GEOTECHNICAL ENGINEERING REPORT

IDAHO-MARYLAND MINE PROJECT – BRUNSWICK INDUSTRIAL SITE

SW CORNER OF BRUNSWICK ROAD AND EAST BENNETT ROAD
APNs 006-441-003, 006-441-004, 006-441-005, 006-441-034, 009-630-037, 009-630-039
GRASS VALLEY, CALIFORNIA 95945

NOVEMBER 18, 2019

Prepared For:

RISE GRASS VALLEY, INC.
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PROJECT NO. 5279.02

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November 18, 2019

Rise Grass Valley, Inc.
Ben Mossman, President
333 Crown Point Circle, Suite 215
Grass Valley, California 95945

Reference: **Idaho-Maryland Mine Project – Brunswick Industrial Site**
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Grass Valley, California 95945

Subject: **Geotechnical Engineering Report**

Dear Mr. Mossman:

This report presents the results of our geotechnical engineering investigation for the 119-acre property located at the southwest corner of Brunswick Road and East Bennett Road in Grass Valley, California. As proposed, the project is to include development of industrial facilities associated with proposed underground hard rock mining operations. We anticipate that site improvements will include the construction of multiple new light and heavily loaded structures, embankment mitigation of the existing pond, retrofitting and expansion of the existing New Brunswick shaft, construction of a new service shaft and headframe, construction of an engineered fill pad for future industrial use, and associated infrastructure elements including roadways, storm drains and underground utilities.

The findings, conclusions and recommendations presented in this report are based on the following relevant information collected and evaluated by NV5: literature review, surface observations, subsurface exploration, laboratory test results, and experience with similar projects, sites and conditions in the area. Our opinion is that the project can be completed as proposed, provided the recommendations presented in this report are implemented. Our primary concerns, from a geotechnical engineering standpoint, include:

- The presence of relatively thin layers/pockets of organic material (predominantly sawdust) in the southwestern end of the existing pond dam, which can be mitigated by conventional earthwork methods, including excavation and removal of the organic materials followed by replacement and recompaction of the overlying dam fill.
- The stability of steep temporary cut slopes proposed for construction of deep sub-surface structures, which can be mitigated by conventional geotechnical engineering methods, including temporary cut slope design and/or shoring design for the proposed temporary excavations.
- The presence of undocumented fill material in portions of previously graded areas, which can be mitigated by testing of the fill material, and if necessary, reworking and recompaction of fill that is deemed unsuitable to support site development.
- The presence of thin lenses of expansive fine-grained near-surface soils in areas of proposed infrastructure improvements, which can be mitigated by conventional earthwork procedures (e.g., excavation and removal from areas to support structures or foundation elements).

- The existing corrugated metal pipe (CMP) culvert that conveys water through the Site to South Fork Wolf Creek appears to be approaching the end of its lifespan. Considering the moderate corrosive potential of the Site soils encountered in the vicinity of the culvert, as well as the proposed industrial fill pad to be constructed over the culvert alignment, we recommend replacing the existing culvert prior to construction of the engineered industrial fill pad.
- The potential for naturally occurring asbestos (NOA) in imported fill material generated underground, which can be mitigated by implementation of conventional engineering controls to limit dust emissions during earthwork.

Recommendations for mitigating these concerns are presented in this report. During construction, we should be retained to observe and confirm site conditions and modify our recommendations, if necessary. This report should not be relied upon without review by NV5 if a period of 24 months elapses between the issuance report date shown above and the date when construction commences.

NV5 appreciates the opportunity to provide geotechnical engineering services for this important project. If you have questions or need additional information, please do not hesitate to contact the undersigned at 530-478-1305.

Sincerely,
NV5

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Appendix C	Exploratory Trench and Boring Logs
Appendix D	Laboratory Test Data
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ACRONYMS

AB	aggregate base
AC	asphalt concrete
ACI	American Concrete Institute
APN	Accessory's Parcel Number
ASCE	American Society of Civil Engineers
ASTM	ASTM International
bgs	below ground surface
CalEPA	California Environmental Protection Agency
CAT	Caterpillar
CBC	California Building Code
CGS	California Geological Survey
CMP	corrugated metal pipe
CQA	Construction Quality Assurance
CUSD	Chico Unified School District
DTSC	Department of Toxic Substances Control
DWR	California Department of Water Resources
EFP	equivalent fluid pressure
GBA	Geoprofessional Business Association
GIS	Geographical Information System
H:V	horizontal to vertical ratio
IBC	International Building Code
MCE	maximum considered earthquake
M_L	local magnitude earthquake
MSL	mean sea level
mybp	million years before present
NEIC	National Earthquake Information Center
OSHA	Occupational Safety and Hazards Administration
PCA	Portland Cement Association
pcf	pounds per cubic foot
PI	plasticity index
psf	pounds per square foot
psf/ft	per square foot, per foot of depth
psi	pounds per square inch
SSD	saturated surface dry
USCS	Unified Soils Classification System
USGS	United States Geological Survey

1.0 INTRODUCTION

At the request of Ben Mossman of Rise Grass Valley, Inc, NV5 performed a geotechnical investigation of the 119-acre property located at the southwest corner of Brunswick Road and East Bennett Road in Grass Valley, California. The geotechnical investigation was performed in general accordance with NV5's *Proposal for Geotechnical Engineering Services* dated August 30, 2019, a copy of which is included as Appendix A of this report. For your review, Appendix B contains a document prepared by Geoprofessional Business Association (GBA) entitled Important Information about Your Geotechnical Engineering Report, which summarizes the general limitations, responsibilities, and use of geotechnical reports.

1.1 SITE DESCRIPTION

The subject site (herein referred to as the "Site" or "subject site") comprises approximately 119 acres and is located approximately 2 miles east-southeast of downtown Grass Valley in Nevada County, California. According to the Nevada County Geographical Information System (GIS) online database (https://gis.nevcounty.net/MyNeighborhood/) the Property is associated with assessor's parcel numbers (APNs) 006-441-003, 006-441-004, 006-441-005, 006-441-034, 009-630-037, and 009-630-039. The subject site sits in a valley created by the South Fork of Wolf Creek. The Site is bordered by Brunswick Road to the east, East Bennett Road and one residential property to the north, minimally developed residential properties upslope to the south and southwest, and undeveloped steeply sloping naturally vegetated terrain to the west. Figure 1 shows the Site vicinity.

At the time of our field investigation, the Site consisted of the generally flat lying surfaces around the abandoned New Brunswick shaft and sawmill landings, gently sloping open fields and tree covered areas extending downstream of the dam, and steep slopes along the southern and western perimeter of the Site. Site topography varied from flat to gently sloping along portions of the developed areas, to steeply sloping in the western and southern portions of the Site. Deep fill was apparent in the vicinity of the New Brunswick Mine workings. We also observed the mine silo, concrete slabs-on-grade, and the covered New Brunswick shaft in this area. The gently sloping surfaces along the valley floor were covered with thick vegetation and we could not evaluate the nature of the material in this area. The South Fork of Wolf Creek is contained within an approximate 48-inch CMP culvert that trends northwest through the center of the Site and discharges back into its natural channel, continuing northwestward along the northwestern border of the property.

1.2 PROPOSED IMPROVEMENTS

We reviewed site plans for the project including a Grading Plan (sheet B-1) prepared by Nevada City Engineering, Inc.; and Infrastructure plan (sheet B101), Biological and Cultural Summary Map (sheet B102), Infrastructure Details (sheet B103) and Section Views (sheet B104) prepared by Rise Grass Valley, Inc. and dated November 2019. Based on our review of these site plans, we understand that the proposed site improvements for the project will likely include the construction of:

- Multiple new light and heavily loaded structures to support industrial mining activities;
- Mitigation of the existing pond dam embankment and liner;
- Retrofitting and expansion of the existing New Brunswick shaft to support industrial mining activities;
- Construction of a new service shaft and headframe to support industrial mining activities;
- Construction of a storm water detention pond;

- Construction of an engineered fill pad for future industrial use; and
- Associated infrastructure elements including retaining structures.

Appurtenant construction will include asphalt concrete paved roads and parking areas, and underground utilities. We anticipate that grading for the project will include cut and fill for building pads, roadways, culvert improvements, dam mitigation, and retaining structures and excavation for underground utilities.

1.3 PURPOSE

We performed a surface reconnaissance and subsurface geotechnical investigation at the Site, collected soil samples for laboratory testing, and performed engineering calculations to provide general grading and drainage recommendations, foundation and retaining wall design criteria, slab-on-grade recommendations, and pavement design for the proposed improvements.

1.4 SCOPE OF SERVICES

To prepare this report, we performed the following scope of services:

- We performed a site investigation, including a literature review and a subsurface investigation.
- We collected relatively undisturbed soil samples and bulk soil samples from selected exploratory trenches and borings.
- We performed laboratory tests on select soil samples obtained during our subsurface investigation to determine their engineering material properties.
- We assessed the stability of the existing dam embankment at the pond.
- We evaluated the condition of the existing South Fork of Wolf Creek culvert that passes through the center of the Site.
- Based on observations made during our subsurface investigation and the results of laboratory testing, we performed engineering calculations to provide geotechnical engineering recommendations for earthwork and structural improvements.

Our scope of services did not include a groundwater flow analysis nor an evaluation of the site for the presence of hazardous materials, historic mining features, asbestiform minerals, mold, or radon gas. Therefore, the presence and removal of these materials are not discussed in this report.

2.0 SITE INVESTIGATION

NV5 performed a site investigation to generally characterize the existing surface and subsurface conditions across the Site, concentrating in the areas of proposed development. The site investigation included a literature review of published and unpublished geologic documents and maps, a surface reconnaissance investigation, and a subsurface exploratory investigation using a track-mounted excavator to excavate exploratory trenches and a truck-mounted drill rig to excavate exploratory borings. Each component of the site investigation is presented below.

2.1 LITERATURE REVIEW

We performed a limited review of geologic literature pertaining to the subject site. The following sections summarize our findings.

2.1.1 Soil Survey

As part of our study, we reviewed the *Soil Survey of the Nevada County Area, California* (USDA Soil Conservation Service, reissued August 1993). The soil survey indicates that the Site is underlain by four soil types, as described below.

The central-southwest and southeast portions of the Site and small isolated areas along Brunswick Road are underlain by Aiken Loam. The soil survey describes the Aiken Loam as a well-drained soil that forms on the sides of andesitic flows. According to the survey, permeability of the Aiken Loam soil type is moderately slow and weathered andesite is commonly encountered at about 64 inches below ground surface (bgs).

The northern and southwest portions of the Site are underlain by Cohasset loam. The Cohasset series is described as well-drained soils that forms on the sides of andesitic flows. According to the survey, permeability of the Aiken Loam soil type is moderately slow and weathered andesite is commonly encountered at about 96 inches bgs.

Most of the central portion of the site is characterized by Placer diggings while the central-northwestern and central-southeastern portion contains clayey Alluvial Land. The soil survey describes clayey Alluvial Land as a miscellaneous land type consisting of narrow areas of alluvial deposits. These soils are moderately well drained to poorly drained and permeability is moderately slow to very slow.

2.1.2 Geologic Setting

The Property is located within a region underlain by a complex assemblage of igneous and metamorphic rocks in the western foothills of the Sierra Nevada. The regional structure of the foothills is characterized by the north-northwest trending Foothills Fault System, a feature formed during the Mesozoic era (between approximately 65 million and 248 million years ago) in a compressional tectonic environment. A change to an extensional tectonic environment during the late Cenozoic (approximately within the last 30 million years), resulted in normal faulting which has occurred coincident with some segments of the older faults near the Site.

According to the Geologic Map of the Chico Quadrangle, California (California Department of Conservation, Division of Mines and Geology, 1992) the northern and central portion of the Site is underlain by massive diabase, and the southeastern portion of the Site is underlain by metavolcanic rocks. Both of these units are associated with the Mesozoic Lake Combie Complex. The south and southwestern portion of the Site is mapped as Miocene to Pliocene volcanics, predominantly andesitic pyroclastic rocks. The Mesozoic era spans the period of time between 525 and 66 million years before present and the Miocene to Pliocene epochs span the period of time between 23 and 2.6 million years before present.

The Geologic Map of the Grass Valley - Colfax Area (A. Tuminas, 1983) presents the findings of a more detailed local study. According to this geologic map, one inferred fault trends north-northwest through the center of the property. Four rock units are mapped as underlying the Property. The eastern portion of the site (and fault) is mapped as early Mesozoic Lake Combie metavolcanic rock. The northern and western sloping flanks of the Site are mapped as early Mesozoic Lake Combie massive diabase. The lower valley portions encompassing the South Fork of Wolf Creek is mapped as Quaternary alluvium (i.e., water lain sediments deposited in the past 2 million years). Tertiary clastic strata of the volcanic Mehrten formation is mapped in the south and southwestern portion of the Site.

We reviewed California Geological Survey Open File Report 96-08, Probabilistic Seismic Hazard Assessment for the State of California, and the 2002 update entitled California Fault Parameters.

The documents indicate the property is located within the Foothills Fault System. The Foothills Fault System is designated as a Type C fault zone, with low seismicity and a low rate of recurrence. The 1997 edition of California Geological Survey Special Publication 42, Fault Rupture Hazard Zones in California, describes active faults and fault zones (activity within 11,000 years), as part of the Alquist-Priolo Earthquake Fault Zoning Act. The map and document indicate the site is not located within an Alquist-Priolo active fault zone.

2.2 PREVIOUS SITE INVESTIGATIONS

NV5 reviewed previous known geological/geotechnical investigation performed at the Site. The following summaries include findings and conclusions presented in these reports that are pertinent to the currently proposed development. The referenced reports are included in Appendix E.

- (Vector, November 3, 1988)
  Vector Engineering (Vector) performed a geologic, hydrogeologic, and subsoil conditions investigation and performed groundwater monitoring of the pond located at the Site, referred to as the Grass Valley Lumber Mill Recycle Pond. At the time of the investigation, the pond was under construction and was proposed to be utilized for containment of Site runoff water to be applied to stacked timber for the mill. Subsurface borings encountered a grayish green sandy silt with organic matter and "lumber mill effluent (sawdust)" from approximately 9 to 17 feet bgs along the south embankment of the pond, and coarse (gravel to cobble size) angular mine tailings from approximately 7 to 16 feet bgs along the northwest embankment. Groundwater monitoring revealed that groundwater movement appeared to be in a west to southwesterly direction, flowing towards the South Fork of Wolf Creek drainage. Findings included that "any seepage that may occur through the clay liner could potentially flow through the pervious coarse mine tailings (and sawdust) located beneath the west (and southwest) side of the pond."

- (Vector, July 20, 1989)
  Vector Engineering provided in-situ and laboratory permeability tests and quality control for construction of the clay liner in the pond. Periodic monitoring of the clay liner construction and lab and field testing, including permeability and compaction testing, was performed from mid-May to mid-June 1989. Conclusions in the summary report stated that the "clay liner system was constructed in substantial accordance with the State Water Resources Control Board Title 23, Subchapter 15, Section 2532 regarding construction of the natural liner system for Class II Designated Waste Management Units."

- (H&K, May 24, 1996)
  NV5, previously Holdrege & Kull (H&K), performed a geotechnical investigation and stability analysis for the pond at the Site for proposed use as a retention basin and water treatment facility during proposed dewatering of the mine. The subsurface investigation included the advancement of six exploratory borings through the dam in April 1996. Subsurface soil conditions encountered in the borings located on the south/southwest end of the dam included well preserved to slightly decomposed sawdust, up to 6 feet thick, at approximately 12.5 to 15 feet bgs. A slope stability analysis was performed on two subsurface soil conditions encountered in the dam embankment. The results of the stability analysis indicated that the earth dam in the south/southwest end was marginally stable under the assumed loading conditions. Consolidation testing of native soil below the dam indicated that the majority of settlement below the dam had already occurred. H&K's conclusions found the dam to be marginally stable in its condition at the time of the investigation and adequate for use during dewatering of the mine. However, reconstruction of the south/southwest end of the dam was strongly recommended for long-term use of the dam for water retention.

- (H&K, April 29, 1997)
  NV5, previously Holdrege & Kull (H&K), performed a limited geotechnical investigation for a previous owner of the property intended for a proposed hoist building to be located west of the existing shaft of the Idaho Maryland Mine. The services performed included a subsurface investigation, including two hand-augured borings, and laboratory-testing program to provide foundation recommendations and design criteria for construction of a proposed hoist building. The proposed hoist building for this investigation was to be located approximately 100 feet west of the existing New Brunswick Shaft on the subject Site. Subsequently, the proposed hoist building was never constructed.

- (H&K, October 25, 2004)
  NV5, previously H&K, prepared a Preliminary Geotechnical Engineering Report for three Grass Valley sites including the northern 37 acres of the subject site, referred to as the New-Brunswick Site. The scope of the preliminary investigation included review of pertinent geologic, soil survey and historical information; report review; and site reconnaissance investigations to observe surface conditions and assess the feasibility of development for industrial facilities associated with mining. The report concluded that the development of the sites, including the New-Brunswick Site, was feasible from a geotechnical engineering standpoint. Primary concerns included the possible presence of and potential to encounter: relict mine features, undocumented fills, waste rock piles, relatively shallow resistant rock, and potentially expansive clay soils. H&K stated that the recommendations presented in the Preliminary Geotechnical Engineering Report should be verified by a design-level geotechnical engineering investigation.

