to comprise *Rock Cap* and *Granular Subbase*, respectively, we do not anticipate and do not recommend over-excavated fill soil will be reused as structural fill for this project. Therefore, over-excavated material must be removed from the site by the contractor and disposed of in a location approved by the County and LHTAC.

Though the uncontrolled fill is not suitable to support planned retaining walls, the fill has existed beneath the Gun Club Road alignment for decades without significant settlement or instability evident at the roadway surface. Therefore, our opinion is the fill may remain in place outside of planned wall alignments and associated reinforced soil zones. However, where existing fill remains below new embankment fills, we recommend subgrades be compacted to Class A requirements prior to fill placement.

230.06 Appendices

Appendix A includes the exploratory boring logs and a summary of the USCS. Appendix B includes laboratory test results. Index laboratory test results are presented on the individual boring logs. Appendix C includes our slope stability analysis results.

230.07 Foundation Investigation Plat

The *Foundation Investigation Plat* is included as Plate 5, which illustrates subsurface conditions beneath the planned wall at Station 59+50.

230.08 Current Specifications and Minimum Testing Requirements

This Materials Report is based on the 2023 Idaho Transportation Department Standard Specifications and 2024 Supplemental Specifications, the AASHTO LRFD Bridge Design Specifications, 9th Edition, and the 2020 Idaho Transportation Department Materials Manual.

230.09 Special Provisional Items

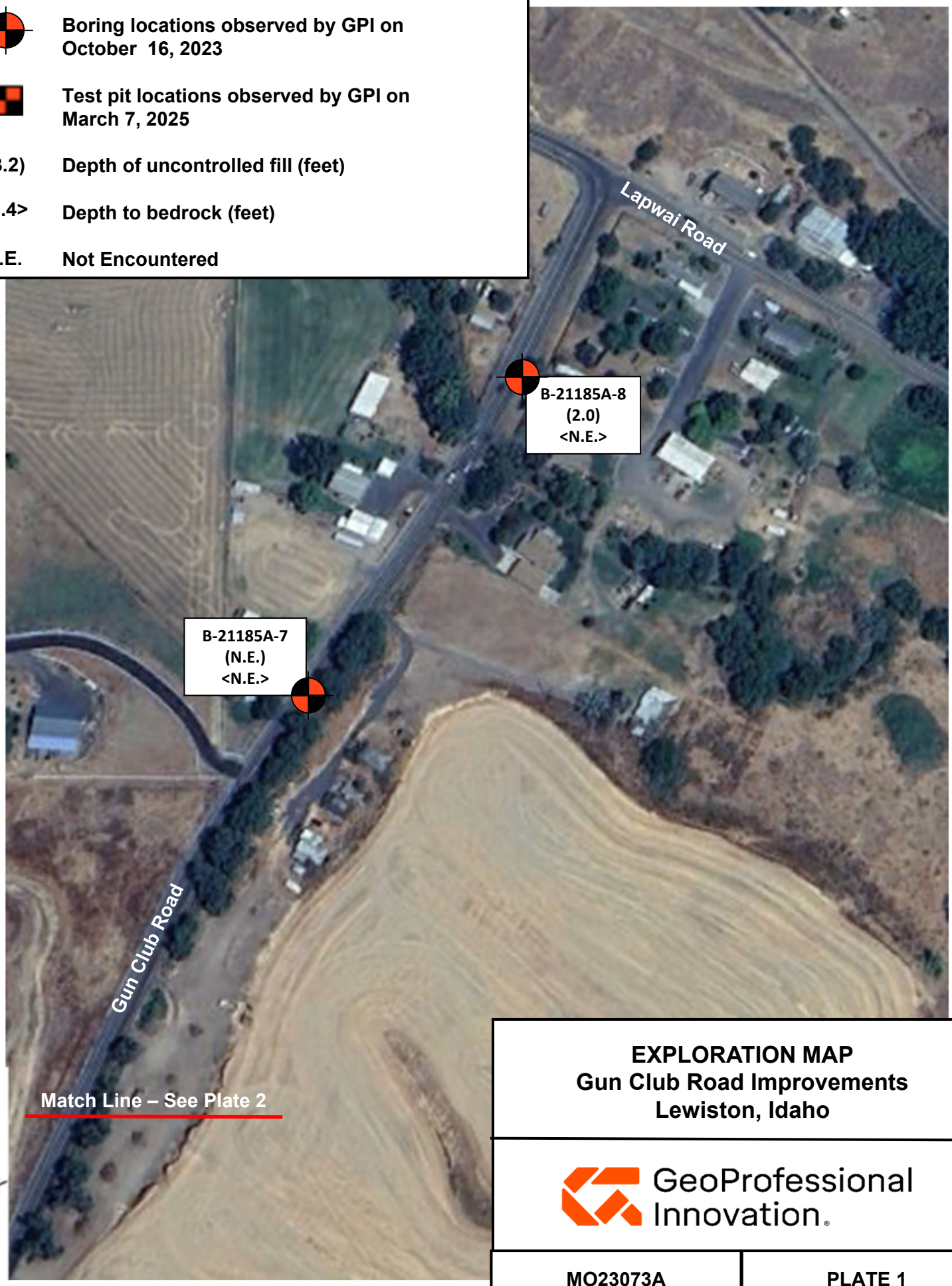
There are no special provision items required for this report.

230.10 References

- Materials Manual, October 2020, Idaho Transportation Department.
- Standard Specifications for Highway Construction, 2023 Idaho Transportation Department.
- *American Association of State Highway and Transportation Officials (AASHTO) Load Resistance Factor Design (LRFD) Bridge Design Specifications* 2024, 10th Edition.
- Federal Highway Administration (FHWA) Publication No. FHWA-NHI-10-024; *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes.*

LEGEND

- Boring locations observed by GPI on October 16, 2023
- Test pit locations observed by GPI on March 7, 2025
- (3.2) Depth of uncontrolled fill (feet)
- <7.4> Depth to bedrock (feet)
- N.E. Not Encountered



EXPLORATION MAP
Gun Club Road Improvements
Lewiston, Idaho

GeoProfessional Innovation

MO23073A	PLATE 1

Image Reference: © 2024 Google Maps. No Scale intended.

LEGEND

- Boring locations observed by GPI on October 16, 2023
- Test pit locations observed by GPI on March 7, 2025
- (3.2) Depth of uncontrolled fill (feet)
- <7.4> Depth to bedrock (feet)
- N.E. Not Encountered



EXPLORATION MAP
Gun Club Road Improvements
Lewiston, Idaho

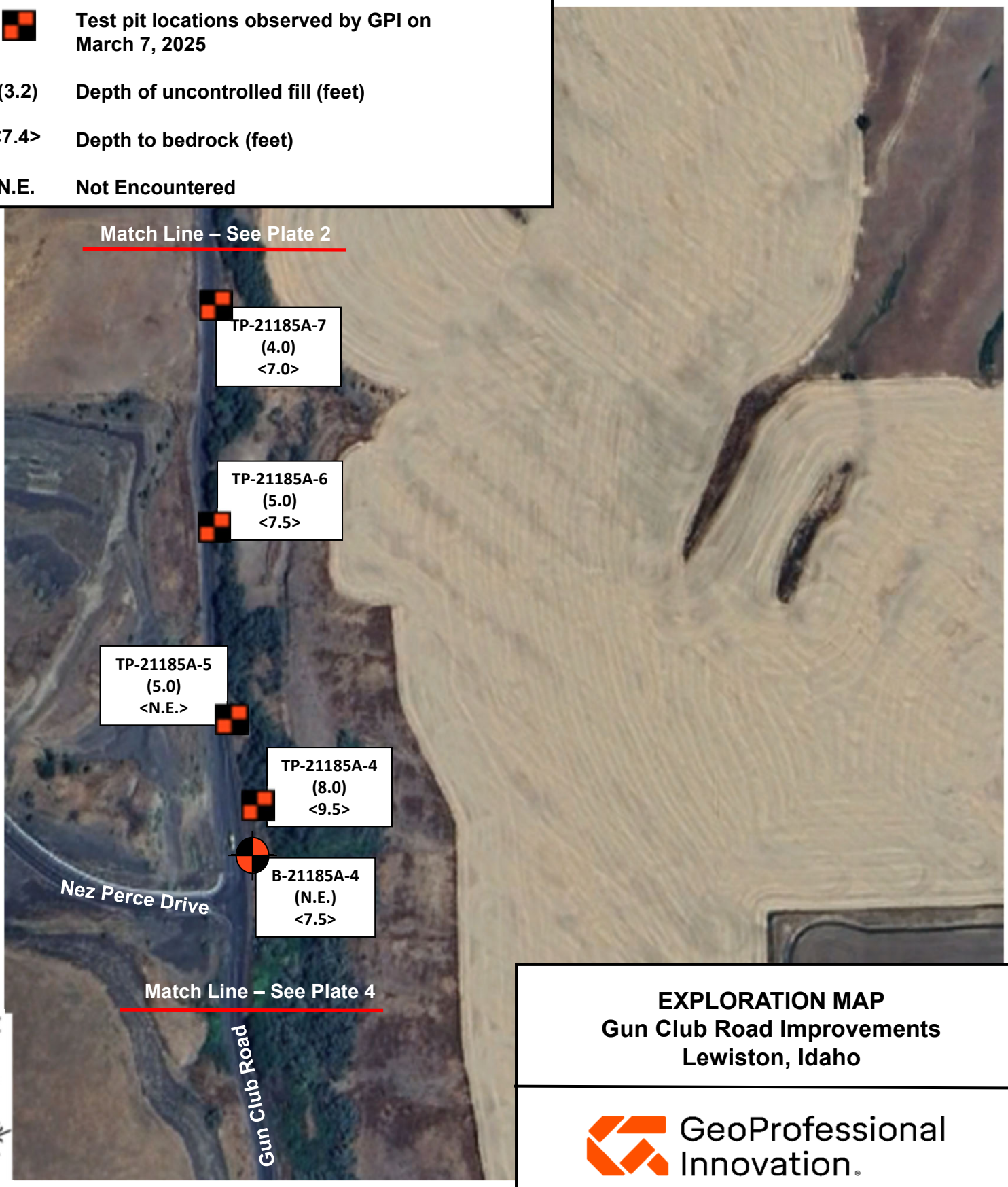
GeoProfessional Innovation

MO23073A	PLATE 2

Image Reference: © 2024 Google Maps. No Scale intended.

LEGEND

- Boring locations observed by GPI on October 16, 2023
- Test pit locations observed by GPI on March 7, 2025
- (3.2) Depth of uncontrolled fill (feet)
- <7.4> Depth to bedrock (feet)
- N.E. Not Encountered