2.3 FIELD INVESTIGATION

We performed our field investigations on September 25 & 30, 2019. During our field investigation, we observed the local topography and surface conditions and performed a subsurface investigation. The following sections summarize surface and subsurface conditions observed during our field investigation.

Our subsurface investigation included the excavation of 17 exploratory trenches across the subject site and the advancement of 7 exploratory borings along the south and southwestern portion of the pond dam. We excavated our trenches to depths ranging between 2 and 9 feet below the ground surface (bgs) using a Hyundai 60CR-9A excavator equipped with a 24-inch bucket. The exploratory borings were advanced to depths ranging between 17 and 26.5 feet below the ground surface (bgs) using a truck-mounted CME-75 drill rig equipped with 7-inch outside diameter, continuous flight, hollow-stem augers. Figure 2 shows the approximate exploratory trench/boring locations.

NV5's field geologist logged each exploratory trench/boring using the ASTM D2487 Unified Soils Classification System (USCS) as guidelines for soil descriptions and the American Geophysical Union guidelines for rock descriptions. Engineering judgment was used to extrapolate the observed soil, rock and groundwater conditions to areas located between and beyond the subsurface exploratory excavations. Additionally, soil samples were collected at select locations and intervals for further classification and laboratory testing. We obtained samples from the exploratory trenches using a hand-actuated slide sampler and shovel. Relatively undisturbed soil samples were collected from the borings using a 2.5 inch inside diameter split spoon sampler equipped with stainless steel liner sampler tubes. The sampler was driven into the soil using an automatic trip hammer weighing 140 pounds with a 30 inch free fall. The stainless-steel liner samples were sealed with labeled plastic caps. All of the representative soil samples were transported to the NV5 Nevada City office soil laboratory facility.

2.3.1 Surface Conditions

At the time of our field investigation, the Site consisted of the generally flat lying surfaces around the abandoned New Brunswick shaft and sawmill landings, gently sloping open fields and tree covered areas extending downstream of the dam, and steep slopes along the southern and western perimeter of the site. Site topography varied from flat to gently sloping along portions of the developed areas, to steeply sloping in the western and southern portions of the Site. According to the base topographic map provided by Nevada City Engineering, site elevations ranged from 2648 feet above mean sea level (MSL) at the northwestern corner of the Site to 2931 feet above MSL at the southwestern corner of the property.

Deep fill was apparent in the vicinity of the New Brunswick Mine workings and the downslope ends of the sawmill landings. We also observed the mine silo, concrete slabs-on-grade, and the covered New Brunswick shaft in the northern area of the Site. The gently sloping surfaces along the valley floor, created by the South Fork of Wolf Creek, were covered with thick vegetation.

Vegetation on the site was typical of the Sierra Nevada Foothills, with areas of dense oak and pine trees, manzanita and poison oak, and open fields of grasses and forbs. Seasonal drainage courses traversed the Site, generally trending north and northwest towards the South Fork of Wolf Creek. The seasonal stream courses were lined with blackberry thickets and riparian grasses. The South Fork of Wolf Creek is contained within an approximate 48-inch CMP culvert that trends northwest through the center of the Site and discharges back into its natural channel continuing northwestward along the northwestern border of the property. The outfall of the CMP culvert showed signs of corrosion and structural distress when observed during our surface reconnaissance. Stream water was observed transmitting below the culvert bottom, indicating that the culvert may have reached, or is approaching, its lifespan.

2.3.2 Subsurface Soil Conditions

The soil/rock types encountered across the Site during the subsurface investigation were classified using the ASTM D2487 Unified Soils Classification System (USCS) as guidelines for soil descriptions and the American Geophysical Union guidelines for rock descriptions. The soil and rock descriptions include: visual field estimates of the particle size percentages (by dry weight), color, relative density or consistency, moisture content and cementation that comprise each soil material encountered. After evaluating the laboratory results and descriptions of each soil/rock unit encountered in our exploratory borings and trenches across the Site, generalized soil and rock units were identified based on similarities in engineering properties and characteristics. While grouping of similar soil types is beneficial for providing general geotechnical engineering evaluations and recommendations, it should be understood that these units are not necessarily stratigraphically continuous across the Site and variations in characteristics, gradation and thickness should be acknowledged. Generalized descriptions of the soil and rock units encountered during the exploratory trenching investigation of the Site are described below:

- **Unit A - (SC) Clayey Sand with Gravel:** Yellowish red, medium dense to dense native residual soil encountered in the northern portion of the Site.
- **Unit B - (ML) Sandy Silt:** Dark reddish brown, very stiff to hard, native residual soil consistent throughout the Site.
- **Unit C - (MH) High Plasticity Silt with Sand:** Mottled pale yellow to brownish yellow, very stiff, highly expansive native residual soil with some cobbles and boulders encountered on the central eastern boundary of the Site.

- **Unit D - (ML) Sandy Silt with Gravel:** Reddish brown, soft to firm, weak and porous, creep prone native residual surface soil encountered on the upper slopes in the southwestern portion of the Site.
- **Unit E - (Rx) Volcanic Rock:** Dark reddish brown, weakly to moderately cemented, highly to completely weathered, friable to weak completely weathered rock encountered in the southwestern portion of the Site.
- **Unit F_a - (SC) Clayey Sand with Gravel:** Yellowish red, medium dense to dense heterogeneous fill with occasional cobbles and trace amounts of debris encountered in the developed areas across the Site.
- **Unit F_b - (GW) Well Graded Gravel with Sand:** Light gray, medium dense to dense crushed mine rock fill with abundant angular cobbles and boulders encountered in developed areas across the Site.
- **Unit F_c - (SC) Clayey Sand with Gravel:** Yellowish red, medium dense heterogeneous fill with wood debris and boulders encountered in the lower elevations on the southern portion of the Site.
- **Unit F_o - (OM) Organic Soil:** Black to brown, very soft to medium stiff, highly compressible, weak and porous, fill comprised of decayed organics (predominantly bark) encountered in fill piles scattered across the southern perimeter of the developed portion of the Site.

Generalized descriptions of the soil units encountered during the exploratory boring investigation of the south/southwestern end of the pond dam are described in stratigraphic succession below:

- **<u>Upper Fill</u> - (CL/ML) Sandy Clay/Sandy Silt:** Yellowish brown, stiff to very stiff fill encountered in all borings from the surface of the dam embankment to approximately 7.5 to 14 feet bgs.
- **<u>Lower Fill</u> - (GC) Clayey Gravel with Sand:** Grayish brown clay with bluish gray rock, medium dense fill approximately 4 to 8 feet thick, was encountered in borings B19-1 through 6.

  (GW) Well Graded Gravel with Sand: Bluish gray, medium dense crushed mine rock fill approximately 2.5 to 5 feet thick, was encountered in Borings B19-2 & -3.

  (OM) Organic Soil: Black, soft to medium stiff, well preserved to partially decomposed organic fill (predominantly sawdust with some bark) approximately 1 to 4 feet thick, was encountered in Borings B19-4 through 6.
- **<u>Bottom of Fill</u> - (ML/CL) Silt with Sand/Sandy Clay:** Mottled dark brown or olive gray, soft to medium stiff to medium dense fill approximately 1 to 3 feet thick, were encountered in Borings B19-2, -4, -5 & -7.
- **<u>Native Floodplain Deposits</u> - (ML) Silt:** Very dark grayish brown to mottled dark brown, soft to very stiff native soil below fill at least 2 to 4 feet thick, was encountered in Borings B19-2 through 6.
- **<u>Native Residual Soil</u> - (CL) Clay with Sand:** Reddish brown, medium stiff to stiff residual soil below fill at least 1 to 2 feet thick, was encountered in Borings B19-2 & -7.

Detailed descriptions of the soil, rock and groundwater conditions that were encountered in each subsurface exploratory location are presented on the exploratory boring and trench logs included in Appendix C.

2.3.3 Groundwater Conditions

During our site investigation, we did not encounter groundwater seepage in our exploratory trenches or borings, except for shallow seepage encountered from 5-8 feet bgs in Boring B19-5 on the pond dam. This depth was located in the upper fill portion of the dam and correlated approximately to the existing water level of the pond at the time of our investigation. We also did not observe onsite springs or seeps emanating from the ground surface. We did observe drainage channels and swales on the property that indicate seasonal flow of surface water.

Our observations of groundwater conditions were made in September 2019 following a period of dry weather. Although we did not observe groundwater in most of our exploratory trenches or borings, our experience has shown that seepage may be encountered in excavations which reveal the soil/weathered rock transition, particularly during or after the rainy season.

3.0 LABORATORY TESTING

NV5 performed laboratory tests on selected soil samples taken from the subsurface exploratory trenches and borings to determine their geotechnical engineering material properties. These engineering material properties were used to develop geotechnical engineering design recommendations for earthwork and structural improvements. The following laboratory tests were performed using the cited ASTM guideline procedures:

- ASTM D2487 Soil Classification by the USCS
- ASTM D2488 Soil Classification by Visual-Manual Procedures
- ASTM D2216 Soil Moisture Content
- ASTM D2937 In Place Density of Soil
- ASTM D2166 Unconfined Compression
- ASTM D3080 Direct Shear
- ASTM D4318 Atterberg Indices (Dry Method)
- ASTM D4829 Expansion Index
- ASTM D2844 Resistance Value

Table 3.0-1 presents a summary of the geotechnical engineering laboratory test results. Appendix D presents the laboratory test data sheets.

Table 3.0-1 - Laboratory Test Results

Boring/ Trench	**Sample**		**ASTM Test Results$^{(1)}$**								
			D2487/ D2488	**D2216**	**D2937**	**D4318**		**D4829**	**D2166**	**D3080**	**D2844**
No.	***No.***	***Depth (ft)***	***USCS (sym)***	***Moisture Content (%)***	***Dry Density (pcf)***	***Plasticity Index (%)***	***Liquid Limit (%)***	***Expansion Index No.***	***UC Compressive Strength (psf)***	***Direct Shear Strength (psf)***	***Resistance Value***
B-17-D1*****	A	49.0	SM						**NA**		
BK-1	BK-1	0.0 - 3.0	CL								33
BK-2	BK-2	0.0 - 3.0	CL								17
T19-2	T19-2-L1	4.0	ML	33.4	77.5				2602		
T19-3	T19-3-L1	3.0	ML	27.6	92.8					447	
T19-4	T19-4-L1	4.0	ML	25.7	95.6				2890		
T19-7	T19-7-L1	1.5	ML	36.5	83.4				3839		
T19-9	T19-9-B1	1.5 - 2.5	MH			42	54	103			
T19-9	T19-9-L1	2.0	MH	41.3	78.8					**0**	
T19-10	T19-10-B1	1.5 - 2.5	ML			67	38				
T19-10	T19-10-L1	1.5	ML	38.1	72.4				3174		

Notes: (1) Laboratory test forms are presented in Appendix D
ASTM = ASTM International
USCS = Unified Soils Classification System
UC = Unconfined Compression
No. = number
NA = Sample was too rocky to run test
% = percent
ft = feet
sym = symbol
psf = pounds per square foot
pcf = pounds per cubic foot
***** Sample was provided by Rise Grass Valley, Inc

3.1 EXPANSIVE SOIL

Atterberg Limits testing was performed on two representative near-surface soil samples (T19-9-B1 and T19-10-B1) collected during the subsurface investigation. The Atterberg Limits test results indicate the fine soil material to be high plasticity silt (MH).

We also performed expansion index testing on one of the samples (T19-9-B1). A portion of the sample was remolded in a 1.0-inch-high ring and submerged in water under an applied loading of 144 pounds per square foot (psf). We observed the loaded sample for a minimum of 24 hours. During that time, we measured the swell (or settlement) with a dial micrometer. Expansion index test results of 103 indicate the sample exhibited high expansion potential, as classified by UBC guidelines.

Based on the results of the Atterberg Limits and expansion index testing and our experience with similar soils in the area, the potential for expansive soil hazards to affect the proposed improvements is considered to be moderate to high. Recommendations for mitigating expansive soils is presented in Section 5.1 of this report.

3.2 SOIL CORROSION POTENTIAL

The site soil corrosion potential was evaluated by Sunland Analytical. The soil samples tested were collected at a depth of approximately 3.0 feet bgs from trenches BK-2 and T19-12. The test results are summarized in Table 3.2-1 below, and are discussed in Section 5.1.14 of this report.

Table 3.2-1, Summary of Corrosion Potential Lab Test Data

Boring No.	Sample No.	Sample Depth (ft)	Test No.	Description	Test Results
BK-2	BK-2	0.0 - 3.0	CA DOT Test #422m	Chloride	7.5 ppm
			CA DOT Test #417	Sulfate	26.2 ppm
			CA DOT Test #643	PH	4.78
			CA DOT Test #643	Min. Resistivity	5360 ohms-cm
T19-12	T19-12-B1	2.0 - 3.0	CA DOT Test #422m	Chloride	1.7 ppm
			CA DOT Test #417	Sulfate	3.0 ppm
			CA DOT Test #643	PH	5.63
			CA DOT Test #643	Min. Resistivity	4020 ohms-cm

Notes:
CA DOT = California Department of Transportation
ppm = parts per million
ohms-cm = ohms-centimeters

4.0 CONCLUSIONS

The following conclusions are based on our field observations, laboratory test results, and our experience in the area.

1. It is NV5's opinion that the site is suitable for the proposed improvements provided that the geotechnical engineering design recommendations presented in this report are incorporated into the earthwork and structural improvement project plans. NV5 should be allowed to review the proposed final earthwork grading plan and structural improvement plans to determine if the geotechnical engineering recommendations have been properly incorporated, are still applicable, or need modifications. Prior to a new phase of development for the project, we should be retained to observe the soil/rock conditions present and confirm or modify our recommendations, if necessary, before grading and construction begins.
2. Our primary concerns, from a geotechnical standpoint include:
   - The presence of relatively thin layer/pockets of organic material (predominantly sawdust) in the southwestern end of the existing pond dam, which can be mitigated by conventional earthwork methods, including excavation and removal of the organic materials followed by replacement and recompaction of the overlying dam fill.
      - Additional borings (B19-4, -5 and -6) advanced along the dam centerline as part of the 2019 drilling program, as well as previous borings (B96-1 and -6) and interview of an equipment operator previously involved with construction of the dam, confirm that a layer of organic fill (sawdust) is present within the southwestern-most 200-foot segment of the dam. Boring logs indicate that the sawdust layer is up to approximately 4 feet thick at the dam centerline at a depth of approximately 12 to 18 feet bgs.

  - Based on the original topography of the dam footprint, it is likely that the organic layer thickness decreases towards the inboard section of pond and increases towards the downslope face of the dam. NV5 recommends that the upslope face of the dam (and the associated clay liner) be preserved during the repairs by the construction of a temporary cut slope. Based on soil conditions observed in the upper fill during our drilling program, we anticipate that a temporary cut slope gradient of approximately 1:1, horizontal to vertical (H:V), will be feasible during the repairs, provided that the water level in the pond is managed so that seepage is not present in the cut slope. A grading plan, cross sections and slope stability analysis of proposed cut slopes should be prepared to describe the proposed earthwork repair prior to grading. If desired, raising of the dam embankment to accommodate higher water levels in the pond is potentially feasible and can be incorporated into dam improvement design and grading plans. The plan and cross section should depict typical temporary cut slope gradients, excavation depths, maximum water surface elevation, and earthwork volume estimates.

- Steep temporary cut slopes are proposed for construction of a shaft collar. Potential slope instability can be mitigated by conventional geotechnical engineering methods, including temporary cut slope design and/or shoring design for the proposed temporary excavations.

- The presence of undocumented fill materials in portions of previously graded areas, which can be mitigated by testing of the fill material and, if necessary, reworking and recompaction of fill that is deemed unsuitable to support site development. During our subsurface investigation of the Site, we encountered existing fill and disturbed soil throughout the property that varied in depth up to approximately 20 feet bgs. In general, existing undocumented fill should not be relied upon to support proposed improvements. Heterogeneous fills of unknown origin, quality and method of placement, such as those encountered, can settle and/or heave erratically under the load of new fills, structures, slabs, and pavements. Footings, slabs, and pavements supported on heterogeneous fill could also crack as a result of such erratic movements. Thus, where not removed by planned grading, heterogeneous fills should not be relied upon to support proposed improvements unless it is excavated and replaced as an engineered fill or verified with supplemental evaluation and confirmation of previous compaction testing (if available). These evaluations will be determined on a case-by-case basis. We provide recommendations for existing fill in Section 5.1.3.

- The presence of thin lenses of expansive clayey near-surface soils in areas of proposed infrastructure improvements, which can be mitigated by conventional earthwork procedures (e.g., removal of the potentially expansive clay or approved blending with granular fill materials). Expansive near-surface soils shrink and swell as they lose and gain moisture throughout the yearly weather cycle. Near the surface, the resulting movements can heave and crack lightly loaded shallow foundations (spread footings) and slabs and pavements. The zone of moisture content fluctuation is dependent on the expansion potential of the soil and the extent of the dry season. The detrimental effects of the above-described movements can be reduced by pre-swelling the expansive soils and covering them with a moisture fixing and confining blanket of properly compacted select fill, as subsequently defined or mixed and blended with granular on-site soil at a mixing ratio provided by our firm. In building areas, the blanket thickness required depends on the expansion potential of the soils and the anticipated performance of the foundations and slabs.

- The existing corrugated metal pipe (CMP) culvert that conveys water through the Site to South Fork Wolf Creek appears to be approaching the end of its lifespan. Considering the moderate corrosive potential of the Site soils encountered in the vicinity of the culvert, as well as the proposed industrial fill pad to be constructed over the culvert alignment, we recommend replacing the existing culvert prior to construction of the engineered industrial fill pad.

- The potential for naturally occurring asbestos (NOA) in imported fill material generated underground, which can be mitigated by implementation of conventional engineering controls to limit dust emissions during earthwork. Earthwork and other disturbance of materials containing mafic and ultramafic rocks is regulated by the California Air Resources Board (CARB) and the Northern Sierra Air Quality Management District (NSAQMD). Pursuant to the California Code of Regulations Title 17, Section 93105, an Asbestos Dust Mitigation Plan (ADMP) is typically required to describe material handling protocols to be used during construction to reduce the release of naturally occurring asbestos (NOA) into the atmosphere during earthwork grading and other soil/rock disturbance.

3. NV5 understands that the proposed mine operations will produce blast rock and sand tailings in approximately equal proportions to be transported to the ground surface and then blended prior to placement as engineered fill to support future industrial site development. Based on particle size data provided by Rise Grass Valley, NV5 anticipates that waste rock and tailings generated by the proposed mining operation will meet the geotechnical engineering criteria for structural fill (Section 5.1.6).

4. Based on our site observations, the geology of the region and our experience in the area, our opinion is that the risk of seismically induced hazards such as slope instability, liquefaction, and surface rupture are remote at the subject site. We do not anticipate that the proposed project will result in the destruction, covering, or modification of any unique geological and/or physical features.

5. Based on the site geology and our observation of the surface conditions, we anticipate that grading and excavation onsite will reveal variably weathered, fractured, metamorphic rock. Areas of resistant rock may be encountered which may require ripping, splitting or hammering, to increase the rate of excavation. In addition, spoil resulting from excavation onsite will likely consist of predominantly angular, gravel to cobble-sized rock fragments. This material may be suitable for use as fill, depending on the nominal size of the rock fragments, but will likely require specific recommendations for fill placement and observation to confirm compaction. Preliminary recommendations addressing rock fill placement are included in this report.

6. During our site investigation, we did not encounter ultramafic rock, serpentinite, or NOA minerals. However, the proposed industrial mining operations may encounter serpentinite, an ultramafic rock often associated with naturally occurring asbestos (NOA). If ultramafic rock, serpentinite or NOA-containing minerals are imported to the ground surface of the site, site grading would be regulated under Cal/EPA Air Resources Board Regulation 93105, Asbestos Airborne Toxic Control Measure for Construction, Grading, Quarrying, and Surface Mining Operations (ATCM) and Placer County Rule 228, Fugitive Dust. We anticipate that, as a minimum, dust mitigation measures such as limiting site access, restricting onsite construction vehicle speeds, covering stockpiled soil, and liberal use of water during grading will be required during grading to prevent the generation of dust from the Site. We can prepare an asbestos dust mitigation plan (ADMP), if required, for project planning and approval purposes.

7. Although we did not observe shallow groundwater or seepage during our surface reconnaissance, we did observe evidence that surface water is seasonally transported through the drainage channels and swales on the property. We anticipate that areas of moist to saturated soil conditions and groundwater seepage will likely be encountered during grading onsite, particularly during the rainy season and/or in excavations which reveal the surface soil/weathered rock transition. Recommendations addressing subsurface drainage, moisture conditioning, surface drainage, and fill placement are presented in the following sections of this report.
8. Based on the site geology and our observation of the surface conditions, we anticipate that resistant bedrock may be encountered in the deep excavation such as the proposed new shaft service raise collar. Areas of encountered resistant rock may require ripping, splitting or hammering to increase the rate of excavation, including utility trenches. In addition, spoils resulting from excavation onsite will likely consist of predominantly angular, gravel to cobble-sized rock fragments. The contractors and subcontractors bidding this job should read this report and become familiar with site conditions as they pertain to their operation and the appropriate equipment needed to perform their tasks. If more detailed information regarding excavatability of the bedrock is required, a seismic refraction study should be performed or additional trenches/borings should be advanced to further define the type and size of equipment planned for construction. This material may be suitable for use as fill, depending on the nominal size of the rock fragments, but will likely require specific recommendations for fill placement and observation to confirm compaction. Preliminary recommendations addressing rock fill placement are included in this report.
9. The conclusions presented in this section are based on information developed from the field and laboratory investigations.

5.0 RECOMMENDATIONS

The following geotechnical engineering recommendations are based on our understanding of the project as currently proposed, our field observations, the results of our laboratory testing program, engineering analysis, and our experience in the area.

5.1 GRADING

The following sections present our grading recommendations. The grading recommendations address import fill, clearing and grubbing, soil preparation, existing fill, cut slope grading, fill placement, fill slope grading, rock fill, erosion control, subsurface drainage, surface water drainage, construction dewatering, underground utility trenches, soil corrosion potential, plan review, and construction monitoring.

5.1.1 Import Fill

Proposed import soil fill and proposed import rock fill should meet the geotechnical engineering material properties described in Sections 5.1.6 and 5.1.8 of this report. This advisory represents the best practice for characterization of soil/rock prior to import for use as engineered fill. The project engineer should approve all proposed import fill for use in constructing engineered fills at the Site.

We understand that the sand tailings will likely have a gradation similar to the historical tailings gradation presented in the table below, and will typically have a large proportion of quartz.

Particle Size Gradation, Historical Sand Tailings

Mesh Size	Particle Size (mm)	Particle Size (micron)	Percent Passing	Description
48	0.300	300	97.7	Medium Sand
65	0.212	212	87.5	Fine Sand
100	0.150	150	63.9	Fine Sand
200	0.075	75	32.3	Silt
325	0.044	44	12.1	Silt

Note: Particle size data provided by Rise Grass Valley.

We understand that the blast rock will typically be crushed underground to approximately 6 inches maximum dimension, and will generally consist of the following rock types.

Typical Blast Rock Composition

Rock Type	Percent of Total
Meta-Andesite	96%
Altered Meta-Andesite	2%
Diabase	1%
Serpentinite	1%

Based on these general particle sizes we make the following recommendations:

1. The proposed use of 100 percent sand tailing for engineered fill is acceptable, provided the grading recommendations presented in this report are followed. Sand tailings should be tested and approved by NV5 prior to placement as engineered fill to confirm it meets the engineered fill criteria and allow for adjustments to gradation or placement and compaction procedures, if necessary.

2. Crushed blast rock with a maximum dimension of 6 inches may be blended into the sand tailings to produce engineered fill material at a ratio of up to 2 parts blast rock to 1 part sand tailings. A rock:sand ratio greater than a 2:1 ratio may be feasible but would not likely be testable using nuclear methods. When testing by nuclear density gauge is not possible, the development of a procedural test method is typically required, involving continuous observation to verify blending, placement and compaction effort.

3. To simplify future site development (e.g., construction of underground utilities and structure foundations), a maximum rock dimension of 2 inches is recommended within five vertical feet of the finished subgrade surface and within five horizontal feet from the face of proposed fill slopes.

4. Onsite blending of blast rock and sand tailings may be performed by earthwork equipment (e.g., windrowing and spreading the rock and sand together in thin lifts). Specific procedures for onsite blending should be developed in conjunction with an NV5 representative during initial fill placement.

5. Based on the particle size distribution and rock types provided (above) by Rise Grass Valley, we anticipate that the density of the engineered fill compacted to 90% of the ASTM D1557 maximum dry density may range from approximately 115 to 140 pounds per cubic foot (pcf) depending upon the percentage of rock blended into the fill. This rough estimate should be verified by laboratory testing of selective ratios of the blended materials.

5.1.2 Clearing and Grubbing

The areas to be graded should be cleared and grubbed to remove vegetation and other deleterious materials, as described below.

1. Strip and remove debris from clearing operations and the topsoil containing shallow vegetation, roots and other deleterious materials. The organic topsoil can be stockpiled onsite and used in landscape areas but is not suitable for use as fill. The project geotechnical engineer should approve any proposed use of the spoil generated from stripping prior to placement.
2. Overexcavate any relatively loose debris and soil that is encountered in our exploratory trenches or any other onsite excavations to underlying, competent material. Possible excavations include exploratory trenches excavated by others, mantles or soil test pits, holes resulting from tree stump or boulder removal, and mining relics.
3. Although not observed during our investigation, if loose, untested fill is encountered during site development, overexcavate to competent native soil or weathered rock a minimum of 5 feet beyond the areas of proposed improvements.
4. Overexcavate any encountered leach lines, abandoned sewer, water, and fuel lines, and loose soil in abandoned subsurface utility line trenches within the proposed improvement areas to underlying competent soil, as determined by a representative of NV5.
5. Remove rocks greater than 8 inches in greatest dimension (oversized rock) from native soil by scarifying to a depth of 12 inches below finish grade in areas to support pavement, slabs-on-grade or other flatwork. Oversized rock may be used in landscape areas, rock landscape walls, or removed from the site. Oversized rock can be stockpiled onsite and used to construct fills, but must be placed at or near the bottom of deep fills and must be placed in windrows to avoid nesting. No oversized rock should be placed in the upper 3 feet of any structural fill. Unless used as rip-rap, oversized rock placed in fill should not be located within 5 feet horizontally of the finished fill slope face. The project geotechnical engineer should approve the use of oversized rock prior to constructing fill.
6. Fine grained, potentially expansive soil, as determined by NV5, that is encountered during grading should be mixed with granular soil, or overexcavated and stockpiled for removal from the subject site or for later use in landscape areas. A typical mixing ratio for granular to expansive soil is 4 to 1. The actual mixing ratio should be determined by NV5.
7. Vegetation, deleterious materials, structural debris, and oversized rocks not used in landscape areas, drainage channels, or other non-structural uses should be removed from the site.

5.1.3 Existing Fill

One of our concerns regarding the subject site is the presence of existing untested fill within the proposed improvement areas. Loose fill beneath footings may contribute to future differential settlement-induced distress. Our opinion is that the existing fill that has not been tested and recorded should not be relied upon to support the proposed improvements without mitigation, as described in the following paragraphs.

Options to mitigate existing fill and loose subsurface conditions include the use of deepened footings, mat foundations, or fill overexcavation and replacement. In general, fill overexcavation and recompaction would likely be a more cost effective and reliable approach to mitigating the existing undocumented fill. Mat slabs may be appropriate for buildings encompassing large footprints over undocumented fills, depending on the fill quality and structural loading of the building. We can provide design recommendations and settlement analysis for alternative foundation systems on a case-by-case basis, if requested.

Relatively loose fill, within and a minimum of 5 feet beyond the proposed structure footprints, shall be overexcavated and stockpiled onsite. The depth of the overexcavation should extend through all loose soil to competent native soil or rock. The fill shall be replaced and compacted using the recommendations presented in the Fill Placement section of this report. Deep fill that has not been tested and recorded may only need to be overexcavated 4-5 feet as the effective stress below these depths may be less than the surcharge of any buildings placed on the fill. We can address these areas when final grading plans and building footprints are available.

5.1.4 Cut Slope Grading

Based on our understanding of the project at this time, we anticipate that permanent cut slopes up to approximately 20 feet in height will be created during grading of the proposed improvements. In general, permanent cut slopes should not be steeper than 2:1, horizontal to vertical (H:V). Steeper cut slopes as steep as 1:1, H:V, may be feasible, depending on the soil/rock conditions encountered and slope heights, and should be reviewed on a case-by-case basis. The upper two feet of all cut slopes should be graded to an approximate 2:1, H:V, slope to reduce sloughing and erosion of looser surface soil.

Temporary cut slopes may be constructed to facilitate construction of retaining walls and deep industrial mining infrastructure. We anticipate that subsurface conditions will be favorable for construction of temporary cut slopes no steeper than ½:1, H:V, for a maximum height of approximately 6 feet. To reduce the likelihood of sloughing or failure, temporary cut slopes should not remain over the winter.

A representative of NV5 must observe temporary cut slopes steeper than 1.5:1, H:V, during grading to confirm the soil and rock conditions encountered. We recommend that personnel not be allowed between the cut slope and the proposed retaining structure, form work, grading equipment, or parked vehicles during construction, unless the stability of the slope has been reviewed by NV5 or the slope has been confirmed to meet OSHA excavation standards. All temporary excavations must comply with applicable local, state and federal safety regulations, including the current Occupational Safety and Hazards Administration (OSHA) excavation and trench safety standards. Construction site safety is the responsibility of the contractor, who is solely responsible for the means, methods and sequencing of construction operations.

5.1.5 Native Soil Preparation for Engineered Fill Placement

After completing site stripping and grubbing activities, the exposed native soil should be prepared for placement and compaction of engineered fill, as described below.

1. The native soil should be scarified to a minimum depth of 8 inches below the existing land surface, or stripped and grubbed surface, and then uniformly moisture conditioned. If the soil is classified as a coarse-grained soil by the USCS (i.e., GP, GW, GC, GM, SP, SW, SC or SM) then it should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content. If the soil is classified as a low plasticity fine-grained soil by the USCS (i.e., CL, ML), then it should be moisture conditioned to between 2 and 4 percentage points greater than the ASTM D1557 optimum moisture content. If soil is classified as a high plasticity fine-grained soil by the USCS (i.e., CH, MH), the soil should be removed from the building pad area or NV5 should be contacted for further recommendations.

2. The native soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry unit weight (density). The moisture content, density and relative percent compaction should be tested by the project engineer or his/her field representative to evaluate whether the compacted soil meets or exceeds the minimum percent compaction and moisture content requirements. The earthwork contractor shall assist the project engineer or his/her field representative by excavating test pads with the on-site earth moving equipment. Native soil preparation beneath concrete slab-on-grade structures (i.e., floors, sidewalks, patios, etc.) and asphalt concrete (AC) pavement should be prepared as specified in Section 5.2 (“Structural Improvements”).

3. Where fill placement is proposed on native slopes steeper than approximately 5:1, H:V, a base key and routine benches must be provided. Unless otherwise recommended by the project geotechnical engineer, the base key should be excavated at the toe of the fill a minimum of 3 feet into competent stratum, as determined by a representative of NV5 during construction observation. The bottom of the base key should be sloped slightly into the hillside at an approximate gradient of 5 percent or greater.

4. A subdrain should be installed at the rear of the keyway and where evidence of seepage is observed. The subdrain should consist of a 4-inch diameter (minimum) perforated plastic pipe embedded in drain rock material wrapped in woven geotextile filter fabric. The drain rock material should be at least 12 inches thick and extend at least 48 inches above the bottom of the keyway and/or 12 inches above and below the seepage zone. The depth and extent of subdrains should be determined by a representative of H&K/NV5 in the field during construction. In addition, subdrains should be installed at a minimum slope of 1 percent and should have cleanouts located at their ends and at turning points. Outlet and riser pipe fittings should not be perforated. A licensed land surveyor or civil engineer should provide “record drawings” depicting the locations of subdrains and cleanouts.

   The fill must be benched into existing side slopes as fill placement progresses. Benching must extend through loose surface soil into firm material, and at intervals such that no loose surface soil is beneath the fill. As a minimum, a horizontal bench should be excavated every 5 vertical feet or as determined by a representative of NV5.

5. The prepared native soil surface should be proof-rolled with a fully-loaded 4,000-gallon-capacity water truck with the rear of the truck supported on a double-axle, tandem-wheel undercarriage or approved equivalent. The proof-rolled surface should be visually observed by the project engineer or his/her field representative to be firm, competent and relatively unyielding. The project engineer or his/her field representative may also evaluate the surface material by hand probing with a ¼-inch-diameter steel probe, however, this evaluation method should not be performed in place of proof rolling as described above.

6. Construction Quality Assurance (CQA) tests should be performed using the minimum testing frequencies presented in Table 5.1.6-1 or as modified by the project engineer to better suit the site conditions.

The native soil surface should be graded to minimize ponding of water and to drain surface water away from the building foundations and associated structures. Where possible, surface water should be collected, conveyed and discharged into natural drainage courses, storm sewer inlet structures, permanent engineered storm water runoff percolation/evaporation basins or engineered infiltration subdrain systems.

Table 5.1.6-1, Minimum Testing Frequencies

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 1,500 CY or Material Change (2)
D6938	Nuclear Density and Nuclear Moisture Content	1 per 250 CY

Notes:
(1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) Whichever criteria provide the greatest number of tests.
ASTM = ASTM International
CY = cubic yards
No. = number

5.1.6 Fill Placement

Construction of engineered fills with non-expansive soil should be performed as described below.