EXPLORATION MAP
Gun Club Road Improvements
Lewiston, Idaho

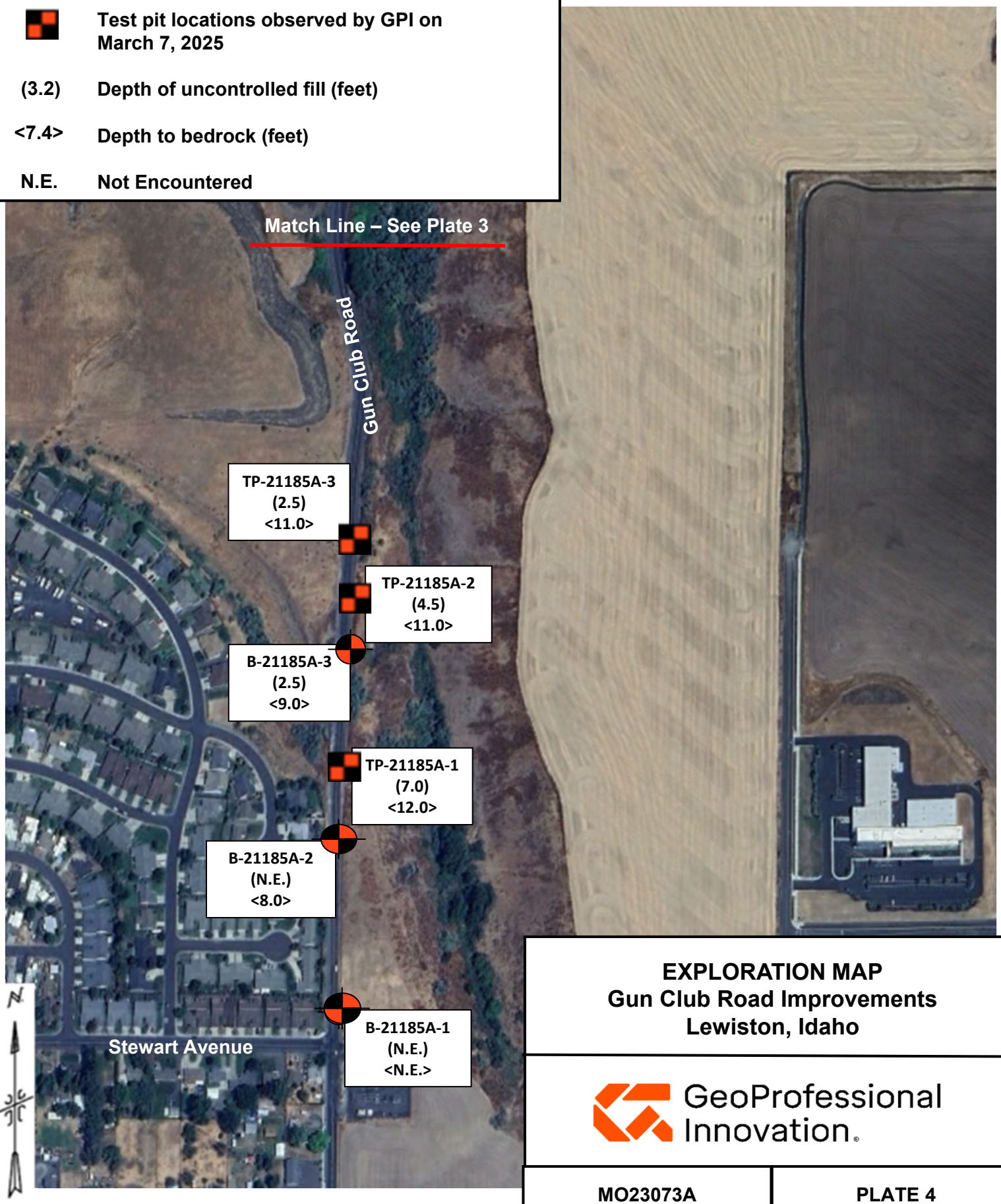
GeoProfessional Innovation

MO23073A	PLATE 3

Image Reference: © 2024 Google Maps. No Scale intended.

LEGEND

- Boring locations observed by GPI on October 16, 2023
- Test pit locations observed by GPI on March 7, 2025
- (3.2) Depth of uncontrolled fill (feet)
- <7.4> Depth to bedrock (feet)
- N.E. Not Encountered

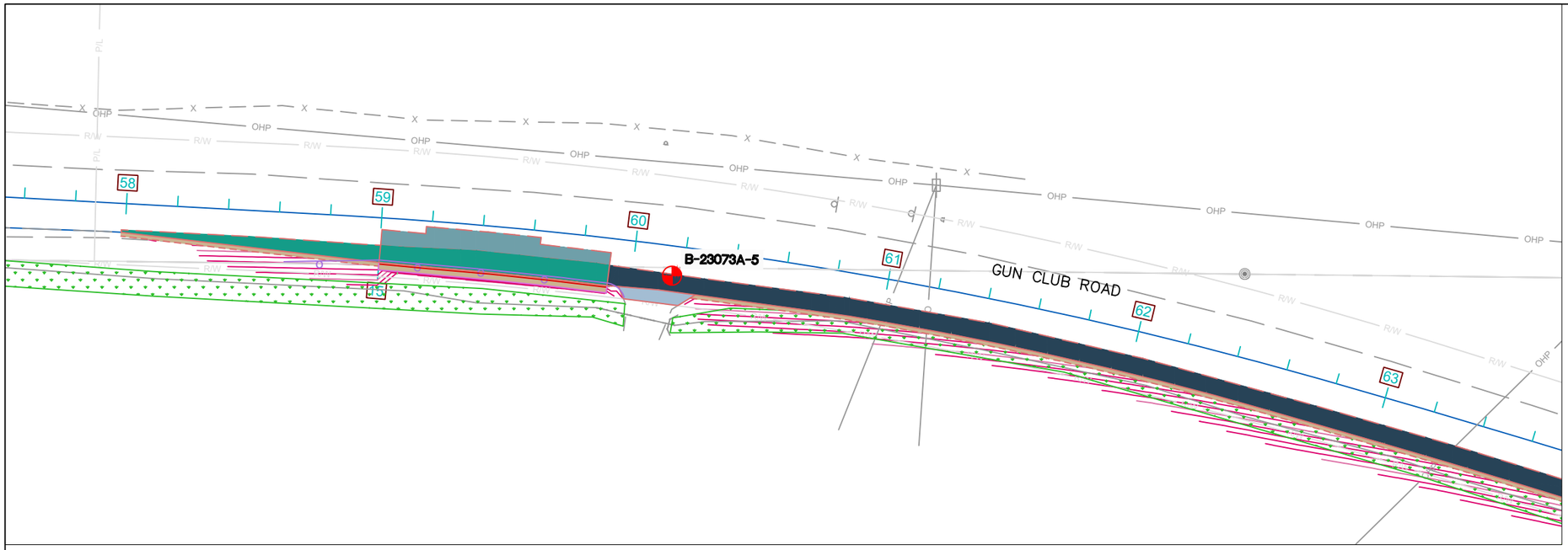


EXPLORATION MAP
Gun Club Road Improvements
Lewiston, Idaho

GeoProfessional Innovation

MO23073A	PLATE 4

Image Reference: © 2024 Google Maps. No Scale intended.

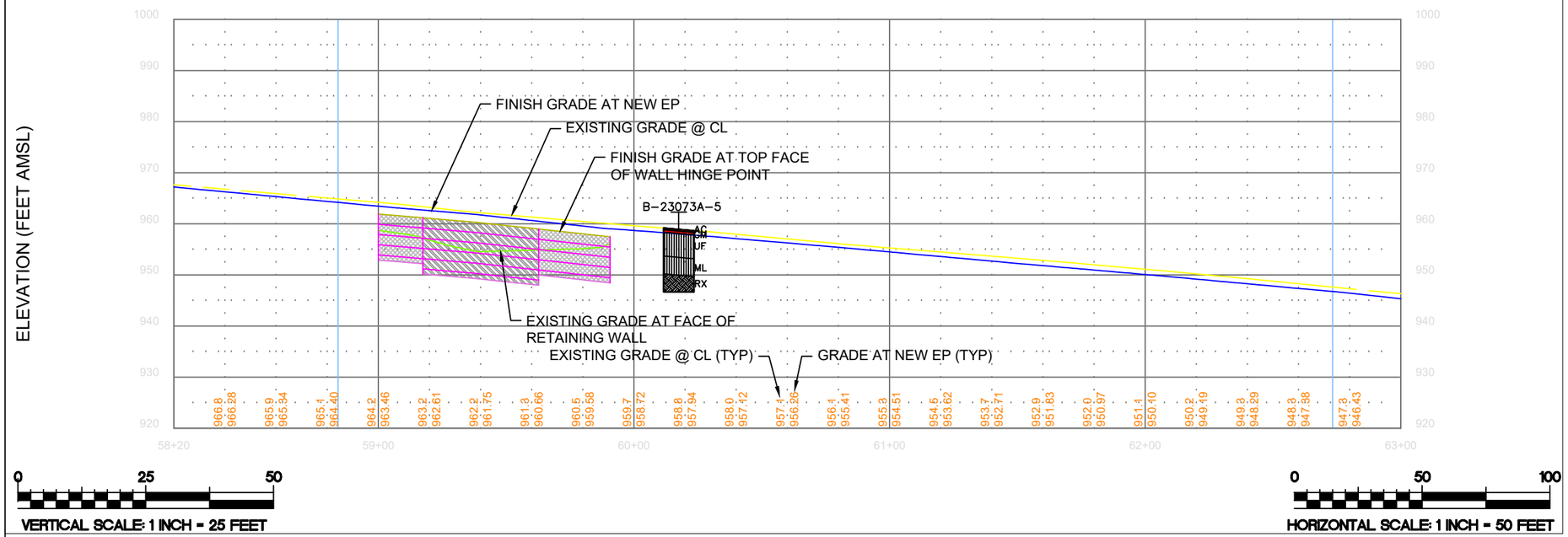


0 50 100

PLAN SCALE: 1 INCH = 50 FEET

PLAN

APPROXIMATE LOCATION OF EXPLORATORY BORING OBSERVED BY GPI ON OCTOBER 16, 2023



0 25 50

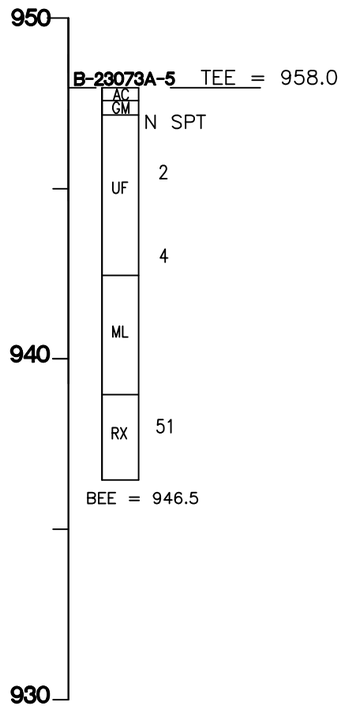
VERTICAL SCALE: 1 INCH = 25 FEET

0 50 100

HORIZONTAL SCALE: 1 INCH = 50 FEET

PROFILE STATION (FEET)

BORING LOG



0 5 10

VERTICAL SCALE: 1 INCH = 5 FEET

LEGEND

TEE– TOP OF EXPLORATION ELEVATION (Approximate)

BEE– BASE OF EXPLORATION ELEVATION (Approximate)

AC	ASPHALT CONCRETE	GM	BASE COURSE – SILTY GRAVEL
UF	UNCONTROLLED FILL – SILT WITH SAND	ML	LOESS – SILT WITH SAND
RX	BEDROCK – BASALT		

REVISIONS			
NO.	DATE	BY	DESCRIPTION
1	5/16/25	NGH	FINAL

DESIGNED: NGH
DESIGN CHECKED: AJA
DETAILED: NGH
DWG. CHECKED: AJA
CORRECTIONS:

SCALES SHOWN ARE FOR 17"X11" PRINTS ONLY

CAD FILE NO. M023073A

DRAWING DATE: MAY 16, 2025

ORIGINAL SIGNED BY:

ORIGINAL DATE SIGNED:

IDAHO TRANSPORTATION DEPARTMENT

GEOPROFESSIONAL INNOVATION CORPORATION

English

PROJECT NO. A023(881)

SOIL PROFILE

GUN CLUB ROAD WALL SECTION – STA 59+50

BRIDGE KEY NO.