1. Non-expansive soil used as engineered fill should consist predominantly of materials less than ½-inch in greatest dimension and should not contain rocks greater than 6 inches in greatest dimension (oversized material). Non-expansive soil should have a plasticity index (PI) of less than or equal to 15, as determined by ASTM D4318 Atterberg Indices testing. Oversized materials should be spread apart to prevent clustering so that void spaces are not created. The project engineer or his/her field representative should approve the use of oversized materials for constructing engineered fills. Import material that is proposed for use onsite should be submitted to NV5 for approval and possible laboratory testing at least 72 hours prior to transport to the site.
2. Non-expansive soil used to construct engineered fills should be uniformly moisture conditioned. If the soil is classified by the USCS as coarse grained (i.e., GP, GW, GC, GM, SP, SW, SC or SM), then it should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content. If the soil is classified by the USCS as fine grained (i.e., CL, ML), then it should be moisture conditioned to between 2 and 4 percentage points greater than the ASTM D1557 optimum moisture content.
3. Cohesive, predominantly fine grained, or potentially expansive soil encountered during grading should be stockpiled for removal, mixed as directed by NV5, or used in landscape areas. As an option, cohesive fine grained, or potentially expansive soil can often be placed in the deeper portions of proposed fill (e.g., depths greater than 3 feet below subgrade in building footprints). However, this option would have to be evaluated on a case-by-case basis with consideration of the fill depth and proposed loading.
4. In areas where expansive soils are present, the upper 24 inches of fill beneath and within 5 feet of the building footprints and the upper 12 inches of fill beneath and within 3 feet of exterior slabs and/or pavement edges should be approved engineered fill.
5. Engineered fills should be constructed by placing uniformly moisture conditioned soil in maximum 12-inch-thick loose lifts (layers) prior to compacting.
6. The soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

7. The earthwork contractor should compact each loose soil lift with a tamping foot compactor such as a Caterpillar (CAT) 815 Compactor or equivalent as approved by NV5's project engineer or his/her field representative. A smooth steel drum roller compactor should not be used to compact loose soil lifts for construction of engineered fills.
8. The field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 5.1.6-1 or as modified by the project engineer to better suit the site conditions.

Table 5.1.6-1, Minimum Testing Frequencies for Non-Expansive Soil

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 1,500 CY or Material Change (2)
D6983	Nuclear Moisture and Density	1 per 250 CY

Notes:
(1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) Whichever criteria provide the greatest number of tests.
ASTM = ASTM International
CY = cubic yards
No. = number

9. The moisture content, density and relative percent compaction of all engineered fills should be tested by the project engineer's field representative during construction to evaluate whether the compacted soil meets or exceeds the minimum compaction and moisture content requirements. The earthwork contractor shall assist the project engineer's field representative by excavating test pads with the on-site earth-moving equipment.
10. The prepared finished grade or finished subgrade soil surface should be proof-rolled, as mentioned above in Section 5.1.5, Paragraph 6.

5.1.7 Differential Fill Depth/Cut-Fill Transitions

The recommendations presented in this section are intended to reduce the magnitude of differential settlement-induced structural distress associated with variable fill depth beneath structures. Care should be taken when removing existing foundations and re-routing underground utilities so that large excavations are not opened which could inadvertently result in differing soil conditions between native soil and utility backfill that could be subject to differential settlement.

1. Site grading should be performed so that cut-fill transition lines do not occur directly beneath any structures. The cut portion of the cut-fill building pads, if proposed, should be scarified to a minimum depth of 8 inches, and recompacted to 95 percent relative compaction.
2. Differential fill depths beneath structures should not exceed 5 feet. For example, if the maximum fill depth is 8 feet across a building pad, the minimum fill depth beneath that pad should not be less than 3 feet. If a cut-fill building pad is used in this example, the cut portion would need to be overexcavated 3 feet and rebuilt with compacted fill.
3. If cut/fill transitions within any building footprint are greater than 5 feet are planned, or demolition requires deep and wide excavations, NV5 should be notified so that additional recommendations to properly construct the fill pad beneath the project location can be provided to ensure that a cut-fill transition is not constructed that may be subject to differential settlement in the future.

5.1.8 Rock Fill Placement

Based on our observation of the rocky nature of the subsurface conditions revealed in our exploratory trenches, we anticipate that fill material generated from the subject site may contain significant rock fragments, and that compaction testing with conventional methods may be difficult or inappropriate. Typically, fill that consists primarily of soil can be tested for relative compaction by using a nuclear density gauge. Our opinion is that rock fill cannot be reliably tested using this method.

We recommend that quality assurance during rock fill placement be based on a procedural approach, or method specification, rather than a specified relative compaction. The procedural requirements will depend on the equipment used, as well as the nature of the fill material, and will need to be determined by the geotechnical engineering firm onsite. Typically, procedural recommendations are based on the measured relative compaction of a test fill constructed onsite.

Based on our experience in the area, we anticipate that the procedural specification will require a minimum of six passes (back and forth equaling one pass) with a Cat 563 or similar, self-propelled, vibratory compactor to compact a maximum 8-inch thick, loose lift. Processing or screening of the fill material will be needed to remove rocks larger than approximately 8 inches in maximum dimension. Continuous or nearly continuous observation by a representative of NV5 would be required during fill placement to confirm that procedural specifications have been met.

5.1.9 Fill Slope Grading

Based on our understanding of the project, we anticipate that fill slopes up to approximately 90 feet in height will be created as part of the proposed improvements. In general, permanent fill slopes created onsite should be no steeper than 2:1, H:V. Fills designed on native slopes steeper than 5:1, H:V, should be supported by a base shear keyway, as described in Section 5.1.5 (“Native Soil Preparation for Engineered Fill Placement”) of this report. Steeper fill slopes may be feasible based on the angularity and durability of the material to be placed or with the use of geotextile reinforcement and/or rock facing. All fill slopes greater than 30 feet in height should be terraced with surface drains that restrict surface runoff from traveling more than 30 feet continuously down the fill slope face. NV5 should review fill slope configurations greater than approximately 10 feet in height, if proposed, prior to fill placement. We can provide slope terracing and drainage recommendations or reinforced/buttressed fill slope design for the project, if requested.

Fill should be placed in horizontal lifts to the lines and grades shown on the project plans. Compaction and fill slope grading must be confirmed by NV5 in the field. Slopes should be constructed by overbuilding the slope face and then cutting it back to the design finished grade slope gradient. Fill slopes should not be constructed or extended horizontally by placing soil on an existing slope face and/or compacted by track walking. Where placement of oversized rock in deep fill is proposed, the oversized rock should be placed a minimum of 5 feet horizontally from the finished fill slope face.

5.1.10 Erosion Controls

Graded portions of the site should be seeded as soon as possible to allow vegetation to become established prior to and during the rainy season. In addition, grading that results in greater than one acre of soil disturbance or in sensitive areas may require the preparation of a site-specific storm water pollution prevention plan. As a minimum, the following controls should be installed prior to and during grading to reduce erosion.

1. Prior to commencement of site work, fiber rolls should be installed down slope of the proposed area of disturbance to reduce migration of sediment from the site. Fiber rolls on slopes are intended to reduce sediment discharge from disturbed areas, reduce the velocity of water flow, and aid in the overall revegetation of slopes. The fiber rolls should remain in place until construction activity is complete and vegetation becomes established.
2. All soil exposed in permanent slope faces should be hydroseeded or hand seeded/strawed with an appropriate seed mixture compatible with the soil and climate conditions of the site as recommended by the local Resource Conservation District.
3. Following seeding, jute netting or erosion control blankets should be placed and secured over the slopes steeper than 2:1, H.V.
4. Surface water drainage ditches should be established as necessary to intercept and redirect concentrated surface water away from cut and fill slope faces. Under no circumstances should concentrated surface water be directed over slope faces. The intercepted water should be discharged into natural drainage courses or into other collection and disposal structures.
5. NV5 should be retained to review final grading plans for Site improvements to confirm that appropriate slope drainage and erosion control measures have been included in the project design.

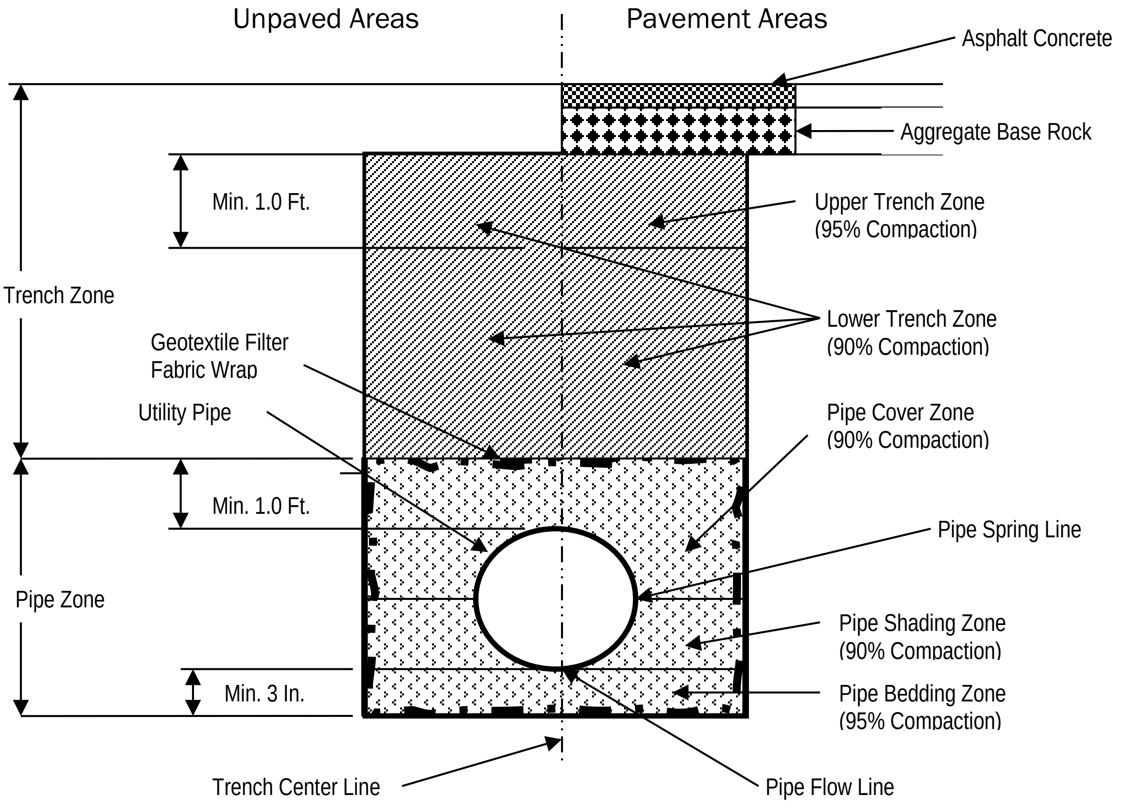
5.1.11 Underground Utility Trenches

Underground utility trenches should be excavated and backfilled as described below.

Underground utility trenches should be excavated and backfilled as described below for each trench zone shown in the figure below.

1. Trench Excavation Equipment: NV5 anticipates that the contractor will be able to excavate relatively shallow (up to 5 ft bgs) underground utility trenches with a Case 580 Backhoe or equivalent. Highly resistant rock, not described in our investigation, may be encountered in deeper trench excavations.
2. Trench Shoring: All utility trenches that are excavated deeper than 4 feet bgs are required by California OSHA to be shored with bracing equipment or sloped back to an appropriate slope gradient prior to being entered by any individuals.
3. Trench Dewatering: NV5 does not anticipate that the proposed underground utility trenches will encounter shallow groundwater. However, if the utility trenches are excavated during the winter rainy season, then shallow or perched groundwater may be encountered. The earthwork contractor may need to employ de-watering methods as discussed in Section 5.1.12 in order to excavate, place and compact the trench backfill materials.
4. Pipe Zone Backfill Type and Compaction Requirements: The backfill material type and compaction requirements for the pipe zone, which includes the bedding zone, the shading zone and the cover zone, are described in Detail 5.1.11-1 below. It is possible that waste rock/sand from the mine processing would be feasible for use as trench backfill and bedding. If proposed, select backfill material should be approved by NV5 prior to use.

Detail 5.1.11-1 TYPICAL TRENCH BACKFILL ZONES



Not to Scale

- Pipe Zone Backfill Material Type: Trench backfill used within the pipe zone, which includes the bedding zone, the shading zone and the cover zone, should consist of ¾-inch-minus, washed, crushed rock. The crushed rock particle size gradation should meet the following requirements (percentages are expressed as dry weights using ASTM D422 test method): 100 percent passing the ¾-inch sieve, 80 to 100 percent passing the ½-inch sieve, 60 to 100 percent passing the 3/8-inch sieve, 0 to 30 percent passing the No. 4 sieve, 0 to 10 percent passing the No. 8 sieve, and 0 to 3 percent passing the No. 200 sieve.

  If groundwater is encountered within the trench during construction, or if groundwater is expected to rise during the rainy season to an elevation that will infiltrate the pipe zone within the trench, then the pipe zone material should be wrapped with a minimum 6 ounce per square yard, non-woven geotextile filter fabric such as TenCate® Mirifi N140 or an approved equivalent. The geotextile seam should be located along the trench centerline and have a minimum 1-foot overlap. If the utility pipes are coated with a corrosion protection material, then the pipes should be wrapped with a minimum 6 ounce per square yard, non-woven, geotextile cushion fabric such as TenCate® Mirifi N140 or an approved equivalent. The geotextile cushion fabric should have a minimum 6-inch seam overlap. The geotextile cushion fabric will protect the pipe from being scratched by the crushed rock backfill material.

- Pipe Bedding Zone Compaction: Trench backfill soil placed in the pipe bedding zone (beneath the utilities) should be a minimum of 3 inches thick, moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density. Crushed rock should be mechanically consolidated under the observation of NV5.
- Pipe Shading Zone Compaction: Trench backfill soil placed within the pipe shading zone (above the bedding zone and to a height of one pipe radius above the pipe spring line) should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. Crushed rock should be mechanically consolidated under the observation of NV5. The pipe shading zone backfill material should be shovel-sliced to remove voids and to promote compaction.
- Pipe Cover Zone Compaction: Trench backfill soil placed within the pipe cover zone (above the pipe shading zone to 1 foot over the pipe top surface) should be moisture conditioned to within ± 3percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. Crushed rock should be mechanically consolidated under the observation of NV5.

5. Trench Zone Backfill and Compaction Requirements: The trench zone backfill materials consist of both lower and upper zones, as discussed below.

    - Trench Zone Backfill Material Type: Soil used as trench backfill within the lower and upper intermediate zones, as shown on the preceding figure, should consist of non-expansive soil with a PI of less than or equal to 15 (based on ASTM D4318) and should not contain rocks greater than 3 inches in greatest dimension.
    - Lower Trench Zone Compaction: Soil used to construct the lower trench zone backfills should be uniformly moisture conditioned to within 0 and 4 percentage points of the ASTM D1557 optimum moisture content, placed in maximum 12-inch-thick loose lifts prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
    - Upper Trench Zone Compaction (Road and Parking Lot Areas): Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 and 4 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 8-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 95percent of the ASTM D1557 maximum dry density.
    - Upper Trench Zone Compaction (Non-Road and Non-Parking Lot Areas): Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 and 2 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 6-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

6. CQA Testing and Observation Engineering Services: The moisture content, dry density and relative percent compaction of all engineered utility trench backfills should be tested by the project engineer’s field representative during construction to evaluate whether the compacted trench backfill materials meet or exceed the minimum compaction and moisture content requirements presented in this report. The earthwork contractor shall assist the project engineer’s field representative by excavating test pads with the on-site earth moving equipment.

- Compaction Testing Frequencies: The field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 5.1.11-2 or as modified by the project engineer to better suit the site conditions.

Table 5.1.11-2, Minimum Testing Frequencies for Utility Trench Backfill

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 500 CY (2) Or Material Change
D6983	Nuclear Moisture and Density	1 per 100 LF per 24-Inch-Thick Compacted Backfill Layer (2) The maximum loose lift thickness shall not exceed 12-inches prior to compacting.

Notes:
(1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) Whichever criteria provide the greatest number of tests.
ASTM = ASTM International
CY = cubic yards
LF = linear foot
No. = number

7. Final Proof Rolling: The prepared finished grade AB rock surface and/or finished subgrade soil surface of utility trench backfills should be proof-rolled, as mentioned above in Section 5.1.5, Paragraph 6.

5.1.12 Construction Dewatering

NV5 does not anticipate the need to perform de-watering of the Site during earthwork grading; however, the earthwork contractor should be prepared to de-water the utility trench excavations and any other excavations if perched water or the groundwater table is encountered during grading. NV5 recommends that the utility trench excavations be performed as late in the summer months as possible to allow the groundwater table to reach its lowest seasonal elevation. Perched groundwater may be encountered on low permeability soil or weathered rock layers even during the summer months.

The following recommendations are preliminary and are not based on performing a groundwater flow analysis. A detailed de-watering analysis was not a part of the proposed work scope. It should be understood that it is the earthwork contractor's sole responsibility to select and employ a satisfactory de-watering method for each excavation.

1. NV5 anticipates that de-watering of utility trenches can be performed by constructing sumps to depths below the trench bottom and removing the water with sump pumps.
2. Additional sump excavations and pumps should be added as necessary to keep the excavation bottom free of standing water and relatively dry when placing and compacting the trench backfill materials.
3. If groundwater enters the trench faster than it can be removed by the de-watering system, thereby allowing the underlying compacted soil to become unstable while compacting successive soil lifts, then it may be necessary to remove the unstable soil and replace it with free-draining, granular drain rock. Native backfill soil can again be used after placing the granular rock to an elevation that is higher than the groundwater table.

4. If granular rock is used, it should be wrapped in a non-woven geotextile fabric, such as TenCate® Mirifi® N140 or an approved equivalent. The geotextile filter fabric should have minimum 1-foot overlapped seams. The granular rock should meet or exceed the following gradation specifications (all percentages are expressed as dry weights using ASTM D422 test method): 100 percent passing the 3/4-inch sieve, 80 to 100 percent passing the 1/2-inch sieve, 60 to 100 percent passing the 3/8-inch sieve, 0 to 30 percent passing the No. 4 sieve, 0 to 10 percent passing the No. 8 sieve, and 0 to 3 percent passing the No. 200 sieve.

If subsurface seepage or groundwater conditions are encountered which prevent or restrict fill placement or construction of the proposed improvements, subdrains may be necessary. If groundwater or saturated soil conditions are encountered during grading, we should be retained to observe the conditions and provide site specific subsurface drainage recommendations.

5.1.13 Soil Corrosion Potential

Based on our review of soil survey information, the near-surface soil conditions in the South Fork of Wolf Creek valley and the previously developed New Brunswick shaft and saw mill landings in the northeastern section of the Site possess a low corrosion potential for uncoated steel and concrete. The soil survey indicates that the near-surface soil conditions in the northwestern section of the Property possess a moderate to high corrosion potential for uncoated steel and low to moderate corrosion potential for concrete.

Index testing of the near-surface soils were performed as part of our soil evaluation in an effort to evaluate corrosion potential. The measured minimum resistivity values indicate that the on-site soil conditions exhibit a “moderate“ risk for corrosion of ferrous metals in contact with the soil or rock. The presence of high acidity, pH of 5.5 or less, in the soil samples is considered corrosive to concrete. Soil with a pH of 5.5 or less can react with the lime in concrete to form soluble reaction products that can easily leach out of the concrete. The result is a more porous, weaker concrete.

To reduce the likelihood of corrosion problems, materials used for underground utilities, permanent subsurface drainage improvements, and foundation systems should be selected based on local experience and practice. If alternative or new construction methods or materials are being proposed, it may be appropriate to have the selected materials evaluated by a corrosion engineer for compatibility with the onsite soil and groundwater conditions.

5.1.14 Surface Water Drainage

Proper surface water drainage is important to the successful development of the project. We recommend the following measures to help mitigate surface water drainage problems:

1. Slope final grades in structural areas so that surface water drains away from building pad finish subgrade at a minimum 2 percent slope for a minimum distance of 10 feet. For structures utilizing slab-on-grade interior floor systems we recommend increasing the slope to 4 percent.

2. To reduce surface water infiltration, compact and slope all soil placed adjacent to building foundations such that water is not allowed to pond. Backfill should be free of deleterious materials.

3. Direct downspouts to positive drainage or a closed collector pipe that discharges flow to positive drainage.

4. Construct V-ditches at the top of cut and fill slopes where necessary to reduce concentrated surface water flow over slope faces. Typically, V-ditches should be 3 feet wide and at least 6 inches deep. Surface water collected in V-ditches should be directed away and downslope from proposed building pads and driveways into a drainage channel.

5.1.15 Grading Plan Review and Construction Monitoring

Construction quality assurance includes review of plans and specifications and performing construction monitoring as described below.

1. NV5 should be retained to review the final grading plans prior to construction to confirm our understanding of the project at the time of our investigation, to determine whether our recommendations have been implemented, and to provide additional and/or modified recommendations, if necessary.
2. Prior to commencement of a new phases of development on the Property, H&K/NV5 should be retained to observe the soil/rock conditions within and surrounding the proposed improvements to confirm or modify our recommendations. A preconstruction meeting with the contractor and subcontractors involved should be held to discuss and review the applicable recommendations of this report as they apply to the proposed construction.
3. NV5 should be retained to perform construction quality assurance (CQA) monitoring of all earthwork grading performed by the contractor to determine whether our recommendations have been implemented, and if necessary, provide additional and/or modified recommendations.
4. NV5's experience, and that of the engineering profession, clearly indicates that during the construction phase of a project the risks of costly design, construction and maintenance problems can be significantly reduced by retaining a design geotechnical engineering firm to review the project plans and specifications and to provide geotechnical engineering observation and CQA testing services. Upon your request we will prepare a CQA geotechnical engineering services proposal that will present a work scope, a tentative schedule and a fee estimate for your consideration and authorization. If NV5 is not retained to provide geotechnical engineering CQA services during the construction phase of the project, then NV5 will not be responsible for geotechnical engineering CQA services provided by others nor any aspect of the project that fails to meet your or a third party's expectations in the future.

5.2 STRUCTURAL IMPROVEMENT DESIGN CRITERIA

The following sections present our structural improvement design criteria and recommendations. The recommendations address foundations, seismic parameters, concrete slabs-on-grade, retaining walls and pavement design.

5.2.1 Seismic Design Criteria

Our classification of on-site soil conditions is based on field observations and laboratory tests. The on-site soil primarily consists of medium dense shallow granular soil composed of silty or clayey sand, with firm and competent fine-grained silt or clay encountered at shallow depths. Based on the presence of predominantly granular soil and firm and competent soil/rock at relatively shallow depths, we classified the on-site soil as very dense soil and soft rock (Site Class C) for design purposes.

Table 5.2.1-1 below summarizes seismic design criteria based on ASCE 7-16, the 2019 California Building Code and the California Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps tool (available online at https://seismicmaps.org/).

Table 5.2.1-1, Seismic Design Parameters

Description	Value	Reference	Description	Value	Reference
Latitude Longitude	39.2095 -121.0165	1	Site Class	C	2
Site Coefficient, F_A	1.276	5	Site Coefficient, F_V	1.5	6
Short (0.2 sec) Spectral Response, S_S	0.560 g	3	Long (1.0 sec) Spectral Response, S_1	0.233 g	4
S_S modified for Site Class Effects, S_{MS}	0.714 g	7	S_1 modified for Site Class Effects, S_{M1}	0.349 g	8
Design Spectral Response Acceleration, Short Periods, S_{DS}	0.476 g	9	Design Spectral Response Acceleration, Long Periods, S_{D1}	0.233 g	10

References:

1) *https://seismicmaps.org/*
2) *ASCE 7-16 Table 20.3-1*
3) *ASCE 7-16 Figure 22-1*
4) *ASCE 7-16 Figure 22-2*
5) *ASCE 7-16 Table 11.4-1*
6) *ASCE 7-16 Table 11.4-2*
7) *ASCE 7-16 Equation 11.4-1*
8) *ASCE 7-16 Equation 11.4-2*
9) *ASCE 7-16 Equation 11.4-3*
10) *ASCE 7-16 Equation 11.4-4*

5.2.2 Foundations

Provided that the grading for the project is performed in accordance with the recommendations presented in this report, our opinion is that the site will be suitable for the use of conventional perimeter foundations, isolated interior footings, and interior slabs-on-grade. Following are our recommendations for foundations constructed on compacted and tested fill or competent native soil:

1. In areas where expansive soils are present, we recommend an 18-inch blanket of import granular fill be placed within and 5 feet beyond the building pad footprint.
2. Footings for single story structures should be a minimum of 12 inches wide and trenched through any loose surface material, potentially expansive soil, or untested fill, and a minimum of 12 inches into competent native soil, weathered rock or compacted fill. Footings for two-story structures, if proposed, should be a minimum of 15 inches wide and trenched a minimum of 18 inches into competent native soil, weathered rock or compacted fill. If clay is encountered at the base of footing excavations, the footing should be deepened through the clay into underlying granular material or weathered rock, as determined in the field by NV5.
3. The base of the footing excavation should be approximately level. On sloping sites, it will be necessary to step the base of the footing excavation as necessary to maintain a slope of less than 10 percent at the base of the footing.
4. Footing trenches should be cleaned of all loose soil and construction debris prior to placing concrete. A representative from NV5 should observe the footing excavations prior to concrete placement.
5. As a minimum, the footings should be designed with two No. 4 rebar reinforcement, one near the top of the footing and one near the bottom. A minimum of 3 inches of concrete coverage should surround the bars.

6. The concrete should have a minimum 2,500 pounds per square inch compressive break strength after 28 days of curing (unless specified by a structural engineer), have a water-to-cement ratio from 0.40 to 0.50, and should be placed with minimum and maximum slumps of 4 and 6 inches, respectively. Since water is often added to uncured concrete to increase workability, it is important that strict quality control measures be employed during placement of the foundation concrete to ensure that the water-to-cement ratio is not altered prior to or during placement.

7. Prior to placing concrete in any foundation excavation, the project geotechnical engineer or his/her field representative should observe the excavations to document that the following requirements have been achieved: minimum foundation dimensions, minimum reinforcement steel placement and dimensions, removal of all loose soil, rock, wood debris or other deleterious materials, and that firm and competent native or engineered fill soil is exposed along the entire foundation excavation bottom. Strict adherence to these requirements is paramount to the satisfactory behavior of a building foundation. Minor deviations from these requirements can cause the foundations to undergo minor to severe amounts of settlement which can result in cracks developing in the foundation and adjacent structural members, such as concrete slab-on-grade floors.

8. In general, structures constructed adjacent to descending slopes should employ a minimum setback of either 1/3 the height of the slope, or 40 feet, whichever is less. The setback for ascending slopes is either 1/2 the slope height or 15 feet, whichever is less. Where footings are proposed within these code-based setbacks, the project geotechnical engineer should review the proposed slope configuration and provide revised setback recommendations, if appropriate.

9. Footing excavations should be saturated prior to placing concrete to reduce the risk of problems caused by wicking of moisture from curing concrete. However, concrete should not be placed through standing water in the footing excavations.

10. In an effort to reduce the likelihood of settlement-induced distress to the proposed structures, we recommend that strip and isolated footings with a minimum embedment depth of 12 inches in competent soil be sized for an allowable bearing capacity of 3,000 psf for dead plus live loads. This value can be increased by 300 psf for each additional foot of embedment up to a limiting value of 3,900 psf. Allowable bearing may be increased by 33 percent for additional transient loading, such as wind or seismic loads.

11. A triangularly-distributed lateral resistance (passive soil resistance) of 350d psf, where d is footing depth, may be used for footings. This value may be increased by 33 percent for wind and seismic. As an alternate to the passive soil resistance described above, a coefficient of friction for resistance to sliding of 0.40 may be used. The higher of the two values should be reduced by 50 percent if both resisting values are to be used.

12. Total settlement of individual foundations will vary depending on the plan dimensions of the foundation and actual structural loading. Based on anticipated foundation dimensions and loads, we estimate that total post-construction settlement of footings designed and constructed in accordance with our recommendations will be on the order of one-half inch. Differential settlement between similarly loaded, adjacent footings is expected to be less than one-quarter inch, provided footings are founded on similar materials (e.g., all on structural fill, native soil or rock). Differential settlement between adjacent footings founded on dissimilar materials (e.g., one footing on soil and an adjacent footing on rock) may approach the maximum anticipated total settlement. Settlement of foundations is expected to occur rapidly and should be essentially complete shortly after initial application of loads.

13. Heavily loaded foundations should be evaluated on a case by case basis. We can provide Cast-in-drilled hole or mat foundations, if requested.

5.2.3 Slab-on-Grade Floor Systems

Our opinion is that interior concrete slab-on-grade floors may be used in conjunction with perimeter concrete foundations for the proposed improvements. The project structural engineer should design slabs-on-grade with regard to the anticipated loading. This section presents typical slab sections and reinforcement schedules used for residential construction in the region and presents construction recommendations. We can provide project specific slab-on-grade design for the proposed improvements once anticipated loading and serviceability criteria have been established.

The heavily loaded concrete slab on grade building floors and driveway areas should be evaluated by a California-licensed civil engineer for expected live and dead loads to determine if the minimum slab thickness and steel reinforcement recommendations presented in this report should be increased or redesigned.

NV5 recommends using the guideline procedures, methods and material properties that are presented in the following ASTM and ACI documents for construction of concrete slab on grade floors:

- ACI 302.1R 04, Guide for Concrete Floor and Slab Construction, reported by ACI Committee 302.
- ASTM E1643 98 (Reapproved 2005), Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs.
- ASTM E1745 97 (Reapproved 2004), Standard Specifications for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs.
- ASTM F710 5, Standard Practice for Preparing Concrete Floors to Receive Resilient Flooring.

The interior building concrete slab-on-grade floor and exterior shop, sidewalk and patio concrete slab-on-grade floor components are described below from top to bottom.

1. Minimum 4-Inch Thick Concrete Slab: The concrete slab should be installed with a minimum 3,000 pounds per square inch (psi) compressive strength after 28 days of curing. NV5 recommends that the concrete design use a water-to-cement ratio between 0.40 and 0.45 and should be placed with minimum and maximum slumps of 3 and 5 inches, respectively. The concrete mix design is the responsibility of the concrete supplier. If floor loads higher than 250 psf or intermittent live loads are anticipated, a structural engineer should determine the slab thickness and steel reinforcing schedule.

2. Steel Reinforcement: Reinforcement should be used to improve the load-carrying capacity, to reduce cracking caused by shrinkage during curing and from both differential and repeated loadings. It should be understood that it is nearly impossible to prevent all cracks from development in concrete slabs; in other words, it should be expected that some cracking will occur in all concrete slabs no matter how well they are reinforced. Concrete slabs that will be subjected to heavy loads should be designed with steel reinforcements by a California-licensed structural engineer.

3. Rebar: As a minimum, use No. 3 rebar (ASTM A615/A 615M 04 Grade 60), tied and placed with 18 inch centers in both directions (perpendicular) and supported on concrete “dobies” to position the rebar in the center of the slab during concrete pouring. NV5 does not recommend that the steel reinforcements of the concrete slab on grade floor be tied into the perimeter or interior continuous strip foundations or interior isolated column foundations. In other words,

we recommend that the concrete slab on grade floors be constructed as independent structural members so that they can move (float) independently from the foundation structures. We do not recommend using rolls of WWM because vertically centered placement of rolled mesh within the slab is difficult to achieve. All rebar and sheets of WWM should be placed in the center of the slab and supported on concrete "dobies". We do not recommend "hooking and pulling" of steel during concrete placement.

4. Underslab Vapor Moisture Retarder Membrane: The underslab retarder membrane should be placed in areas with moisture sensitive floor coverings as a floor component that will minimize transmission of both liquid water and water vapor transmission through the concrete slab on grade floor. NV5 recommends using at a minimum a Class A (ASTM E1745 97 [Reapproved 2004]), minimum 10 mil thick, plastic, vapor moisture, retarder membrane material such as Stego Wrap® underslab vapor retarder membranes or equivalents. Additionally, the following materials are recommended: Stego® Tape and Stego® Mastic or equivalents to seal membrane joints and any utility penetrations.

   Regardless of the type of moisture vapor retarder membrane used, moisture can wick up through a concrete slab on grade floor. Excessive moisture transmission through a concrete slab floor can cause adhesion loss, warping and peeling of resilient floor coverings, deterioration of adhesive, seam separation, formation of air pockets, mineral deposition beneath flooring, odor and both fungi and mold growth. Slabs can be tested for water transmissivity in areas that are moisture sensitive. Commercial sealants, polymer additives to the concrete at the batch plant, entrained air, flyash, and a reduced water-to-content ratio can be incorporated into the concrete slab on grade floor mix design to reduce its permeability and water vapor transmissivity properties. A waterproofing consultant should be contacted to provide detailed recommendations if moisture sensitive flooring materials will be installed on the concrete slab on grade floors.

   Prior to placing the vapor retarder and concrete, slab subgrade soil must be moisture conditioned to between 75 and 90 percent saturation to a depth of 24 inches. Moisture conditioning should be performed for a minimum of 24 hours prior to concrete placement. Clayey soil may take up to 72 hours to reach this required degree of saturation. If the soil is not moisture conditioned prior to placing concrete, moisture will be wicked out of the concrete, possibly contributing to shrinkage cracks. Additionally, our opinion is that moisture conditioning the soil prior to placing concrete will reduce the likelihood of soil swell or heave following construction at locations where fine grained, potentially expansive soil is encountered. To facilitate slab-on-grade construction, we recommend that the slab subgrade soil be moisture conditioned following rock placement. Following moisture conditioning, the vapor retarder should be placed.

5. Minimum 4-Inch Thick Crushed Rock or Class II Aggregate Base Rock Layer: Interior floors should be underlain by clean crushed rock, while exterior floors should use either crushed rock or Class II AB rock. Crushed rock should be mechanically consolidated under the observation of NV5. AB rock layers should be placed and compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. The crushed rock should be washed to produce a particle size distribution of 100 percent (by dry weight) passing the ¾ inch sieve and 5 percent passing the No. 4 sieve and 0 to 3 percent passing the No. 200 sieve. An alternative rock material for external slab-on-grade concrete surfaces would include AB rock meeting the specification of Caltrans Class II AB. Just prior to pouring the concrete slab, the rock layer should be moistened to a saturated surface dry (SSD) condition. This measure will reduce the potential for water to be withdrawn from the bottom of the concrete slab while it is curing and will help minimize the development of shrinkage cracks.