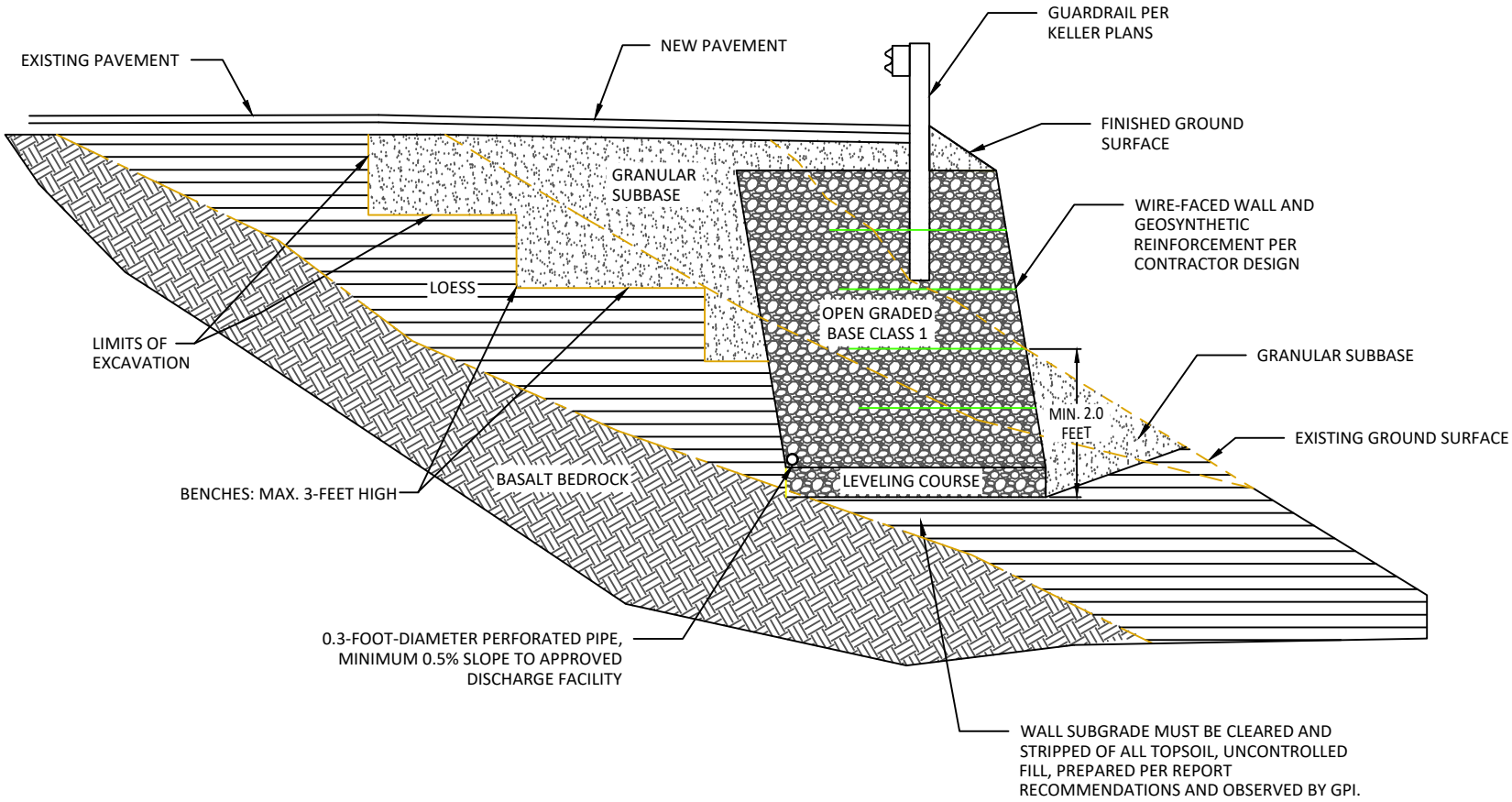
COUNTY: NEZ PERCE

KEY NO.

DRAWING NO.

SHEET: 1 OF 1

Reference: Base files were provided by Keller & Associates Inc.



NOTES:

1. EXCAVATE AND REMOVE UNCONTROLLED FILL BENEATH WALL AND BACKFILL TO EXPOSE NATIVE SOIL OR BEDROCK.
2. PREPARE SUBGRADES PER THE GEOTECHNICAL REPORT REQUIREMENTS.
3. PLACE 0.3-FOOT-DIAMETER, PERFORATED ADS OR PVC PIPE AT LOWEST POSSIBLE ELEVATION, SLOPED AT LEAST 0.5% TOWARDS DAYLIGHT OR APPROVED DISCHARGE FACILITY.
4. ACCOMPLISH KEYING INTO EXISTING SUBGRADES PER THE *EARTHWORK* REPORT SECTION.
5. COMPACT STRUCTURAL FILL PER THE GEOTECHNICAL REPORT REQUIREMENTS.

INFORMATION ONLY (NOT TO SCALE)

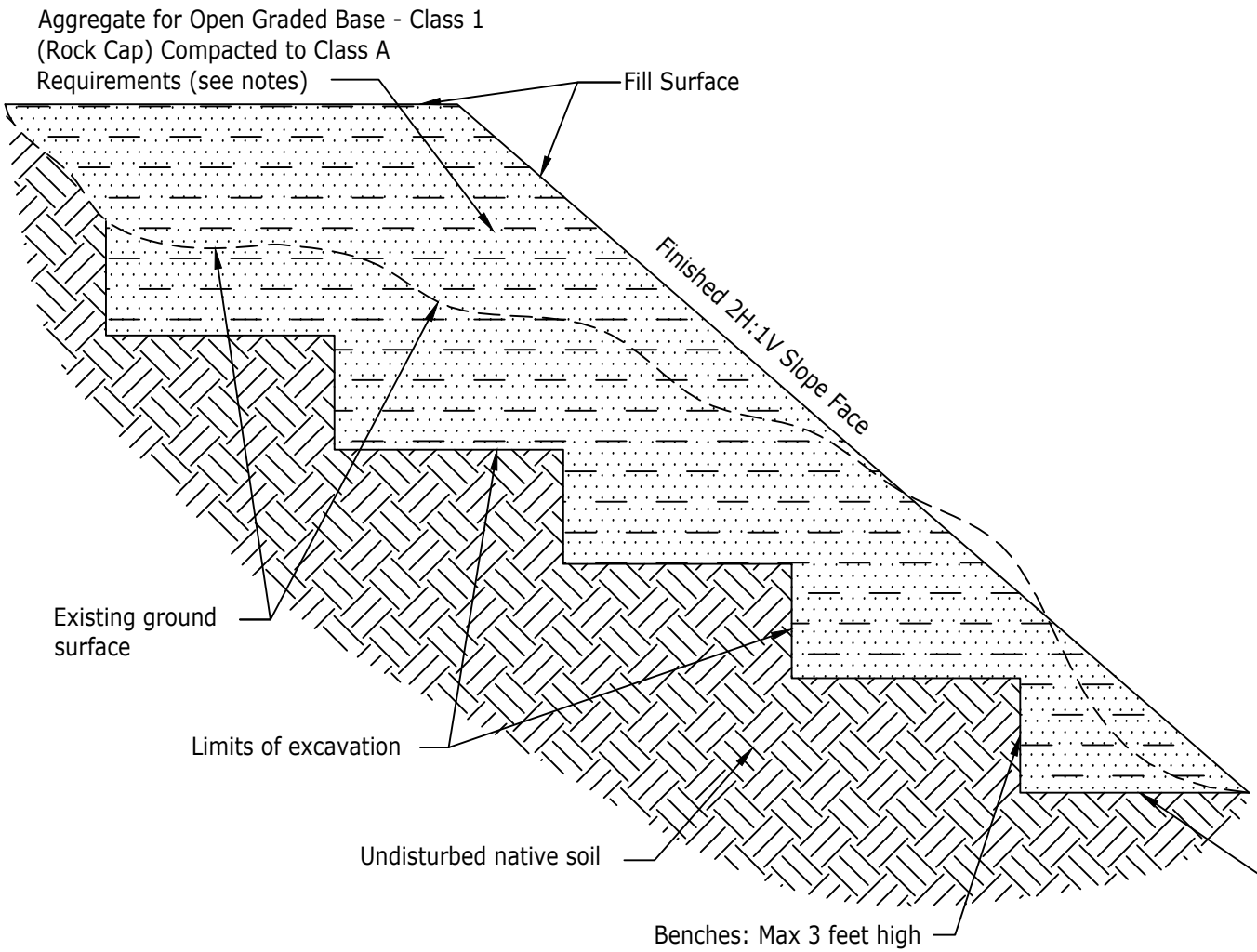
WALL CONSTRUCTION SCHEMATIC
Gun Club Road, Lapwai Road to Stewart Avenue
Nez Perce County
Lewiston, Idaho

GeoProfessional Innovation®

NOT TO SCALE

MO23073A	PLATE: 6
DRAWN BY: NGH	CHECKED BY: AJA

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NOTES:

1. Compact subgrade to Class A requirements
2. Rock cap to be placed in 1-foot-thick, loose lifts and compacted to Class A requirements.
3. Rock cap shall consist of approved material conforming to section 703.08 of the last edition of the ITD Standard Specifications for Highway Construction.

INFORMATIONAL ONLY

SLOPE CONSTRUCTION SCHEMATIC
Gun Club Road, Lapwai Road to Stewart Avenue
Nez Perce County
Lewiston, Idaho

NOT TO SCALE

GeoProfessional Innovation

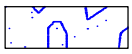
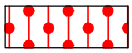
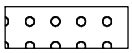
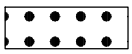
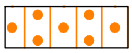
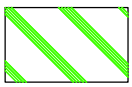
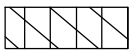
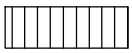
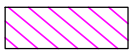
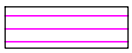
MO23073A	PLATE: 7
DRAWN BY: NGH	CHECKED BY: AJA

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APPENDIX A

Unified Soil Classification System (USCS) and Exploration Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE GRAINED SOIL	GRAVEL	CLEAN GRAVEL		GW	WELL-GRADED GRAVEL, GRAVEL-SAND MIXTURES.
				GP	POORLY-GRADED GRAVEL, GRAVEL-SAND MIXTURES.
		GRAVEL WITH FINES		GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURES.
				GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURES.
	SAND	CLEAN SAND		SW	WELL-GRADED SAND, GRAVELLY SAND.
				SP	POORLY-GRADED SAND, GRAVELLY SAND.
		SAND WITH FINES		SM	SILTY SAND, SAND-SILT MIXTURES.
				SC	CLAYEY SAND, SAND-CLAY MIXTURES.
FINE GRAINED SOIL	SILT AND CLAY LIQUID LIMIT LESS THAN 50%			ML	INORGANIC SILT, SANDY OR CLAYEY SILT.
				CL	INORGANIC CLAY OF LOW TO MEDIUM PLASTICITY, SANDY OR SILTY CLAY.
				CL-ML	INORGANIC MIXED CLAY AND SILT.
				OL	ORGANIC SILT AND CLAY OF LOW PLASTICITY.
	SILT AND CLAY LIQUID LIMIT GREATER THAN 50%			MH	INORGANIC SILT, MICA- CEOUS SILT, PLASTIC SILT.
				CH	INORGANIC CLAY OF HIGH PLASTICITY, FAT CLAY.
				OH	ORGANIC CLAY OF MEDIUM TO HIGH PLASTICITY.
				PT	PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS.

BORING LOG SYMBOLS

STANDARD 2 INCH OD SPLIT SPOON SAMPLE



CALIFORNIA MODIFIED 3 INCH OD SPLIT SPOON SAMPLE



ROCK CORE



SHELBY TUBE 3 INCH OD UNDISTURBED SAMPLE



TEST PIT LOG SYMBOLS

GRAB BAG SAMPLE

BG

BULK SAMPLE

BK

RING SAMPLE

RG

GROUNDWATER SYMBOLS

GROUNDWATER AFTER 24 HOURS



GROUNDWATER AT TIME OF EXPLORATION



GROUNDWATER AT THE END OF EXPLORATION



GeoProfessional Innovation®

USCS Description	Depth (ft)	Symbol	Sample Type	SPT Blows Per 6 Inches	SPT N	Dry Density (pcf)	TEST RESULTS: Pocket Penetrometer, TSF ▲ (0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5); SPT, N-Value ●; % Passing No. 200 Sieve ★; PL MC LL (20 40 60 80)	Remarks Note: BGS = Below Ground Surface
ASPHALT CONCRETE - 5.0" THICKNESS, black (SP) BASE COURSE - POORLY GRADED SAND WITH GRAVEL, gray, medium dense, moist (ML) LOESS - SILT WITH SAND, brown, firm to stiff, moist	0 5	SP ML	BK BG BK	1 3 4 4 5 5	7 10		3.3 13.2	
Borehole Terminated at 6.5 Feet.								Borehole plugged with bentonite and patched with cold mix asphalt

Client: Keller Associates	**Boring Number:** B-23073A-1	GeoProfessional Innovation.	**EXPLORATORY BORING LOG**
Project: MO23073A	**Date Drilled:** 10-16-2023		
Drill Rig: G2400	**Borehole Diameter:** 8" O.D.		
Depth to Groundwater: N.E.	**Logged By:** AJA		Sheet 1 Of 1

GPI BORING - GPI.GDT - 5/16/25 08:33 - V:\ENGINEERING\GINT FILES\2023\UNSORTED\23073A GUN CLUB ROAD.GPJ

USCS Description	Depth (ft)	Symbol	Sample Type	SPT Blows Per 6 Inches	SPT N	Dry Density (pcf)	TEST RESULTS	Remarks
							Pocket Penetrometer, TSF ▲ 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 SPT, N-Value ● % Passing No. 200 Sieve ★ PL MC LL 20 40 60 80	Note: BGS = Below Ground Surface
ASPHALT CONCRETE - 2.5" THICKNESS, black (SP) BASE COURSE - POORLY GRADED SAND WITH GRAVEL, gray, medium dense, moist (ML) LOESS - SILT WITH SAND, brown, firm to stiff, moist	0	SP						
		ML		6 9 8	17			
	5			1 2 7	9			
(RX) BEDROCK - BASALT, dark gray with orange staining, hard, Moderately Weathered and Moderately Fractured		RX						Drill chatter from 8.0 to 10.0 feet BGS
	10			50/4.0"	50+		50+	
Borehole Terminated at 10.4 Feet.								Borehole terminated due to practical refusal on dense bedrock. Borehole plugged with bentonite and patched with cold mix asphalt

Client: Keller Associates	**Boring Number:** B-23073A-2
Project: MO23073A	**Date Drilled:** 10-16-2023
Drill Rig: G2400	**Borehole Diameter:** 8" O.D.
Depth to Groundwater: N.E.	**Logged By:** AJA

GeoProfessional Innovation.