If the current property owner elects to eliminate the crushed rock or AB rock layer beneath the interior and exterior concrete slabs on grade for economic reasons, then there will be an inherent greater risk assumed by the developer for the development of both shrinkage and bearing-related cracks in the associated slabs.

6. Subgrade Soil Preparation: In general, the subgrade soil around the slabs-on-grade should be sloped away from the proposed slab subgrade a minimum of 4 percent for a distance of 10 feet as discussed in the Surface Water Drainage section of this report. NV5 recommends that subgrade elevations on which the concrete slab on grade floors are constructed be a minimum of 6 inches above the elevation of the surrounding parking lots, driveways and landscaped areas. Elevating the building will reduce the potential for subsurface water to enter beneath the concrete slab on grade floors and exterior surfaces and underground utility trenches. A representative from NV5 should observe pad and subgrade elevations prior to forming the slab footings.

   The subgrade soil should be prepared and compacted consistent with the recommendations of Section 8.1. All deleterious material must be removed prior to placing concrete. The top 12 inches of the non-expansive soil should be compacted to a minimum of 90 percent of the ASTM D1557 dry density with relatively uniform moisture content within 3 percentage points of the ASTM D1557 optimum moisture content.

7. Crack Control Grooves: Crack control grooves should be installed during placement or saw cuts should be made in accordance with the ACI and Portland Cement Association (PCA) specifications. Generally, NV5 recommends that expansion joints be provided between the slab and perimeter footings, and that crack control grooves or saw cuts are installed no more than 10 foot centers in both directions (perpendicular).
8. Concrete slabs should be moisture cured for at least seven days after placement. Excessive curling of the slab may occur if moisture conditioning is not performed. This is especially critical for slabs that are cast during the warm summer months.
9. Concrete slabs impart a relatively small load on the subgrade (approximately 50 psf). Therefore, some vertical movement should be anticipated from possible expansion or differential loading.
10. Field Observations: Field observations should be made by an NV5 construction monitor of all concrete slab on grade surfaces and installed steel reinforcements prior to pouring concrete.

5.2.4 Rock Anchors

Rock anchors or doweling may be used to provide lateral and uplift resistance where shallow, competent rock limits footing excavation. Rock anchors should only be installed in competent rock, to be determined in the field by a representative of NV5. The design of rock anchors should include the following criteria.

1. Pull-out resistance for rock anchors will generally be limited by the shear resistance between the grout and the native rock. For design purposes, a pull-out resistance of 150 pounds per square inch of grout/competent rock contact may be used. Because of the strain in the anchor steel during pull-out, we recommend that the upper 6 inches of grout/competent rock contact be neglected when sizing for uplift.
2. We recommend that the drilled hole have a minimum ½-inch annular clearance between the steel and surrounding rock. Thus, grouting a No. 4 rebar would require a 1½-inch diameter hole.

3. Lateral shear resistance for rock anchors should be designed using Vs=0.45 Fy, where Fy equals the tensile strength of the steel. To develop this shear resistance, a minimum steel embedment of 24 inches into undisturbed, competent rock should be used.
4. Prior to anchor placement, loose debris, dust, and standing water in the hole must be removed by blowing with oil-free compressed air, cleaning the hole with a nylon brush, and then blowing out the remaining dust. Dust and debris left in the hole will significantly reduce anchor capacity.
5. We recommend using a cement grout that has a water/cement ratio of less than 0.5 to construct rock anchors. If high strength epoxy or other adhesives are proposed, NV5 should review the proposed rock anchor detail prior to construction.
6. If rock anchors are used on more than 10 percent of the foundation system of any given structure, a representative of NV5 should perform pull tests on select anchors.

5.2.5 Retaining Wall Design Criteria

The following active and passive pressures are for retaining walls in cut native soil or backfilled with granular onsite soil. If import soil is used, a representative from our firm should be retained to observe and test the soil to determine its strength properties. The pressures exerted against retaining walls may be assumed to be equal to a fluid of equivalent unit weight.

Table 5.2.5-1, Equivalent Fluid Unit Weights (1)

Loading Condition	**Retained Cut or Compacted Fill (approximately horizontal backfill)**	**Retained Cut or Compacted Fill (retained slope up to 2:1, H:V)**
Active Pressure (pcf)	30	45
Passive Pressure (pcf)	350	350
At-Rest Pressure (pcf)	50	60
Coefficient of Friction	0.40	0.40

Note: (1) The equivalent fluid unit weights presented are ultimate values and do not include a factor of safety.
The passive pressures provided assume footings are founded in competent native soil or engineered fill.

Table 5.2.5-1 presents equivalent fluid unit weights for cut native soil and onsite fill compacted per the grading recommendations presented in this report. For approximately horizontal backfill we assume that the retained fill surface will be no steeper than 10% for a minimum distance of the wall height from the back of the retaining wall. If surcharge loads (such as adjacent building foundations) or live loads will be applied within a distance of the wall height from the back of the wall, we should be retained to review the loading conditions and revise our recommendations, if necessary.

The passive pressures provided assume footings are founded in competent native soil or engineered fill.

Please note that the use of the tabulated active pressure unit weight requires that the wall design accommodate sufficient deflection for mobilization of the retained soil to occur. Typically, a wall yield of less than 1 percent of the wall height is sufficient to mobilize active conditions in granular soil. However, if the walls are rigid or restrained to prevent rotation, at-rest conditions should be used for design.

Recommendations for design and construction of retaining walls are listed below:

1. Compaction equipment should not be used directly adjacent to retaining walls unless the wall is designed or braced to resist the additional lateral pressures.
2. If any surface loads are closer to the top of the retaining wall than its height, NV5 should review the loads and loading configuration. We should be retained to review wall details and plans for any wall over 12 feet in height.
3. All retaining walls must be well drained to reduce hydrostatic pressures. Walls should be provided with a drainage blanket to reduce additional lateral forces and minimize saturation of the backfill soil. Drainage blankets may consist of graded rock drains or geosynthetic blankets.
4. Rock drains should consist of a minimum 12-inch wide, Caltrans Class II, permeable drainage blanket, placed directly behind the wall; or crushed washed rock enveloped in a non-woven geotextile filter fabric such as Amoco 4546™ or equivalent. Drains should have a minimum 4-inch diameter, perforated, schedule 40, PVC pipe placed at the base of the wall, inside the drainrock, with the perforations placed down. The PVC pipe should be sloped so that water is directed away from the wall by gravity. A geosynthetic drainage blanket such as Enkadrain™ or equivalent may be substituted for the rock drain, provided the collected water is channeled away from the wall. If a geosynthetic blanket is used, backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.
5. Adequate drainage and waterproofing for retaining walls associated with finished interior spaces are essential to reduce the likelihood of seepage and vapor transmission into the living space. We recommend that an appropriate waterproofing sealant be applied to the exterior surface of such retaining walls. A waterproofing consultant may be contacted to further review seepage and vapor transmission.
6. Additional lateral loading on retaining structures due to seismic accelerations may be considered at the designer's option. For an earthquake producing a design horizontal acceleration of 0.236g, we recommend that the resulting additional lateral force applied to unrestrained (cantilevered) retaining structures with drained level backfill onsite be estimated as $Pae=4H^2$ pounds, where H is the height of the wall in feet. A value of $9H^2$ should be used for restrained walls. The additional seismic force may be assumed to be applied at a height of 0.3H above the base of the wall. This seismic loading is for a drained, level backfill condition only; NV5 should be consulted for values of seismic loading due to non-level or non-drained backfill conditions. The use of reduced factors of safety is often appropriate when reviewing overturning and sliding resistance during seismic events.

5.2.6 Pavement Design

The following recommended asphalt concrete flexible pavement sections are based on design R-value of 17 in the central eastern portion of the Site and a design R-value of 33 near the existing New Brunswick Shaft (northern portion of the Site). Preliminary traffic indices (TIs) of 5-7 were selected based on anticipated light to heavy industrial activity at the Site. The TIs are being considered on a preliminary basis to facilitate planning of the proposed onsite and offsite roadways. Other Tis, or R-values may need to be considered in design if heavy vehicle loads, truck traffic, or improvements deviate from the proposed development plan. Pavement design is presented in Table 5.2.6-1 below.

Table 5.2.6.1 – Recommended Pavement Sections

Development Zone (Sample ID)	R-Value	Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Baserock at 95% Compaction (inches)	Subgrade Soil at 95% Compaction (inches)
Light Industrial (BK-2)	17	5	2.5	9	12
		6	2.5	12.5	12
		7	2.8	15	12
Heavy Industrial (BK-1)	33	6	2.5	9	12
		7	2.8	11.0	12

We make the following recommendations regarding paving at the site.

1. Fill must be compacted to a minimum of 90 percent of the maximum dry density per ASTM D 1557, Modified Proctor. The upper 12 inches of subgrade in areas to be paved must be compacted to a minimum of 95 percent per ASTM D 1557. Baserock should be compacted to a minimum of 95 percent per ASTM D 1557. Moisture content, density and relative percent compaction should be verified by NV5. In addition to density testing, the subgrade must be proofrolled under the observation of a representative of NV5, prior to baserock placement.
2. Subgrade should be sloped to drain away from the proposed road alignment.
3. Import soil, if used, should be predominantly granular, non-expansive and free of deleterious material. Proposed import should be submitted to NV5 for testing prior to transport to the site.
4. Steel reinforced concrete slabs should be considered for use in loading bays, service docks, garbage facilities, and other areas where frequent, heavy vehicle loads are anticipated. The project structural engineer should determine slab thickness and steel reinforcement.
5. Depending on the subsurface conditions encountered and the sources of fill, the actual subgrade material may vary significantly from that tested during this investigation. Representative subgrade samples should be obtained, and additional R-value tests performed, if appropriate, to confirm the recommendations in this report. If the results of confirmation testing vary significantly from those used in design, the recommended pavement sections may need to be revised.

6.0 LIMITATIONS

The following limitations apply to the findings, conclusions and recommendations presented in this report:

1. This report should not be relied upon without review by NV5 if a period of 24 months elapses between the issuance report date shown above and the date when construction commences.
2. Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in northern California. No warranty is expressed or implied.

3. These services were performed consistent with our agreement with our client. We are not responsible for the impacts of any changes in environmental standards, practices, or regulations subsequent to performance of our services. We do not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of our client unless noted otherwise. Any reliance on this report by a third party is at the party's sole risk.

4. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report should be considered invalid. Only our firm can determine the validity of the conclusions and recommendations presented in this report. Therefore, we should be retained to review all project changes and prepare written responses with regards to their impacts on our conclusions and recommendations. However, we may require additional fieldwork and laboratory testing to develop any modifications to our recommendations. Costs to review project changes and perform additional fieldwork and laboratory testing necessary to modify our recommendations are beyond the scope of services presented in this report. Any additional work will be performed only after receipt of an approved scope of services, budget, and written authorization to proceed.

5. The analyses, conclusions and recommendations presented in this report are based on site conditions as they existed at the time we performed our surface and subsurface field investigations. We have assumed that the subsurface soil and groundwater conditions encountered at the location of our exploratory trenches and borings are generally representative of the subsurface conditions throughout the entire subject site. However, the actual subsurface conditions at locations between and beyond our exploratory trenches/borings may differ. Therefore, if the subsurface conditions encountered during construction are different than those described in this report, then we should be notified immediately so that we can review these differences and, if necessary, modify our recommendations.

6. The elevation or depth to the groundwater table underlying the subject site may differ with time and location; therefore, the depth to the groundwater table encountered in the exploratory trenches and borings is only representative of the specific time and location where it was observed.

7. The subject site map shows approximate exploratory trench/boring locations as determined by pacing distances from identifiable site features. Therefore, the trench/boring locations should not be relied upon as being exact nor located with surveying methods.

8. Our geotechnical investigation scope of services did not include evaluating the subject site for the presence of historic mining surface features or hazardous materials. Although we did not observe evidence of historic mining activity or hazardous materials within the proposed building area at the time of our field investigation, all project personnel should be careful and take the necessary precautions should hazardous materials be encountered during construction. Possible historic mining excavation not detected during our investigation may impact the proposed improvements.

9. NV5's experience and that of the civil engineering profession clearly indicates that during the construction phase of a project the risks of costly design, construction and maintenance problems can be significantly reduced by retaining a design geotechnical engineering firm to review the project plans and specifications and to provide geotechnical engineering CQA observation and testing services. Upon your request NV5 will prepare a CQA geotechnical engineering services proposal that will present a work scope, a tentative schedule and fee estimate for your consideration and authorization.

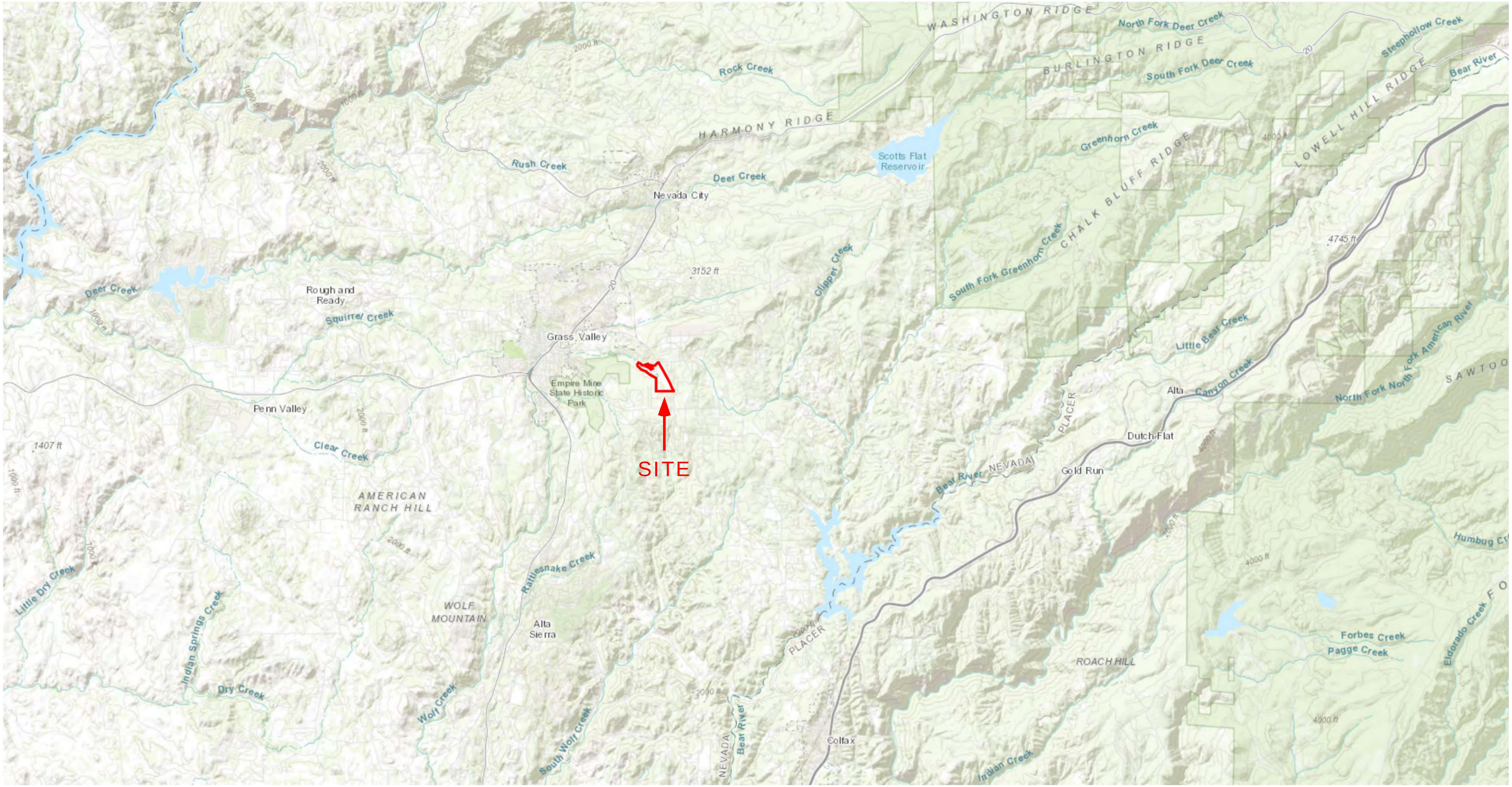
If NV5 is not retained to provide geotechnical engineering CQA services during the construction phase of the project, then NV5 will not be responsible for geotechnical engineering CQA services provided by others nor any aspect of the project that fails to meet your or a third party's expectations in the future.

10. All temporary excavations must comply with applicable local, state and federal safety regulations, including the current Occupational Safety and Hazards Administration (OSHA) excavation and trench safety standards. Construction site safety is the responsibility of the contractor, who is solely responsible for the means, methods and sequencing of construction operations. Under no circumstances should the findings, conclusions and recommendations presented herein be inferred to mean that NV5 is assuming any responsibility for temporary excavations, or for the design, installation, maintenance and performance of any temporary shoring, bracing, underpinning or other similar systems.

11. The findings of this report are valid as of the present date. However, changes in the conditions of the property can occur with the passage of time. The changes may be due to natural processes or to the works of man, on the subject site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or the broadening of knowledge. Therefore, the recommendations presented in this report should not be relied upon after a period of two years from the issue date without our review.

FIGURES

Figure 1 Site Vicinity Map
Figure 2 Exploration Map



N

2 0 2 4 miles

Scale: 1 Inch = 2 Miles

Reference: NEVADA COUNTY GIS, ACCESSED 11/9/19

NV5

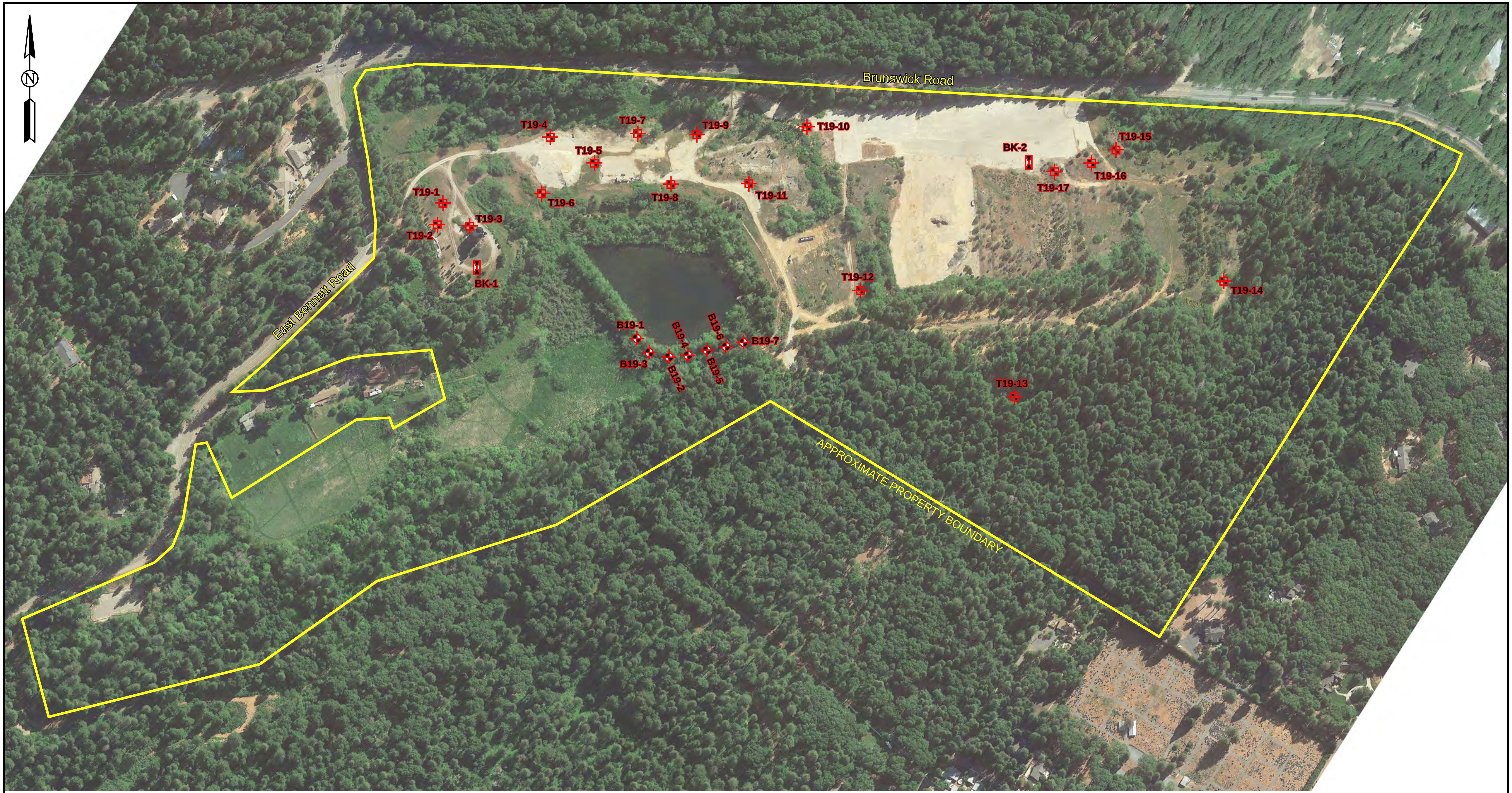
792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

SITE VICINITY MAP

Project Name: IDAHO-MARYLAND MINE - BRUNSWICK SITE
Location: GRASS VALLEY, CALIFORNIA
Project No.: 5279.02

Date: NOV 2019
DRAFTED By: DAV
CHECKED By: CRK

FIGURE
1



EXPLANATION

BK-1 Bulk Sample Location and Number

T19-1 Trench Location and Number

B19-1 Boring Location and Number

300 0 300 600 feet

Scale: 1" = 300'

Reference: GOOGLE EARTH IMAGERY, ACQUISITION DATE 5/17/18, ACCESSED 11/4/19

NV5

792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

EXPLORATION MAP

Project Name: IDAHO-MARYLAND MINE - BRUNSWICK SITE

Location: GRASS VALLEY, CALIFORNIA

Project No.: 5279.02

Date: NOV 2019

DRAFTED By: DAV

CHECKED By: CRK

FIGURE

2

APPENDIX A

Proposal

Project No. 5279.02
August 30, 2019

Rise Gold Corporation
333 Crown Point Circle, Suite 215
Grass Valley, CA 95945

Attention: Benjamin Mossman, P.Eng.
President, CEO, and Director

Reference: Idaho-Maryland Gold Project
Grass Valley, California

Subject: Proposal for Geotechnical Engineering Services

Dear Mr. Mossman:

NV5 proposes to provide geotechnical engineering services in support of near-surface development planning and environmental review for the Idaho-Maryland Gold Project. The geotechnical investigation will include the Brunswick Industrial Site, Centennial Industrial Site, and locations of surface mine features on third-party owned properties. As described below, investigation tasks include:

1. Geotechnical engineering investigation and reports for the Brunswick Industrial Site and Centennial Industrial Site:
    a. Subsurface investigation of Brunswick Industrial Site development area and dam;
    b. Dam stability analysis and grading recommendations if mitigation is necessary;
    c. Evaluation of an existing subsurface culvert at the Brunswick Industrial Site;
    d. Surface reconnaissance of Centennial Industrial Site and document review;
    e. Geotechnical engineering reports, including analysis, recommendations and design criteria for the Brunswick Industrial Site and Centennial Industrial Site:
        i. Recommendations for subgrade preparation, drainage and grading;
        ii. Stability analysis of proposed fill slope configurations;
        iii. Foundation design criteria for proposed industrial development; and
        iv. Design criteria for retaining walls, roads and utilities.
2. Geotechnical engineering review of near-surface mine features:
    a. Surface reconnaissance, document review and/or subsurface investigation of surface features as identified by Rise Gold Corporation; and
    b. Summary letter providing an opinion regarding geotechnical conditions associated with the surface features and, if necessary, recommendations for mitigation and/or monitoring.

SCOPE OF SERVICES

The scope of services described below includes a design-level geotechnical investigation and report for the two industrial sites, as well as geotechnical engineering review of surface features.

TASK 1 – DESIGN-LEVEL GEOTECHNICAL ENGINEERING INVESTIGATION

NV5 will perform a design-level geotechnical investigation of the Centennial Industrial Site and will review previous investigation findings for the Centennial Industrial Site. The investigation will be performed in general accordance with the 2016 California Building Code (CBC), and will focus on the currently proposed development.

Our design-level geotechnical engineering investigation will include map and document review, subsurface exploration and laboratory testing to determine soil engineering material properties, and data analysis to develop earthwork, paving and general foundation design criteria for the proposed development. We will also perform a stability analysis of the existing dam at the Brunswick Industrial Site and proposed fill slope configurations at both sites.

Plan and Document Review

NV5 will perform a map and literature review of published documents pertinent to the two sites including geologic maps, soil survey maps and previous known historical works on the sites.

Surface Reconnaissance

NV5 will perform a surface reconnaissance of both sites to identify surface conditions that may impact the proposed site development plans.

Subsurface Investigation

NV5 will perform a subsurface investigation at the Brunswick Industrial Site to characterize near-surface soil, rock and groundwater conditions, including:

- One day of drilling with truck-mounted auger drill rig to depths of approximately 20 to 30 feet below the ground surface (bgs), or until practical drilling refusal is met. The exploratory borings will focus on the dam to estimate the extent of weak materials identified during previous investigation. If time permits, we will advance borings at other locations in the development area where exploratory trenching is impractical (e.g., paved areas).
- One day of exploratory trenching to depths of approximately 10 feet bgs. The exploratory trenches will focus on the general development area.

Underground Utilities

NV5 will mark the site for Underground Service Alert (USA) prior to subsurface investigation. USA will notify public utility companies of the proposed investigation so that the utility companies can mark their utility locations. We will rely on the client to mark private utilities so that we can avoid them during subsurface exploration. NV5 will not be responsible for damage to subsurface utilities that were not marked or were improperly marked prior to our investigation. To reduce the chance of damage to underground utilities, at your request we can retain a private utility locating service for an additional fee.

Exploratory Drilling

Exploratory boring locations will be determined based on access capabilities of the drill rig and location of existing onsite utilities (if any). Drill cuttings will remain at the boring locations unless Rise Gold Corporation requests their removal. If NV5 must remove excess drill cuttings, additional costs will be incurred based on our fee schedule. The borings will be backfilled with grout pursuant to local code requirements.

NV5's field engineer/geologist will record the subsurface conditions encountered in the exploratory borings. If groundwater is encountered, the depth to groundwater will be measured.

NV5 will collect both relatively undisturbed and disturbed soil samples. Relatively undisturbed soil samples will be collected with a standard penetration test (SPT) sampler and a 2.5-inch inside diameter split spoon barrel sampler equipped with brass liner tubes. The soil samples will be labeled, sealed, and transported to our laboratory facility where selected samples will be tested to determine their engineering material properties.

Generally, soil samples will be collected at depths of 0, 2, 5 and 10 feet bgs, and on five-foot intervals below that depth or at changes in subsurface conditions, until the boring is terminated. Sample intervals are subject to change depending upon the soil conditions encountered.

Exploratory Trenching

NV5 will excavate 6 to 10 exploratory trenches in the proposed development area to depths up to 10 feet bgs, or to refusal if encountered at shallower depths. Excavated soil will be placed back into the exploratory trenches but will not be compacted. Recompaction of the trenches should be performed during site development.

An NV5 engineer/geologist will record subsurface conditions and obtain soil samples using hand tools such as a drive sampler. The soil samples will be labeled, sealed, and transported to our laboratory where selected samples will be tested to determine their engineering material properties.

Laboratory Testing

Laboratory tests will be performed using ASTM International (ASTM) and Caltrans methods as guidelines. Depending on the subsurface conditions encountered, we anticipate that laboratory testing may include:

D1140	200 Mesh Wash
D2166	Unconfined Compressive Strength (rock and soil)
D2216	Moisture Content
D2487	Unified Soil Classification System
D2844	Resistance Value
D2937	Density

- D3080 Direct Shear Strength
- D4318 Atterberg Limits
- D4829 Expansion Index
- Caltrans Method 417 and 422, Sulfate and Chloride
- Caltrans Method 643, Resistivity

Data Analysis And Engineering

NV5 will evaluate slope stability and perform calculations to develop geotechnical engineering design criteria:

- Stability analysis will include the existing dam and proposed fill slope configurations.
- Evaluation of the existing culvert will include non-destructive testing of the culvert thickness at locations near the inlet and outlet, soil corrosivity analysis, and estimation of service life.
- The geotechnical engineering recommendations will include general procedures and geotechnical design parameters for earthwork and structural improvements.

Geotechnical Engineering Reports

Two design-level geotechnical reports will be prepared: one for the Brunswick Industrial Site and one for the Centennial Industrial Site.

The reports will summarize our investigation methods, summarize the results of previous investigation, provide an opinion regarding the geotechnical feasibility of the proposed developments, and present findings and geotechnical engineering design recommendations for the proposed earthwork and structural improvements. The geotechnical engineering design recommendations will address the following topics:

Earthwork Improvements

1. Site clearing and subgrade preparation.
2. Fill moisture conditioning, placement, and compaction.
3. Deep fill placement and rock slope protection.
4. Cut and fill slope grading.
5. Utility trench backfill placement and compaction.
6. Retaining wall backfill.
7. Retaining wall drainage.
8. Surface water drainage.
9. Expansive soil mitigation (if appropriate).
10. Temporary construction dewatering methods.
11. Subdrain recommendations (if appropriate).

Structural Improvements

1. Seismic design parameters (2016 CBC)
2. Foundation types and minimum embedment depths.
3. Allowable soil bearing capacity.
3. Foundation soil friction coefficients.

4. Lateral earth pressures for foundation and retaining wall design.
5. General construction recommendations for slabs-on-grade.
6. Retaining wall design criteria.
7. Typical pavement sections based on R-value results and typical traffic indices.

The reports will include a site plan showing the approximate locations of exploratory borings/trenches and prominent surface features. The report appendices will present the exploratory boring/trench logs and laboratory test data.

TASK 2 – GEOTECHNICAL ENGINEERING REVIEW OF SURFACE FEATURES

NV5 will perform a geotechnical engineering review of near-surface mine features identified by Rise Gold Corporation. We understand that Rise Gold Corporation will provide historical maps and data and will describe the features with respect to proposed underground mining operations.

The geotechnical engineering review will include surface reconnaissance and document review. Although we do not anticipate that subsurface investigation will generally be necessary, this may change depending upon the specific characteristics of each feature.

NV5 will prepare a summary letter providing an opinion regarding geotechnical conditions associated with the surface features and, if necessary, recommendations for mitigation and/or monitoring.

If mitigation measures include engineered physical closure, NV5 will provide an engineered closure plan and specifications. We will be able to provide engineering consultation and testing during physical closure construction and a   to assist you and/or your contractor and summarize our visits in field reports. Upon completion of each closure, we will provide a summary letter documenting the closure work and including our field reports.

ADDITIONAL SERVICES

Review of Plans and Specifications

NV5 should be retained to review the plans and specifications for the development project, when available, to confirm that the findings of our geotechnical engineering investigation are incorporated into the project geotechnical design. Prior to securing a permit, the local building official may require this plan conformance review.

Construction Observation and Testing

The geotechnical engineering recommendations presented in our reports must be validated by NV5 during construction. The purpose of our construction observation and testing is to verify subsurface conditions and confirm that the project is constructed in accordance with the geotechnical engineering recommendations presented in our reports and in the project plans and specifications.

ASSUMPTIONS AND CLIENT RESPONSIBILITIES

This proposal is based on the following assumptions:

- The client will provide NV5 with authorization to access the site.

- NV5 will mark the Brunswick Site for USA prior to performing the subsurface investigation. The client will provide information regarding the location of existing onsite private utilities. Although reasonable care will be used during our investigation, the client understands that unmarked underground utilities may be damaged. NV5 will not be responsible for repair of utilities that were not marked or were improperly marked prior to the investigation. If requested, we can retain a private utility locating service for an additional fee.
- Upon completion of each task, a PDF digital copy of the reports or design sheets will be provided to the client and/or the client's engineers and architects.
- Client meetings, report revisions and consultation services following report submittals are not included in the fee estimate but can be provided on a time and materials basis at the client's request.
- This proposal and our associated fee are based on the use of the attached terms and conditions.

FEES

Task 1 – Design-Level Geotechnical Engineering Investigation

NV5 will perform Task 1, Design-Level Geotechnical Engineering Investigation, on a lump sum basis as described below.

Site	Fee
Brunswick Industrial Site	$
Centennial Industrial Site	$
Total Fee	$

Billing will be monthly on a percent complete basis and terms of payment are net 30 days. Full payment is due upon completion of the work and issuance of the report. The cost associated with this scope of service is valid for a period of 60 days from the date of this proposal.

We request a retainer in the amount of $ that will be applied to the subcontracted exploration services. The remainder of the retainer will be applied to the final invoice upon project completion.

Task 2 – Geotechnical Engineering Review of Surface Features

NV5 will perform Task 2, Geotechnical Engineering Review Of Surface Features, on a time and expense basis according to the attached 2019 Fee Schedule. Our estimated fee for Task 2 is $. Billing will be monthly according to the fee schedule.

SCHEDULE

We have tentatively scheduled the drill rig for October 4, 2019. We can typically provide verbal preliminary design recommendations within one to two weeks of our field investigation. We anticipate the geotechnical reports can be issued within six weeks of the field investigation.

If we encounter field conditions that may require additional investigation or otherwise impact our proposed schedule, we will contact you promptly to discuss.

AUTHORIZATION TO PROCEED

If this proposal is acceptable, please review and sign the attached agreement for engineering services and return one copy to our Nevada City office with a retainer of $9,000 as our authorization to proceed.

Thank you for the opportunity to provide this proposal. If you have any questions, please contact our office.