EXPLORATORY BORING LOG

Sheet 1 Of 1

GPI BORING - GPI.GDT - 5/16/25 08:33 - V:\ENGINEERING\GINT FILES\2023\UNSORTED\23073A GUN CLUB ROAD.GPJ

USCS Description	Depth (ft)	Symbol	Sample Type	SPT Blows Per 6 Inches	SPT N	Dry Density (pcf)	TEST RESULTS: Pocket Penetrometer, TSF ▲ (0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5); SPT, N-Value ●; % Passing No. 200 Sieve ★; PL MC LL (20 40 60 80)	Remarks Note: BGS = Below Ground Surface
ASPHALT CONCRETE - 2.2" THICKNESS, black	0	SP	BK BG					
(SP) BASE COURSE - POORLY GRADED SAND WITH GRAVEL, gray, medium dense, moist		ML	BK					
(ML) UNCONTROLLED FILL - SILT WITH GRAVEL AND COBBLES, brown, medium dense, moist				1 1 1	2		11.9	
(ML) LOESS - SILT WITH SAND, brown, soft to stiff, moist	5	ML		2 3 5	8			
(RX) BEDROCK - BASALT, dark gray with orange staining, hard, Moderately Weathered and Moderately Fractured		RX		1/1.0"	50+		50+	Drill chatter from 9.0 to 9.4 feet BGS
Borehole Terminated at 9.5 Feet.								Borehole terminated due to practical refusal on dense bedrock. Borehole plugged with bentonite and patched with cold mix asphalt

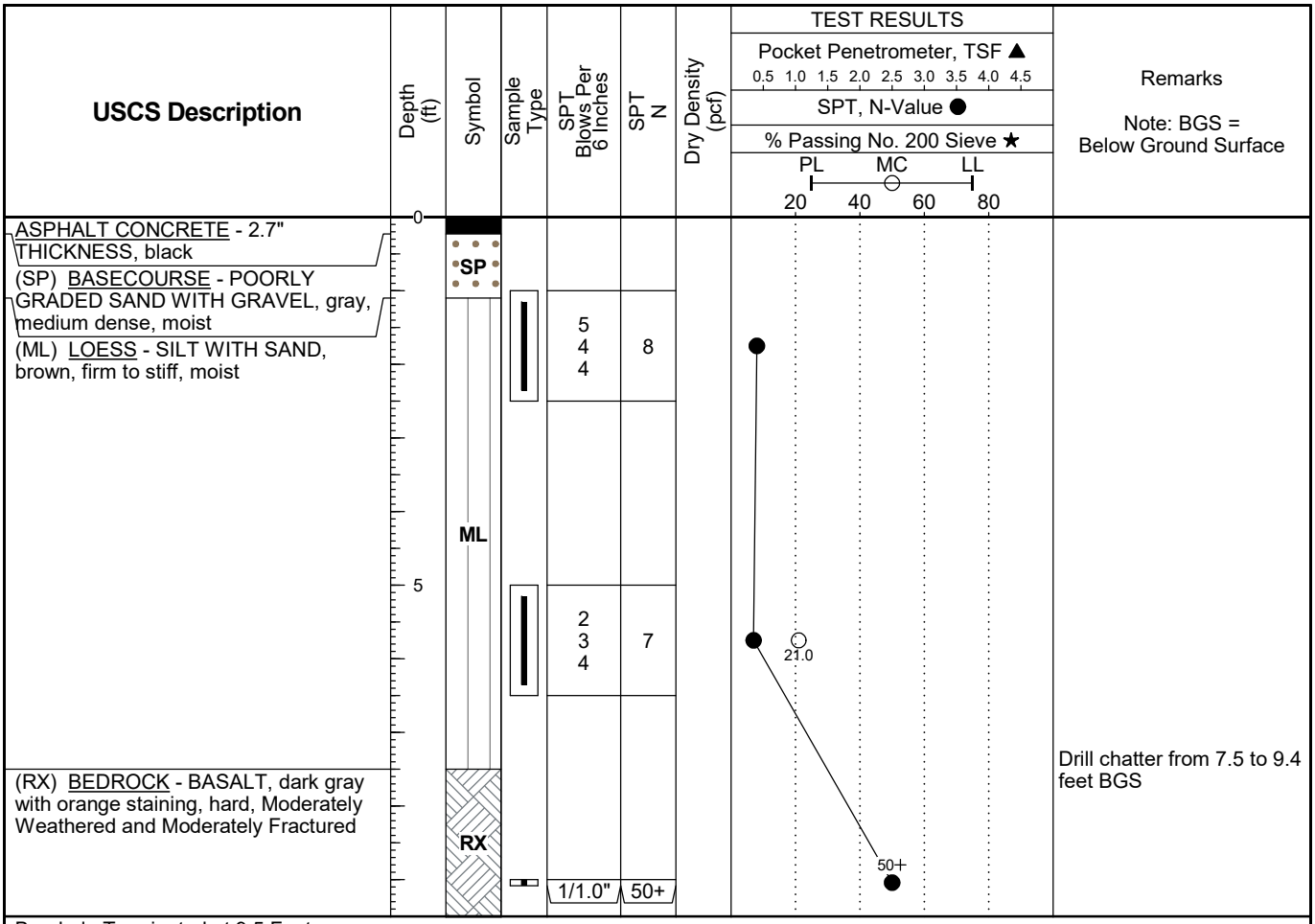
Client: Keller Associates	**Boring Number:** B-23073A-3
Project: MO23073A	**Date Drilled:** 10-16-2023
Drill Rig: G2400	**Borehole Diameter:** 8" O.D.
Depth to Groundwater: N.E.	**Logged By:** AJA

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EXPLORATORY BORING LOG

Sheet 1 Of 1

GPI BORING - GPI.GDT - 5/16/25 08:33 - V:\ENGINEERING\GINT FILES\2023\UNSORTED\23073A GUN CLUB ROAD.GPJ



GPI BORING - GPI.GDT - 5/16/25 08:33 - V:\ENGINEERING\GINT FILES\2023\UNSORTED\23073A GUN CLUB ROAD.GPJ

Client: Keller Associates	**Boring Number:** B-23073A-4
Project: MO23073A	**Date Drilled:** 10-16-2023
Drill Rig: G2400	**Borehole Diameter:** 8" O.D.
Depth to Groundwater: N.E.	**Logged By:** AJA

GeoProfessional Innovation.

EXPLORATORY BORING LOG

USCS Description	Depth (ft)	Symbol	Sample Type	SPT Blows Per 6 Inches	SPT N	Dry Density (pcf)	TEST RESULTS	Remarks
							Pocket Penetrometer, TSF ▲ 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 SPT, N-Value ● % Passing No. 200 Sieve ★ PL MC LL 20 40 60 80	Note: BGS = Below Ground Surface
ASPHALT CONCRETE - 4.5" THICKNESS, black (SP) BASE COURSE - POORLY GRADED SAND WITH GRAVEL, gray, medium dense, moist (ML) UNCONTROLLED FILL - SILT WITH SAND, brown, soft, moist	0	SP ML		4 1 1	2		○ 9.9	Trace crushed gravel in sample.
(ML) LOESS - SILT WITH SAND, brown, firm, moist	5	ML		1 2 2	4			
(RX) BEDROCK - BASALT, very dense, Highly weathered and Highly Fractured	10	RX		16 26 25	51			Drill chatter from 9.0 to 10.0 feet BGS Borehole plugged with bentonite and patched with cold mix asphalt
Borehole Terminated at 11.5 Feet.								

Client: Keller Associates	**Boring Number:** B-23073A-5	GeoProfessional Innovation.	**EXPLORATORY BORING LOG**
Project: MO23073A	**Date Drilled:** 10-16-2023		
Drill Rig: G2400	**Borehole Diameter:** 8" O.D.		
Depth to Groundwater: N.E.	**Logged By:** AJA		Sheet 1 Of 1

GPI BORING - GPI.GDT - 5/16/25 08:33 - V:\ENGINEERING\GINT FILES\2023\UNSORTED\23073A GUN CLUB ROAD.GPJ

USCS Description	Depth (ft)	Symbol	Sample Type	SPT Blows Per 6 Inches	SPT N	Dry Density (pcf)	TEST RESULTS: Pocket Penetrometer, TSF ▲ (0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5); SPT, N-Value ●; % Passing No. 200 Sieve ★; PL MC LL (20 40 60 80)	Remarks Note: BGS = Below Ground Surface
ASPHALT CONCRETE - 2.8" THICKNESS, black BST - 6.0" THICKNESS, black (GM) UNCONTROLLED FILL - SILTY GRAVEL WITH COBBLES, dark brown, loose to medium dense, moist	0							Heavy drill chatter at 2.0 to 7.5 feet BGS
	5	GM		14 13 12	25			
				4 3 4	7			
				5 10 13	23		7.3	
Borehole Terminated at 9.0 Feet.								Borehole plugged with bentonite and patched with cold mix asphalt

Client: Keller Associates	**Boring Number:** B-23073A-6
Project: MO23073A	**Date Drilled:** 10-16-2023
Drill Rig: G2400	**Borehole Diameter:** 8" O.D.
Depth to Groundwater: N.E.	**Logged By:** AJA

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EXPLORATORY BORING LOG

Sheet 1 Of 1

GPI BORING - GPI.GDT - 5/16/25 08:33 - V:\ENGINEERING\GINT FILES\2023\UNSORTED\23073A GUN CLUB ROAD.GPJ

USCS Description	Depth (ft)	Symbol	Sample Type	SPT Blows Per 6 Inches	SPT N	Dry Density (pcf)	TEST RESULTS: Pocket Penetrometer, TSF ▲ (0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5); SPT, N-Value ●; % Passing No. 200 Sieve ★; PL MC LL (20 40 60 80)	Remarks Note: BGS = Below Ground Surface
ASPHALT CONCRETE - 2.7" THICKNESS, black BST - 3.0" THICKNESS, black (SM) ALLUVIUM - SILTY SAND, brown, loose to medium dense, moist	0		BK				6.1 ○ ★	
				2 2 3	5		●	
	5	SM		5 6 6	12		●	
	10			6 11 12	23		●	
Borehole Terminated at 12.0 Feet.								Borehole plugged with bentonite and patched with cold mix asphalt

Client: Keller Associates	**Boring Number:** B-23073A-7	GeoProfessional Innovation.	**EXPLORATORY BORING LOG**
Project: MO23073A	**Date Drilled:** 10-16-2023		
Drill Rig: G2400	**Borehole Diameter:** 8" O.D.		
Depth to Groundwater: N.E.	**Logged By:** AJA		Sheet 1 Of 1

GPI BORING - GPI.GDT - 5/16/25 08:33 - V:\ENGINEERING\GINT FILES\2023\UNSORTED\23073A GUN CLUB ROAD.GPJ

USCS Description	Depth (ft)	Symbol	Sample Type	SPT Blows Per 6 Inches	SPT N	Dry Density (pcf)	TEST RESULTS	Remarks
							Pocket Penetrometer, TSF ▲ 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 SPT, N-Value ● % Passing No. 200 Sieve ★ PL MC LL 20 40 60 80	Note: BGS = Below Ground Surface
ASPHALT CONCRETE - 4.5" THICKNESS, black	0	SP						
(SP) BASE COURSE - POORLY GRADED SAND WITH GRAVEL, gray, medium dense, moist								
(SM) FILL - SILTY SAND WITH GRAVEL		SM						
(SM) ALLUVIUM - SILTY SAND, brown, loose, moist	5	SM		1 2 2	4		●	
	10			2 2 3	5		●	
Borehole Terminated at 11.5 Feet.								Borehole plugged with bentonite and patched with cold mix asphalt

Client: Keller Associates	**Boring Number:** B-23073A-8	GeoProfessional Innovation.	**EXPLORATORY BORING LOG**
Project: MO23073A	**Date Drilled:** 10-16-2023		
Drill Rig: G2400	**Borehole Diameter:** 8" O.D.		
Depth to Groundwater: N.E.	**Logged By:** AJA		Sheet 1 Of 1

GPI BORING - GPI.GDT - 5/16/25 08:33 - V:\ENGINEERING\GINT FILES\2023\UNSORTED\23073A GUN CLUB ROAD.GPJ

USCS Description	Depth (ft)	U.S.C.S. Class	Symbol	Sample Type	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	Atterberg Limits LL PI	Remarks Note: BGS = Below Ground Surface
(ML) UNCONTROLLED FILL - SILT WITH SAND GRAVEL, AND COBBLES, brown, loose, moist	0.0	ML		BK	24.0					Trace vegetation and organics at the ground surface. Debris including metal, concrete, paper, plastic, and wood encountered from 0 to 7 feet below ground surface.
	2.5									
	5.0									
(ML) LOESS - SILT WITH SAND, tan, firm, moist	7.5	ML		BG						
	10.0									
(RX) BEDROCK - BASALT, highly to moderately weathered, highly fractured, dark gray with whiteish orange staining, hard	12.5	RX								
Test Pit Terminated at 13.0 Feet.										Test pit terminated at 13 feet due to refusal on dense bedrock. Loosely backfilled upon completion.

Client: Keller Associates	**Test Pit Number:** TP-23073A-1
Project: MO23073A	**Date Excavated:** 03-07-2025
Backhoe: CASE 580	**Bucket Width:** 2.0 feet
Depth to Groundwater: N.E.	**Logged By:** AJA

GeoProfessional Innovation

EXPLORATORY TEST PIT LOG

Sheet 1 Of 1

GPI TEST PIT - GPI.GDT - 5/16/25 08:28 - V:\ENGINEERING\GINT FILES\2025\250307 MO23073A GUN CLUB ROAD TEST PITS.GPJ

USCS Description	Depth (ft)	U.S.C.S. Class	Symbol	Sample Type	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	Atterberg Limits LL PI	Remarks Note: BGS = Below Ground Surface
(ML) UNCONTROLLED FILL - SILT WITH SAND GRAVEL, AND COBBLES, brown, loose, moist	0.0	ML								Trace vegetation at ground surface. Debris including wood, plastic, and asphalt encountered from 0 to 5 feet below ground surface.
(ML) LOESS - SILT WITH SAND, tan, firm, moist	2.5 5.0	ML		RG		100.7	15.1			Direct Shear Phi = 36.5 degrees C = 325 psf
(RX) BEDROCK - BASALT, highly to moderately weathered, highly fractured, dark gray with whiteish orange staining, hard	7.5 10.0	RX								
Test Pit Terminated at 12.0 Feet.										Test pit terminated at 12 feet below ground surface upon refusal due to bedrock density. Loosely backfilled opun completion.

Client: Keller Associates	**Test Pit Number:** TP-23073A-2	GeoProfessional Innovation.	**EXPLORATORY TEST PIT LOG**
Project: MO23073A	**Date Excavated:** 03-07-2025		
Backhoe: CASE 580	**Bucket Width:** 2.0 feet		Sheet 1 Of 1
Depth to Groundwater: N.E.	**Logged By:** AJA		

GPI TEST PIT - GPI.GDT - 5/16/25 08:28 - V:\ENGINEERING\GINT FILES\2025\250307 MO23073A GUN CLUB ROAD TEST PITS.GPJ

USCS Description	Depth (ft)	U.S.C.S. Class	Symbol	Sample Type	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	Atterberg Limits LL PI	Remarks Note: BGS = Below Ground Surface
(ML) UNCONTROLLED FILL - SILT WITH SAND GRAVEL AND COBBLES, brown, loose, moist	0.0	ML								Trace vegetation and organics at ground surface.
(ML) LOESS - SILT WITH SAND, light tan, loose, moist	2.5			BK						
	5.0	ML								
	7.5									
	10.0									
(RX) BEDROCK - BASALT, highly to moderately weathered, highly fractured, whiteish with orange staining		RX								
Test Pit Terminated at 12.0 Feet.										Test pit terminated at 12 feet below ground surface upon refusal due to bedrock density. Loosely backfilled upon completion.

Client: Keller Associates	**Test Pit Number:** TP-23073A-3	GeoProfessional Innovation.	**EXPLORATORY TEST PIT LOG**
Project: MO23073A	**Date Excavated:** 03-07-2025		
Backhoe: CASE 580	**Bucket Width:** 2.0 feet		Sheet 1 Of 1
Depth to Groundwater: N.E.	**Logged By:** AJA		

GPI TEST PIT - GPI.GDT - 5/16/25 08:28 - V:\ENGINEERING\GINT FILES\2025\250307 MO23073A GUN CLUB ROAD TEST PITS.GPJ

USCS Description	Depth (ft)	U.S.C.S. Class	Symbol	Sample Type	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	Atterberg Limits LL PI	Remarks Note: BGS = Below Ground Surface
(ML) TOPSOIL - SILT, brown, loose, moist	0.0	ML								Trace vegetation and organics to 8 inches below ground surface.
(ML) UNCONTROLLED FILL - SILT WITH SAND, GRAVEL AND COBBLES, brown, loose, moist	2.5	ML								Debris including metal, plastic, and glass encountered from 1 to 6 feet below ground surface.
(GM) UNCONTROLLED FILL - SILTY GRAVEL WITH COBBLES AND BOULDERS, gray, dense, moist	5.0	GM								Boulders up to 2 feet in diameter from 6 to 8 feet below ground surface.
(ML) LOESS - SILT WITH SAND, tan, firm, moist	7.5	ML								
(RX) BASALT, highly to moderately weathered, highly fractured, dark gray whiteish orange staining, hard	10.0	RX								
Test Pit Terminated at 12.0 Feet.										Test pit terminated at 12 feet below ground surface upon refusal to due bedrock density. Loosely backfilled upon completion.

Client: Keller Associates	**Test Pit Number:** TP-23073A-4	GeoProfessional Innovation.	**EXPLORATORY TEST PIT LOG**
Project: MO23073A	**Date Excavated:** 03-07-2025		
Backhoe: CASE 580	**Bucket Width:** 2.0 feet		Sheet 1 Of 1
Depth to Groundwater: N.E.	**Logged By:** AJA		

GPI TEST PIT - GPI.GDT - 5/16/25 08:28 - V:\ENGINEERING\GINT FILES\2025\250307 MO23073A GUN CLUB ROAD TEST PITS.GPJ

USCS Description	Depth (ft)	U.S.C.S. Class	Symbol	Sample Type	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	Atterberg Limits LL PI	Remarks Note: BGS = Below Ground Surface
(ML) UNCONTROLLED FILL - SILT WITH SAND, GRAVEL AND COBBLES, brown, loose, moist	0.0 2.5	ML								Trace vegetation and organics. Debris including plastic, glass, and metal encountered from 0 to 5 feet below ground surface.
(ML) LOESS - SILT WITH SAND, brown, firm, moist	5.0 7.5 10.0 12.5	ML		BG	52.0					
Test Pit Terminated at 14.0 Feet.										Test pit terminated at 14 feet below ground surface, loosely backfilled upon completion.

Client: Keller Associates	**Test Pit Number:** TP-23073A-5	GeoProfessional Innovation.	**EXPLORATORY TEST PIT LOG**
Project: MO23073A	**Date Excavated:** 03-07-2025		
Backhoe: CASE 580	**Bucket Width:** 2.0 feet		Sheet 1 Of 1
Depth to Groundwater: N.E.	**Logged By:** AJA		

GPI TEST PIT - GPI.GDT - 5/16/25 08:29 - V:\ENGINEERING\GINT FILES\2025\250307 MO23073A GUN CLUB ROAD TEST PITS.GPJ

USCS Description	Depth (ft)	U.S.C.S. Class	Symbol	Sample Type	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	Atterberg Limits LL PI	Remarks Note: BGS = Below Ground Surface
(ML) TOPSOIL - SILT, brown, loose, moist	0.0	ML								Trace vegetation and organics from 0 to 1 foot below ground surface.
(ML) UNCONTROLLED FILL - SILT WITH SAND, GRAVEL AND COBBLES, brown, loose, moist	2.5	ML								Debris including metal, plastic, and glass encountered from 0 to 5 feet below ground surface.
(ML) LOESS - SILT WITH SAND, brown, firm, moist	5.0	ML								
(RX) BASALT, highly to moderately weathered, highly fractured, dark gray whiteish orange staining, hard	7.5	RX								
Test Pit Terminated at 8.5 Feet.										Test pit terminated at 8.5 feet below ground surface upon refusal to due bedrock density. Loosely backfilled upon completion.

Client: Keller Associates	**Test Pit Number:** TP-23073A-6	GeoProfessional Innovation.	**EXPLORATORY TEST PIT LOG**
Project: MO23073A	**Date Excavated:** 03-07-2025		
Backhoe: CASE 580	**Bucket Width:** 2.0 feet		
Depth to Groundwater: N.E.	**Logged By:** AJA		Sheet 1 Of 1

GPI TEST PIT - GPI.GDT - 5/16/25 08:29 - V:\ENGINEERING\GINT FILES\2025\250307 MO23073A GUN CLUB ROAD TEST PITS.GPJ

USCS Description	Depth (ft)	U.S.C.S. Class	Symbol	Sample Type	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	Atterberg Limits LL PI	Remarks Note: BGS = Below Ground Surface
(ML) UNCONTROLLED FILL - SILT WITH GRAVEL AND COBBLES, brown, loose, moist	0.0	ML								Trace vegetation and organics at ground surface.
(ML) LOESS - SILT, brown, firm, moist	2.5	ML								
(RX) BASALT, highly to moderately weathered, highly fractured, dark gray whiteish orange staining, hard	5.0	RX								
Test Pit Terminated at 8.0 Feet.	7.5									Test pit terminated at 8 feet below ground surface upon refusal to due bedrock density. Loosely backfilled upon completion.

Client: Keller Associates	**Test Pit Number:** TP-23073A-7	GeoProfessional Innovation.	**EXPLORATORY TEST PIT LOG**
Project: MO23073A	**Date Excavated:** 03-07-2025		
Backhoe: CASE 580	**Bucket Width:** 2.0 feet		Sheet 1 Of 1
Depth to Groundwater: N.E.	**Logged By:** AJA		

GPI TEST PIT - GPI.GDT - 5/16/25 08:29 - V:\ENGINEERING\GINT FILES\2025\250307 MO23073A GUN CLUB ROAD TEST PITS.GPJ

APPENDIX B

Laboratory Test Results

GeoProfessional Innovation

Project Name: Gun Club Rd, Lapwai Rd to Stewart Ave
Project Number: MO23073A
Client: Keller Associates

Laboratory Test Results Summary

Boring/Test Pit	Depth (feet)	Lab Number	Description (U.S.C.S. Classification)	In situ Moisture, %	In situ Dry Density, pcf	Max Dry Density, pcf	Optimum Moisture, %	C, psf	ϕ, deg	Atterberg Limits		#200 Sieve Passing, %	Organic Matter, %	Cation Exchange Capacity (meq/100g)	pH	Resistivity Ω·cm	Sulfates ppm
										Liquid Limit	Plasticity Index						
B-23073A-1	0.5-1.0	15722	Poorly-Graded Sand with Gravel (SP)	3.3		-	-	-	-	-	-	4.7	-	-	-	-	-
B-23073A-1	1.5-2.5	15723	Sandy Silt (ML)	13.2		112.5	15.5	-	-	23	4	53	-	-	-	-	-
TP-23073A-2	6.5-7.0	19807	Silt with Sand (ML)	15.1	100.7	-	-	325	36.5	-	-	-	-	-	-	-	-
B-23073A-3	2.5-4.0	15724	Silt with Sand (ML)	11.9		-	-	-	-	-	-	-	-	-	-	-	-
B-23073A-4	5.0-6.5	15725	Silt with Sand (ML)	21.0		-	-	-	-	-	-	-	1.55	12.7	8.7	2,700	72.9
B-23073A-5	2.5-4.0	15726	Silt with Sand (ML)	9.9		-	-	-	-	-	-	-	-	-	-	-	-
B-23073A-6	7.5-9.0	15727	Silty Gravel with Cobbles (GM)	7.3		-	-	-	-	-	-	-	-	-	-	-	-
B-23073A-7	1.0-2.5	15729	Silty Sand (SM)	6.1		115.0	12.5	-	-	Non-Plastic		43	-	-	-	-	-
B-23073A-7	10.0-11.5	15728	Silty Sand (SM)	13.3		-	-	-	-	-	-	-	-	-	-	-	-
B-23073A-8	5.0-6.5	19808	Silty Sand (SM)	-	-	-	-	-	-	-	-	-	1.58	15.2	8.7	1,370	70.5
TP-23073A-1	2.0-3.0	19308	Silty Sand with Gravel (SM)	-		-	-	-	-	-	-	24	-	-	8.2	2,860	9.0
TP-23073A-5	5.0-6.0	19307	Silt (ML)	-		-	-	-	-	-	-	52	-	-	8.2	1,690	60.0

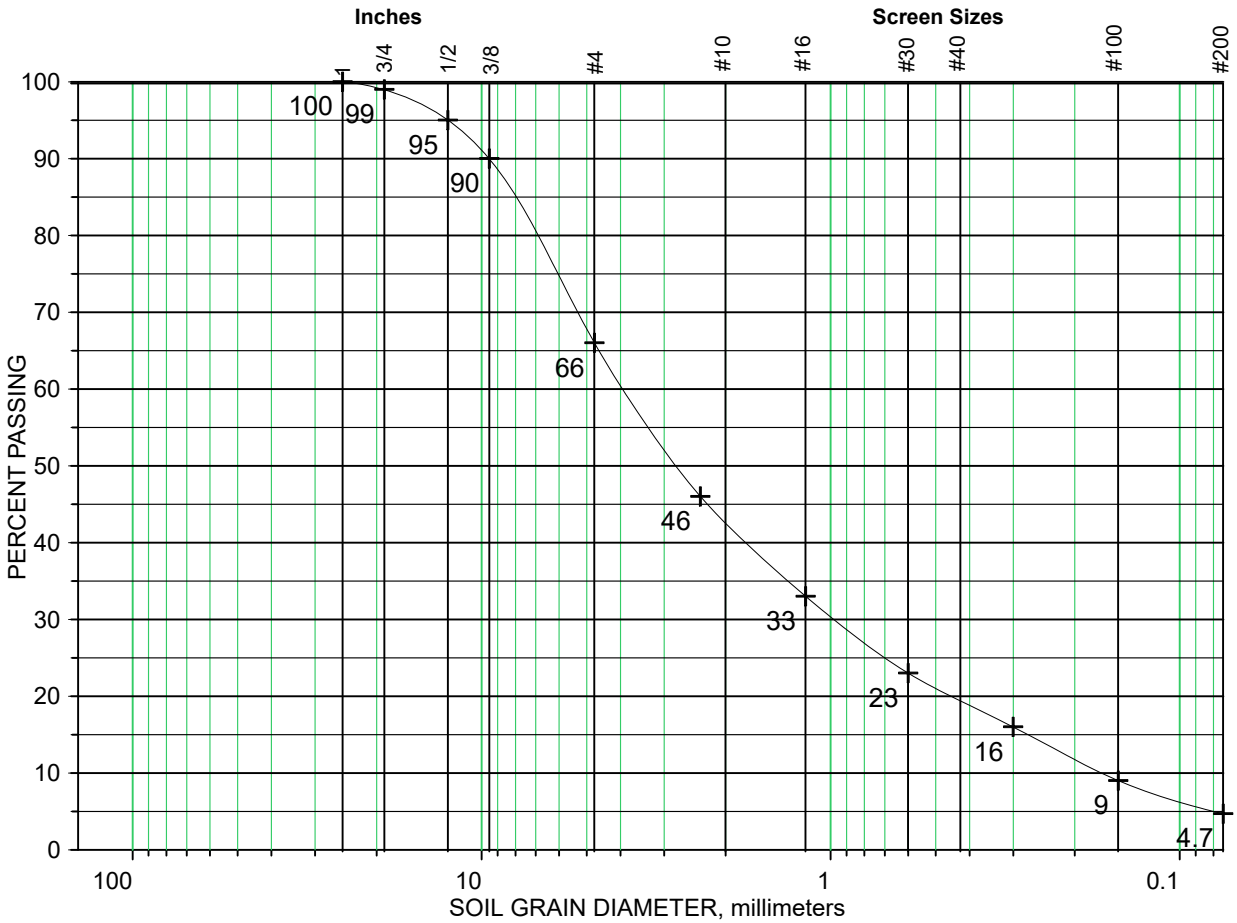
Reviewed By: Andy Abre

GRADATION ANALYSIS

ASTM D6913

Project Name: Gun Club Rd, Lapwai Rd to Stewart Ave
Project Number: MO23073A
Client: Keller Associates
Lab Number: 15722
Sample Location: B-23073A-1 @ 0.5-1.0 feet BGS
Sample Classification: Poorly-Graded Sand with Gravel (SP)
Date Tested: 11/2/2023 By: SS

Cobbles	Gravel		Sand		
	Coarse	Fine	Coarse	Medium	Fine



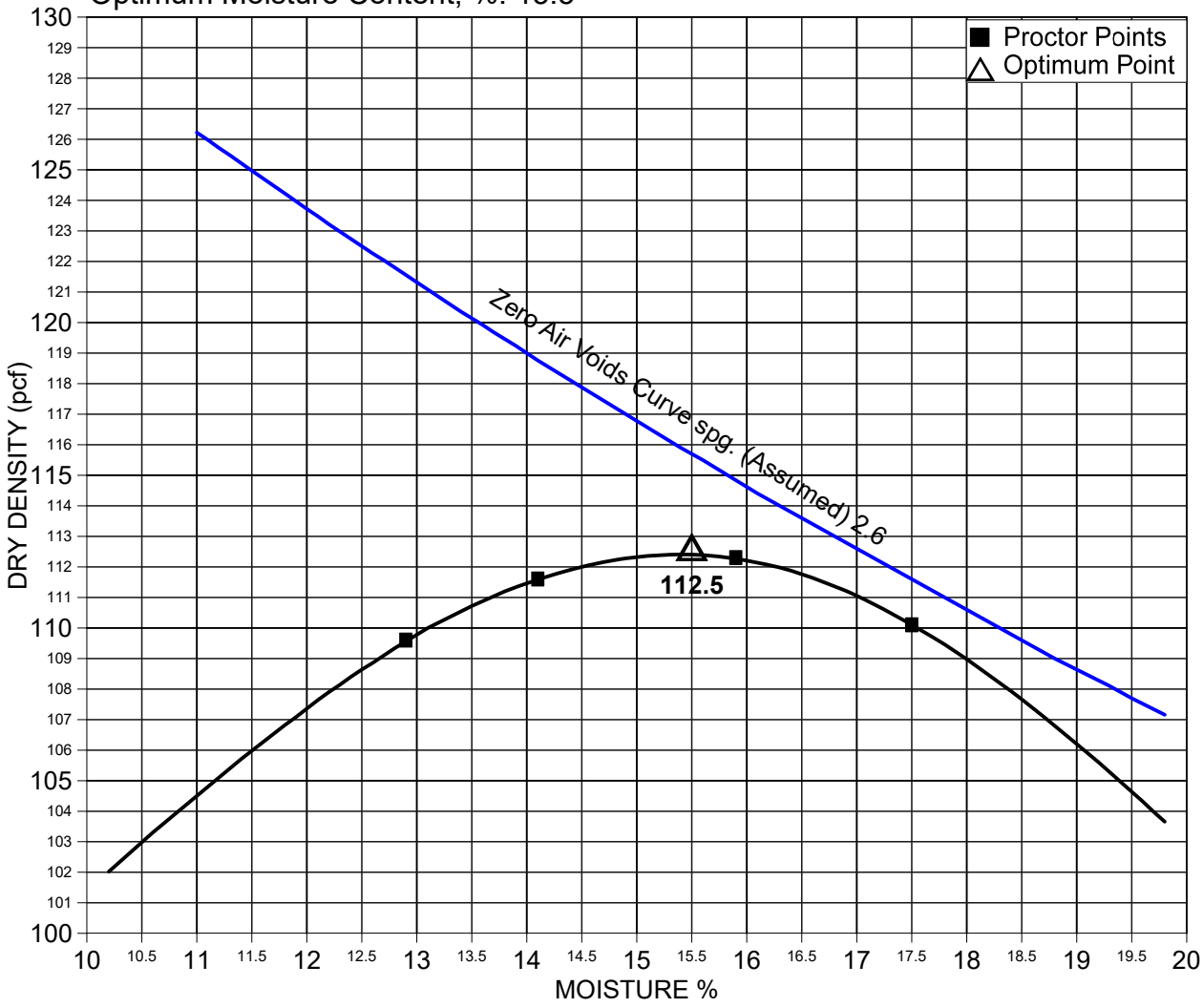
Reviewed by: Campbell

GeoProfessional Innovation.

MOISTURE-DENSITY RELATIONSHIP CURVE
ASTM D 1557
Method A

Project Name: Gun Club Rd, Lapwai Rd to Stewart Ave
Project Number: MO23073A
Client: Keller Associates
Lab Number: 15723
Sample Location: B-23073A-1 @ 1.5-2.5 feet BGS
Sample Classification: Sandy Silt (ML)
Date Tested: 11/9/2023 By: SS
Rammer Type: Manual

Maximum Dry Density, pcf : 112.5
Optimum Moisture Content, %: 15.5



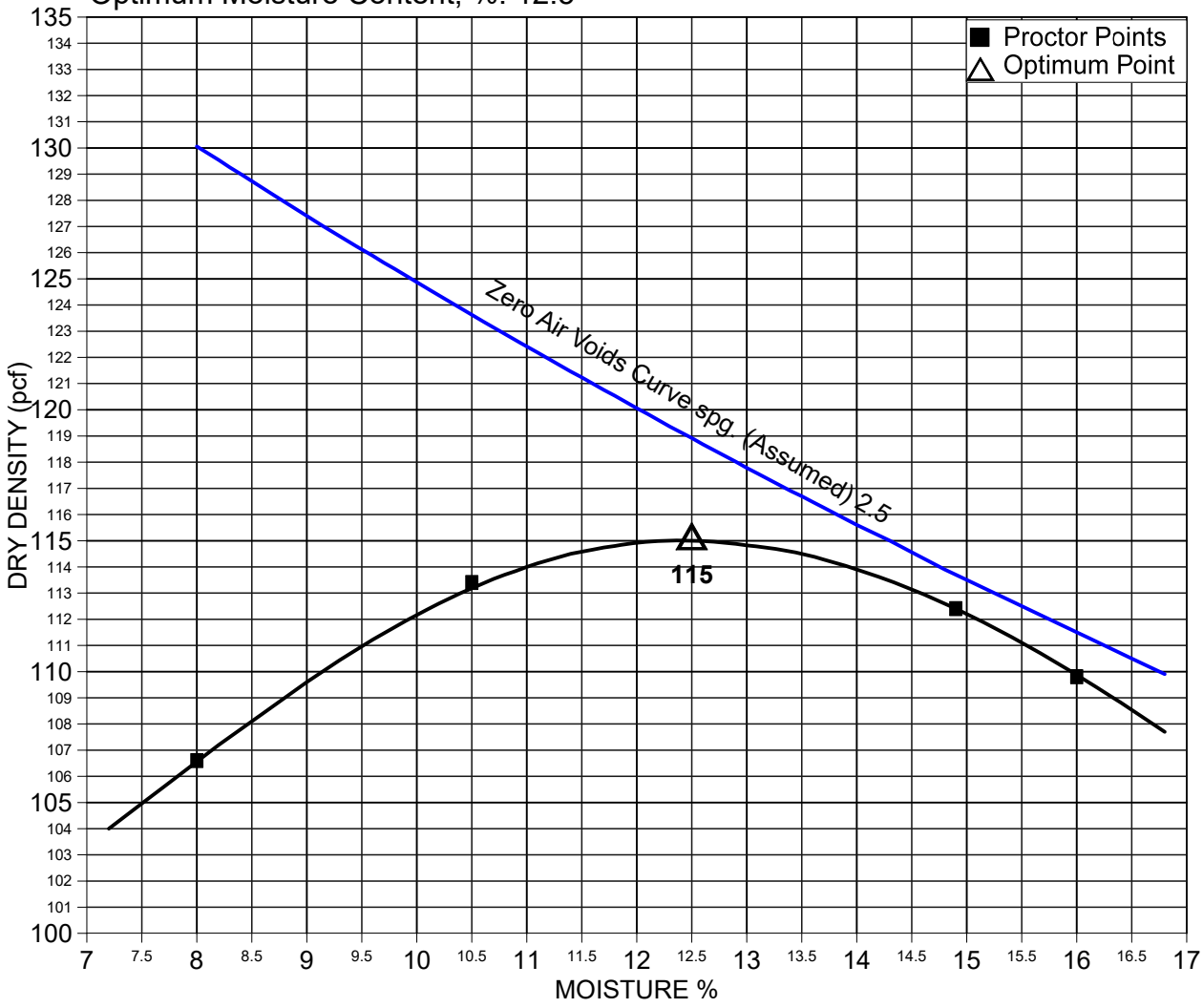
Reviewed By: _____

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MOISTURE-DENSITY RELATIONSHIP CURVE
ASTM D 1557
Method A

Project Name: Gun Club Rd, Lapwai Rd to Stewart Ave
Project Number: MO23073A
Client: Keller Associates
Lab Number: 15729
Sample Location: B-23073A-7 @ 1.0-3.0 feet BGS
Sample Classification: Silty Sand (SM)
Date Tested: 01/22/2024 By: RG
Rammer Type: Manual

Maximum Dry Density, pcf : 115.0
Optimum Moisture Content, %: 12.5



Reviewed By: _____

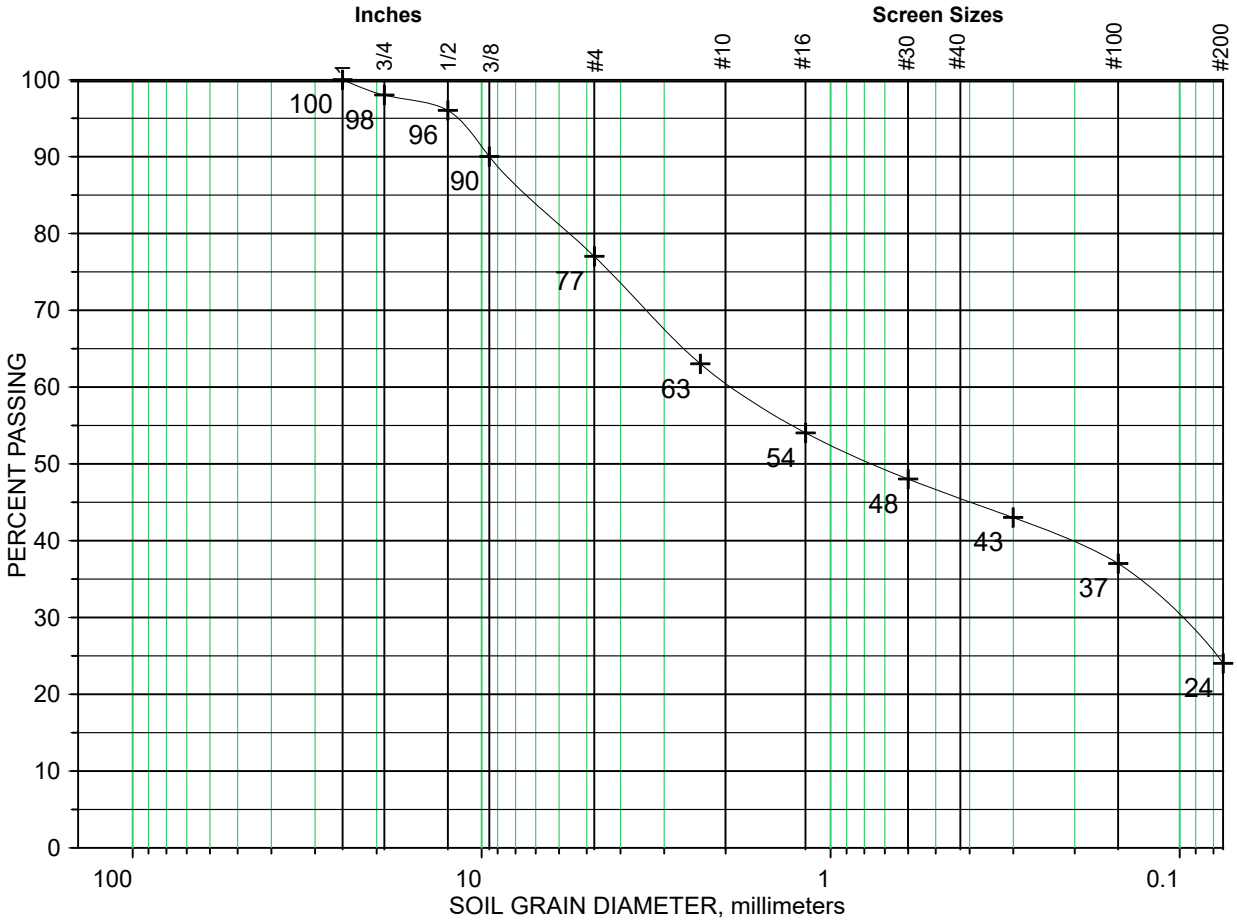
GeoProfessional Innovation.

GRADATION ANALYSIS

ASTM D6913

Project Name: Gun Club Rd, Lapwai Rd to Stewart Ave
Project Number: MO23073A
Client: Keller Associates
Lab Number: 19308
Sample Location: TP-23073A-1 @ 2.0-3.0 feet BGS
Sample Classification: Silty Sand with Gravel (SM)
Date Tested: 03/12/2025 By: GD

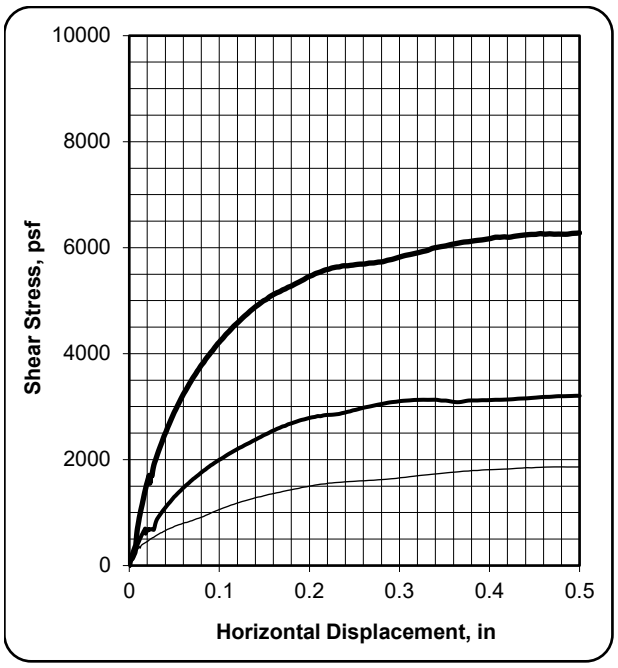
Cobbles	Gravel		Sand		
	Coarse	Fine	Coarse	Medium	Fine

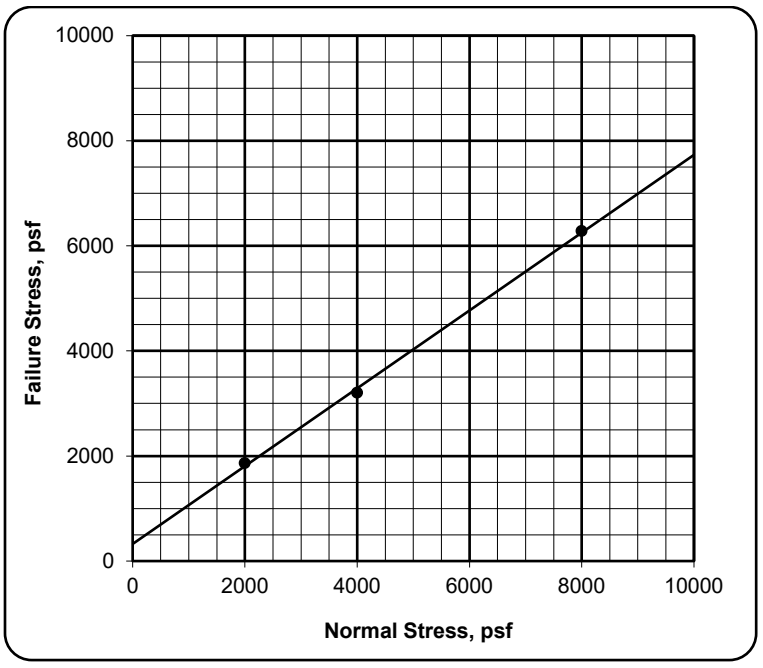


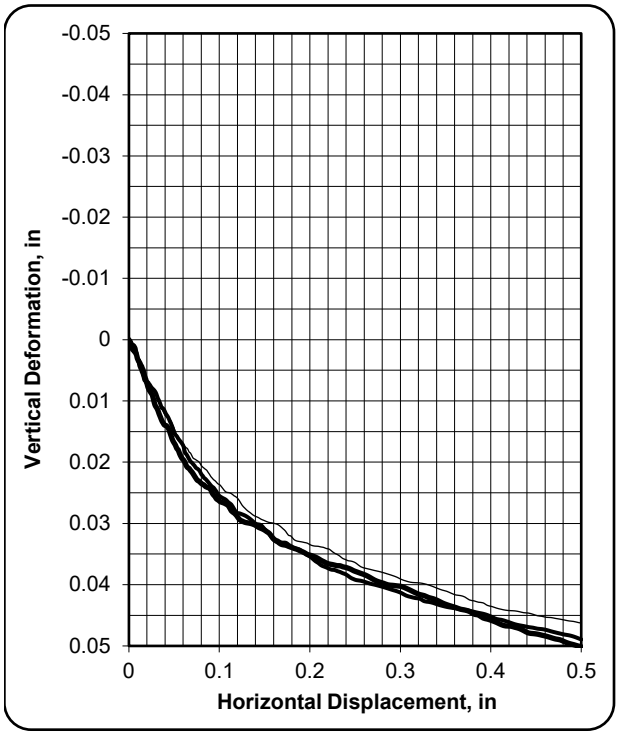
Reviewed by: Campbell

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Direct Shear Test Report - ASTM D-3080







	Sample Number	1	2	3
Initial	Water Content, %	15.1	15.1	15.1
	Dry Density, pcf	102.1	100.5	100.7
	Saturation, %	64.4	61.9	62.2
	Void Ratio	0.62	0.65	0.64
	Diameter, in	2.421	2.421	2.421
	Height, in (before consol.)	0.995	0.995	0.995
At Test	Water Content, %	17.8	17.6	16.6
	Dry Density, pcf	104.3	105.8	108.9
	Saturation, %	80.3	83.1	84.9
	Void Ratio	0.59	0.56	0.52
	Diameter, in	2.421	2.421	2.421
	Height, in (prior to shear)	0.974	0.945	0.920
	Normal Stress, psf	2000	4000	8000
	Failure Stress, psf	1863	3204	6278
	Displacement @ Failure, in	0.50	0.50	0.50
	Time to Failure, min	100.0	99.3	100.0
	Ultimate Stress, psf	1863	3203	6278
	Displacement @ Ultimate, in	0.50	0.50	0.50
	Rate of Strain, in/min	0.0050	0.0050	0.0050

Client	GeoProfessional Innovation
Project	Gun Club Road
Project Number	108524-025
Sample Location	TP-23073A-2, Ring; 6.5'-7.0'
Lab Number	25-0268
Date	3/31/2025

Sample Type	In-situ
Description	Silt with Sand (ML)
Sp. Gravity (assumed)	2.650

C, psf	325
ϕ , deg	**36.5**
tan ϕ	0.741

Tested by: PC

Reviewed by: Travis Thomsen

SHANNON & WILSON

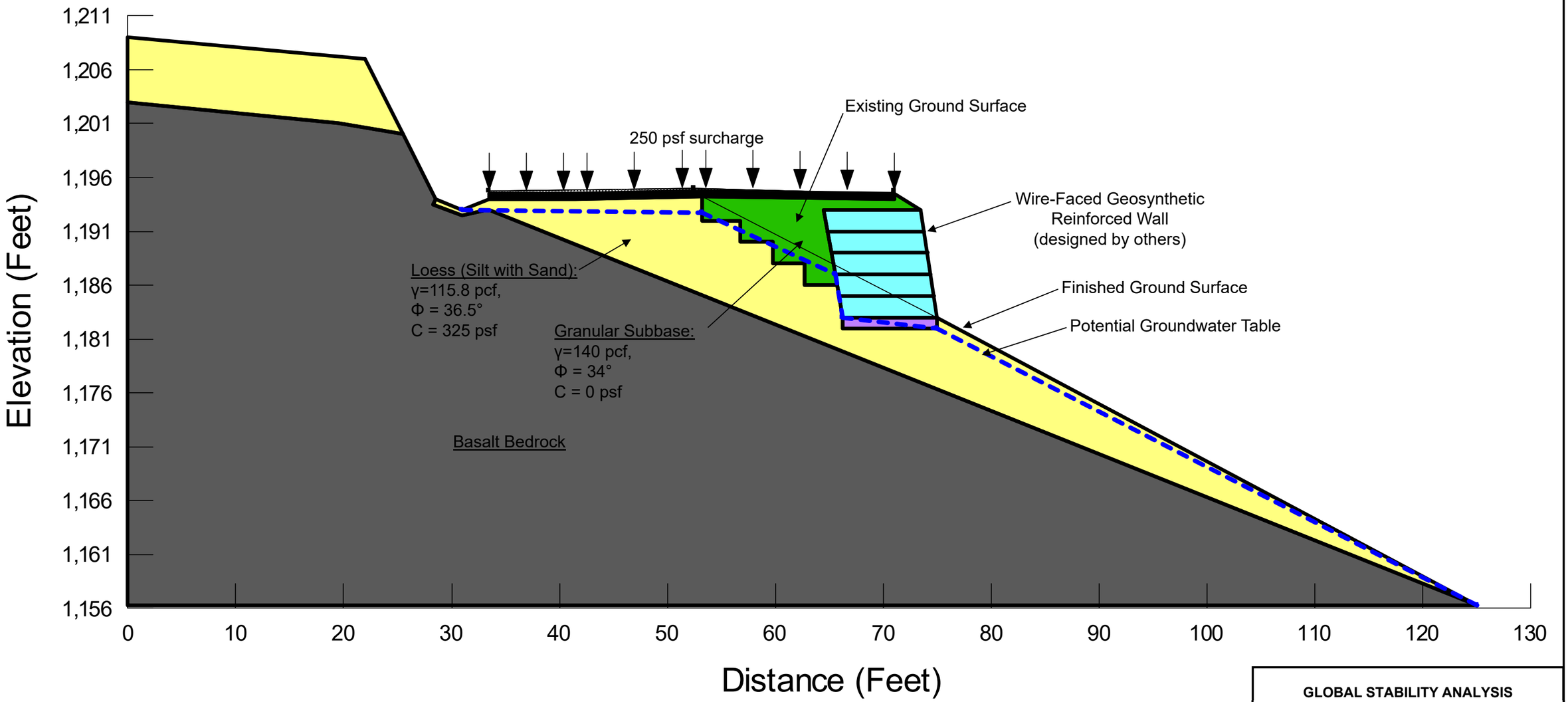
APPENDIX C

Stability Analysis Results

Global Stability Analysis - Geometry

Approximate Station 28+25

Static Conditions



GLOBAL STABILITY ANALYSIS
Gun Club Road, Lapwai Road to Stewart Avenue
Lewiston, Idaho

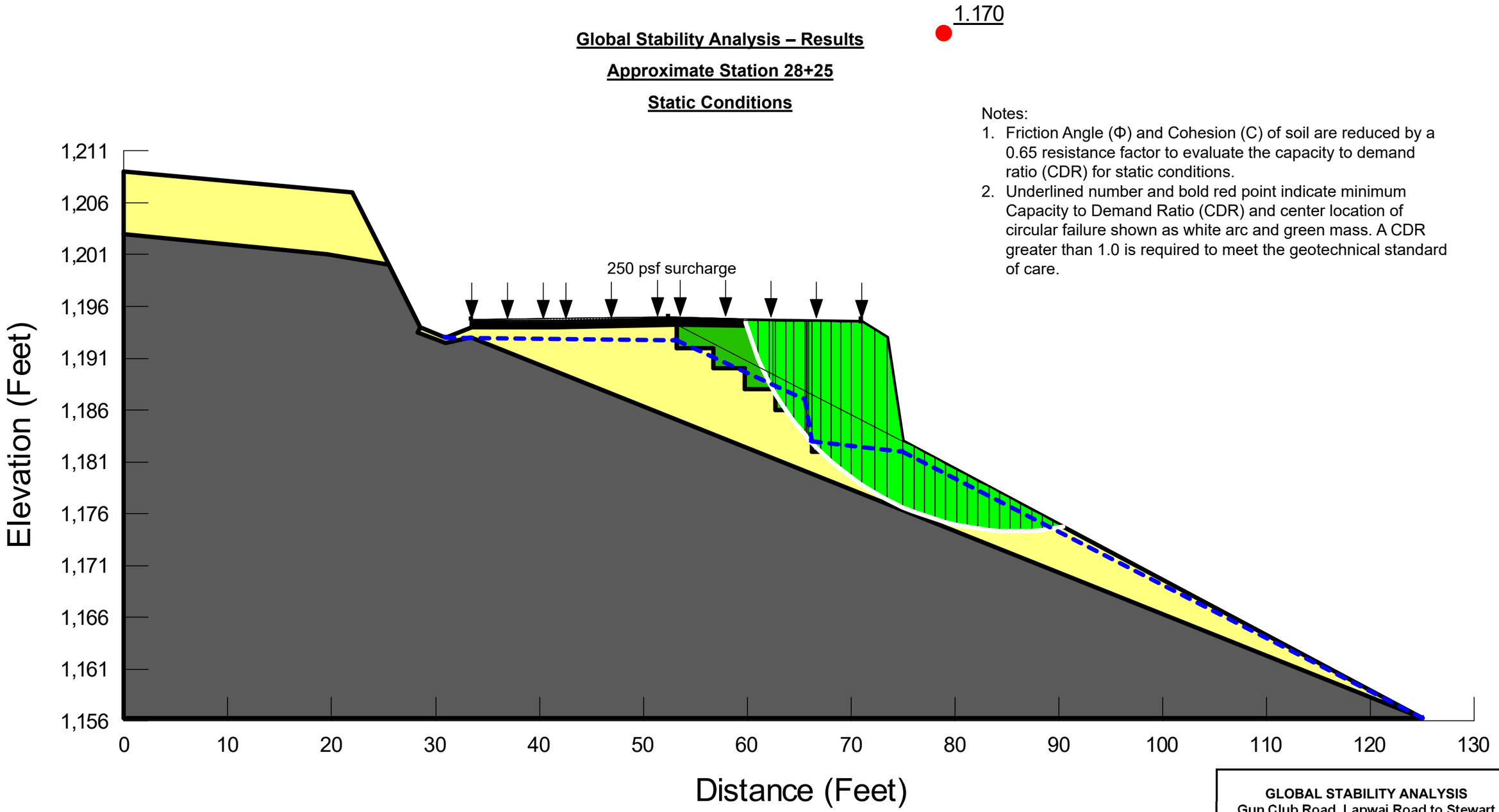
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MO23073A	Figure C.1

Global Stability Analysis – Results

Approximate Station 28+25

Static Conditions



Notes:

1. Friction Angle (Φ) and Cohesion (C) of soil are reduced by a 0.65 resistance factor to evaluate the capacity to demand ratio (CDR) for static conditions.
2. Underlined number and bold red point indicate minimum Capacity to Demand Ratio (CDR) and center location of circular failure shown as white arc and green mass. A CDR greater than 1.0 is required to meet the geotechnical standard of care.

GLOBAL STABILITY ANALYSIS
Gun Club Road, Lapwai Road to Stewart Avenue
Lewiston, Idaho

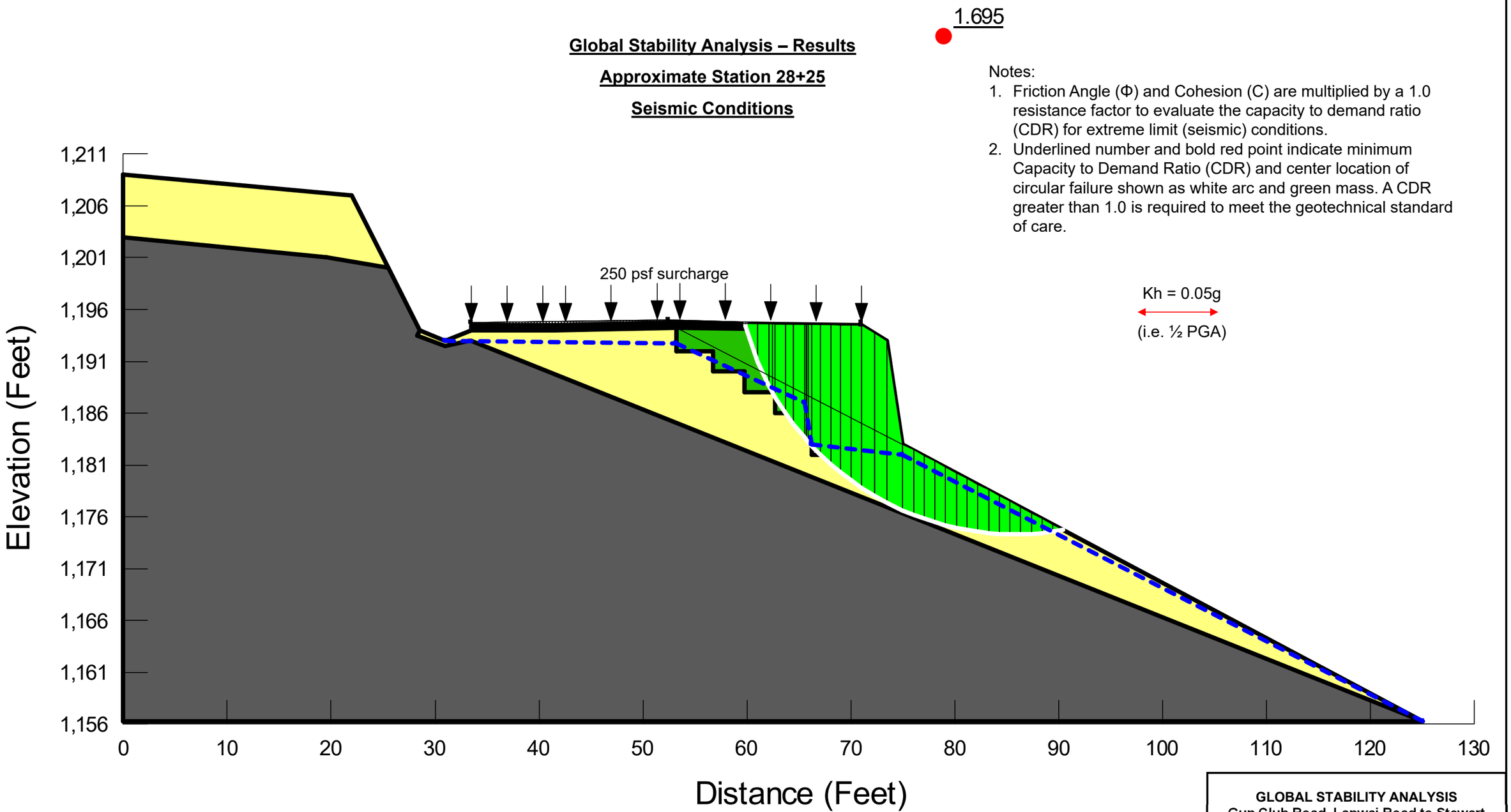
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MO23073A	Figure C.2

Global Stability Analysis – Results

Approximate Station 28+25

Seismic Conditions



Notes:

1. Friction Angle (Φ) and Cohesion (C) are multiplied by a 1.0 resistance factor to evaluate the capacity to demand ratio (CDR) for extreme limit (seismic) conditions.
2. Underlined number and bold red point indicate minimum Capacity to Demand Ratio (CDR) and center location of circular failure shown as white arc and green mass. A CDR greater than 1.0 is required to meet the geotechnical standard of care.

GLOBAL STABILITY ANALYSIS
Gun Club Road, Lapwai Road to Stewart Avenue
Lewiston, Idaho

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MO23073A	Figure C.3