Sincerely,

NV5

Prepared by:
Daniel Vieira, P.G.
Project Geologist

Reviewed by:
Chuck Kull, G.E.
Principal Engineer

Attached: Agreement for Geotechnical Engineering Services
2019 Fee Schedule

Copy: PDF to Rise Gold Corporation /Attn: Ben Mossman, ceo@risegoldcorp.com
PDF to Rise Gold Corporation /Attn: Tessa Brinkman, tbrinkman.peng@gmail.com

F:\1 Projects\5279 Idaho-Maryland Mine\02 Geotechnical\Proposal\5279.02 NV5 Proposal for Geotechnical Services, Idaho-Maryland Mine.docx

APPENDIX B

Important information about your Geotechnical engineering report
(Included with Permission of GBA, copyright 2016)

Important Information about This
Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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e-mail: info@geoprofessional.org   www.geoprofessional.org

APPENDIX C

Exploratory Boring and Trench Logs

NV5

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

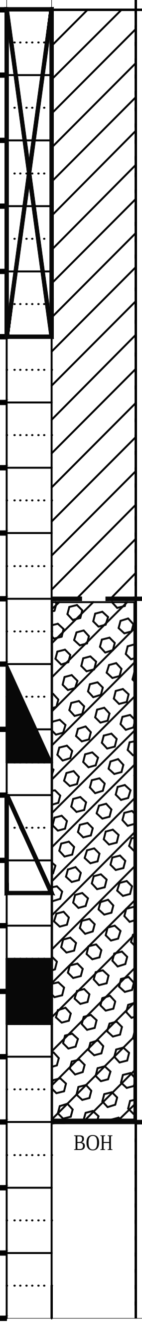
Boring No. B19-1

Project Name: Idaho-Maryland Gold - Brunswick Industrial Site
Project No.: 5279.02
Task: 01
Start Date: 9-30-19
Location: Brunswick Drive, Grass Valley, California
Estimated Ground Surface Elevation (Ft. AMSL):
Finish Date: 9-30-19
Sheet: 1 Of 1
Logged By: Daniel A. Vieira
Drilling Cmpny: H1 Drilling Company
Drill Rig Type: CME-75
Driller: Will Taber
Drilling Method: Solid Stem Augers (SSA)
Hammer Type: 140 Pound Auto Trip Hammer
Boring Dia. (In.): 4.0
Total Depth (Ft.): 17.0
Backfill or Well Design: Grout to Surface

Ground Water Information

Date	9-30-19			
Time (24 Hour)				
Depth (Ft.)	None			

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)
8:50			SSA			0
					B19-1-BK-1	1
						2
						3
						4
						5
						6
						7
						8
						9
9:05		10	SPT			10
		12				
		15		0.6/1.5	B19-1-B1	11
9:15			SSA			
		9	SPT			12
		8				
		6		0/1.5		13
9:22			SSA			
9:25		5	2.5SS			14
		9			B19-1-L1	
		15		1.0/1.5	B19-1-L2	15
9:31			SSA			
						16
						17
						18
						19
						20



Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

(CL) SANDY CLAY, FLD. EST: 70% Clay, 25% Fine to Coarse Sand, 5% Fine Gravel; Yellowish Brown (10YR 5/4); Stiff to Very Stiff, Moist, Few Rootlets, [Fill].

(Grinding/Rig Chatter)

(GC) CLAYEY GRAVEL WITH SAND, FLD. EST: 25% Clay, 35% Fine to Coarse Sand, 40% Fine to Coarse Angular Gravel, few Cobbles and Boulders; Grayish Brown (10YR 5/2) with Bluish Gray (Gley 2 5/1) Crushed Mine Rock; Medium Dense; Damp to Moist. [Fill]

(Grinding)

Practical Refusal at 17.0 Feet BGS, NFWE

NOTES:
SSA - Solid Stem Augers
SPT - Standard Penetration Test
2.5SS - Split Spoon Sampler
BOH - Bottom of Hole
BGS - Below Ground Surface
NFWE - No Free Water Encountered

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. B19-2

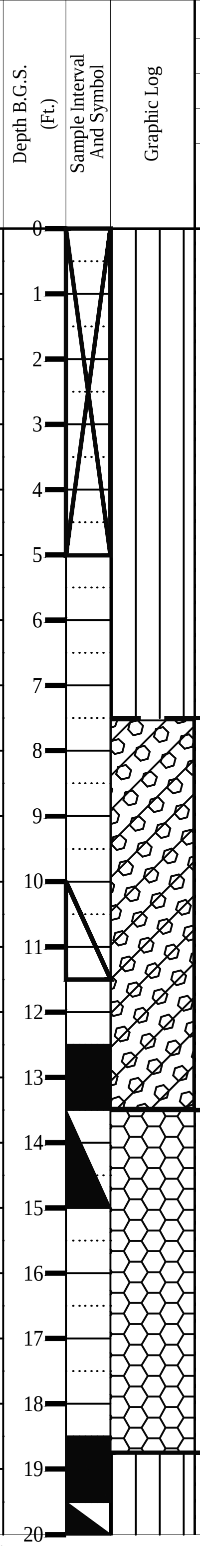
Sheet: 1 Of 2

Project Name: Idaho-Maryland Gold - Brunswick Industrial Site
Project No.: 5279.02
Task: 01
Start Date: 9-30-19
Location: Brunswick Drive, Grass Valley, California
Estimated Ground Surface Elevation (Ft. AMSL):
Finish Date: 9-30-19
Logged By: Daniel A. Vieira
Drilling Cmpny: H1 Drilling Company
Drill Rig Type: CME-75
Driller: Will Taber
Drilling Method: Solid Stem Augers (SSA)
Hammer Type: 140 Pound Auto Trip Hammer
Boring Dia. (In.): 4.0
Total Depth (Ft.): 26.5
Backfill or Well Casing: Grout to Surface

Ground Water Information

Date	9-30-19			
Time (24 Hour)				
Depth (Ft.)	None			

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.
9:50			SSA		
					B19-2-B1
10:00		11	SPT		
		12			
		12		0/1.5	
10:05			SSA		
10:12		2	2.5SS		
		6			B19-2-L1
		15		.8/1.5	B19-2-L2
10:18		8	SPT		
		13			B19-2-B2
		25		.33/1.5	
			SSA		
10:27		22	2.5SS	0/1.5	
10:39			SSA		
10:45		7	2.5SS		
		12			B19-2-L3
		18		1.3/1.5	B19-2-L4
		5	SPT		



Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

(ML) SANDY SILT, FLD. EST: 70% Low Plasticity Clay+Silt Fines, 25% Fine to Coarse Sand, 5% Fine Gravel; Yellowish Brown (10YR 5/6); Stiff, Moist, Few Rootlets, [Fill].

(Grinding/Rig Chatter)

(GC) CLAYEY GRAVEL WITH SAND, FLD. EST: 25% Clay, 35% Fine to Coarse Sand, 40% Fine to Coarse Angular Gravel, few Cobbles and Boulders; Strong Brown (7.5YR 5/6) to Grayish Brown (10YR 5/2) Soil with Bluish Gray (Gley 2 5/1) Crushed Mine Rock; Medium Dense, Moist, [Fill].

(GW) WELL GRADED GRAVEL WITH SAND, FLD. EST: 5% Low Plasticity Clay+Silt Fines, 30% Fine to Coarse Sand, 65% Fine to Coarse Angular Gravel, some Cobble and Gravels; Bluish Gray (Gley 2 5/1); Medium Dense, Damp. [Crushed Mine Rock, Fill].

(Pushed Cobble)

(ML) SILT WITH SAND, FLD. EST: 80% Low Plasticity Clay+Silt Fines, 20% Fine to Coarse Sand; Mottled Dark Brown (7.5YR 3/2); Medium Dense; Moist; Few Rootlets [Fill].

NOTES:

SSA - Solid Stem Augers	BOH - Bottom of Hole
SPT - Standard Penetration Test	BGS - Below Ground Surface
2.5SS - Split Spoon Sampler	NFWE - No Free Water Encountered

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No.

B19-2

Sheet: 2 Of 2

Project Name: Idaho-Maryland Gold - Brunswick Industrial Site
Project No.: 5279.02
Task: 01
Start Date: 9-30-19
Location: Brunswick Drive, Grass Valley, California
Estimated Ground Surface Elevation (Ft. AMSL):
Finish Date: 9-30-19
Logged By: Daniel A. Vieira
Drilling Cmpny: H1 Drilling Company
Drill Rig Type: CME-75
Driller: Will Taber
Drilling Method: Solid Stem Augers (SSA)
Hammer Type: 140 Pound Auto Trip Hammer
Boring Dia. (In.): 4.0
Total Depth (Ft.): 26.5
Backfill or Well Casing: Grout to Surface

Ground Water Information

Date	9-30-19			
Time (24 Hour)				
Depth (Ft.)	None			

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Sample Interval And Symbol	Graphic Log	Soil And/Or Rock Material Descriptions
		8	SPT			20			(ML) SILT WITH SAND, FLD. EST: 80% Low Plasticity Clay+Silt Fines, 20% Fine to Coarse Sand; Dark Brown (7.5YR 3/2); Medium Dense; Moist; Few Rootlets [Fill].
		11	↓	1.0/1.5	B19-2-B3				
10:55			SSA			21			
			↓						
11:00		4	2.5SS			22			(ML) SILT, FLD. EST: 90% Low Plasticity Clay+Silt Fines, 10% Fine to Medium Sand; Very Dark Grayish Brown (2.5YR 3/2); Stiff; Moist to Wet.
	1.25-1.5	6			B19-2-L5				(Native/Floodplain Deposits)
	1.0-1.75	8	↓	1.25/1.5	B19-2-L6	23			
11:06			SSA						
						24			
			↓						
		2	SPT			25			
	1.75	2							(CL) CLAY WITH SAND, FLD. EST: 85% Low Plasticity Clay+Silt Fines, 15% Fine to Coarse Sand; Reddish Brown (5YR 4/4); Medium Stiff to Stiff, Moist; (Residual Soil).
		3	↓	.7/1.5	B19-2-B4	26			
								BOH	Boring Terminated at 26.5 Feet BGS, NFWE
						27			
						28			
						29			
						30			
						31			
						32			
						33			
						34			
						35			
						36			
						37			
						38			
						39			
						40			

NOTES:

SSA - Solid Stem Augers
SPT - Standard Penetration Test
2.5SS - Split Spoon Sampler
BOH - Bottom of Hole
BGS - Below Ground Surface
NFWE - No Free Water Encountered

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. B19-3

Sheet: 1 Of 2

Project Name: Idaho-Maryland Gold - Brunswick Industrial Site
Project No.: 5279.02
Task: 01
Start Date: 9-30-19
Location: Brunswick Drive, Grass Valley, California
Estimated Ground Surface Elevation (Ft. AMSL):
Finish Date: 9-30-19
Logged By: Daniel A. Vieira
Drilling Cmpny: H1 Drilling Company
Drill Rig Type: CME-75
Driller: Will Taber
Drilling Method: Solid Stem Augers (SSA)
Hammer Type: 140 Pound Auto Trip Hammer
Boring Dia. (In.): 4.0
Total Depth (Ft.): 22.0
Backfill or Well Casing: Grout to Surface

Ground Water Information

Date	9-30-19			
Time (24 Hour)				
Depth (Ft.)	None			

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Soil And/Or Rock Material Descriptions
11:30			SSA			0	(ML) SANDY SILT, FLD. EST: 70% Low Plasticity Clay+Silt Fines, 25% Fine to Coarse Sand, 5% Fine Gravel; Yellowish Brown (10YR 5/6); Stiff; Moist; [Fill].
						1	
						2	
						3	
						4	
						5	
						6	
						7	
						8	
						9	(Grinding/Rig Chatter)
						10	(GC) CLAYEY GRAVEL WITH SAND, FLD. EST: 25% Low Plasticity Clay+Silt Fines, 35% Fine to Coarse Sand, 40% Fine to Coarse Angular Gravel with a few Cobbles and Boulders; Strong Brown (7.5YR 5/6) Soil with Bluish Gray (Gley 2 5/1) Rock; Medium Dense, Moist, [Fill].
						11	
						12	
11:48		17	2.5SS	.8/1.5		13	
		22					
		22			B19-3-L1	14	
11:54		12	SPT	1.0/1.5			(GW) WELL GRADED GRAVEL WITH SAND, FLD. EST: 5% Low Plasticity Clay+Silt Fines, 30% Fine to Coarse Sand, 65% Fine to Coarse Angular Gravel, some Cobble and Gravels; Bluish Gray (Gley 2 5/1); Medium Dense, Damp.
		12			B19-3-B1	15	
		8					
			SSA			16	
						17	[Fill]
						18	(GC) CLAYEY GRAVEL WITH SAND, FLD. EST: 25% Low Plasticity Clay+Silt Fines, 35% Fine to Coarse Sand, 40% Fine to Coarse Angular Gravel with a few Cobbles and Boulders; Grayish Brown (10YR 5/2) Soil with Bluish Gray (Gley 2 5/1) Rock; Medium Dense, Moist, [Fill].
12:15							
		10	2.5SS			19	
		13			B19-3-L2		
						20	

NOTES:
SSA - Solid Stem Augers
SPT - Standard Penetration Test
2.5SS - Split Spoon Sampler
BOH - Bottom of Hole
BGS - Below Ground Surface
NFWE - No Free Water Encountered

NV5

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. B19-3

Sheet: 2 Of 2

Project Name: Idaho-Maryland Gold - Brunswick Industrial Site
Project No.: 5279.02
Task: 01
Start Date: 9-30-19
Location: Brunswick Drive, Grass Valley, California
Estimated Ground Surface Elevation (Ft. AMSL):
Finish Date: 9-30-19
Logged By: Daniel A. Vieira
Drilling Cmpny: H1 Drilling Company
Drill Rig Type: CME-75
Driller: Will Taber
Drilling Method: Solid Stem Augers (SSA)
Hammer Type: 140 Pound Auto Trip Hammer
Boring Dia. (In.): 4.0
Total Depth (Ft.): 22.0
Backfill or Well Casing: Grout to Surface

Ground Water Information

Date	9-30-19			
Time (24 Hour)				
Depth (Ft.)	None			

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Sample Interval And Symbol	Graphic Log	Soil And/Or Rock Material Descriptions SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc. ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.
		16	2.5SS	0.5/1.5	B19-3-L3	20			(GC) CLAYEY GRAVEL WITH SAND
		8	SPT						
		10	↓		B19-3-B2	21			(ML) SILT, FLD. EST: 90% Low Plasticity Clay+Silt Fines, 10% Fine to Medium Sand; Very Dark Grayish Brown (2.5YR 3/2); Very Stiff; Moist. (Native/Floodplain Deposits)
		8		0.7/1.5					
						22		BOH	Boring Terminated at 22.0 Feet BGS, NFWE
						23			
						24			
						25			
						26			
						27			
						28			
						29			
						30			
						31			
						32			
						33			
						34			
						35			
						36			
						37			
						38			
						39			
						40			

NOTES:

SSA - Solid Stem Augers	BOH - Bottom of Hole
SPT - Standard Penetration Test	BGS - Below Ground Surface
2.5SS - Split Spoon Sampler	NFWE - No Free Water Encountered

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

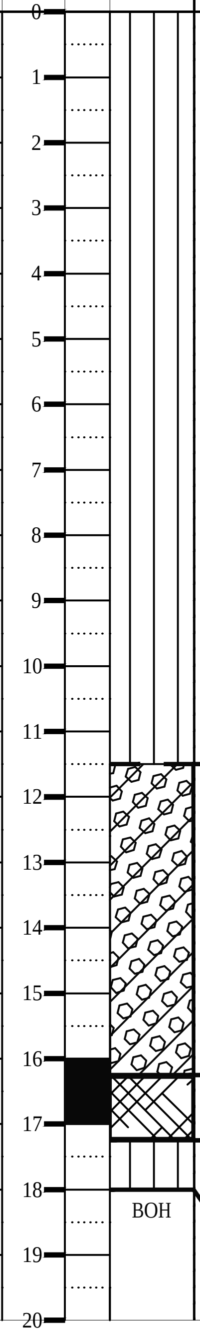
Boring No. B19-4
Sheet: 1 Of 1

Project Name: Idaho-Maryland Gold - Brunswick Industrial Site
Project No.: 5279.02
Task: 01
Start Date: 9-30-19
Location: Brunswick Drive, Grass Valley, California
Estimated Ground Surface Elevation (Ft. AMSL):
Finish Date: 9-30-19
Logged By: Daniel A. Vieira
Drilling Cmpny: H1 Drilling Company
Drill Rig Type: CME-75
Driller: Will Taber
Drilling Method: Solid Stem Augers (SSA)
Hammer Type: 140 Pound Auto Trip Hammer
Boring Dia. (In.): 4.0
Total Depth (Ft.): 18.0
Backfill or Well Casing: Grout to Surface

Ground Water Information

Date	9-30-19			
Time (24 Hour)				
Depth (Ft.)	None			

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.
14:20			SSA		
14:36		5	2.5SS		
		7			
		11		0.8/1.5	B19-4-L1
14:39		4	2.0SS		B19-4-B1
		6			
14:45	1.75-2.25	8		1.5/1.5	B19-4-B2



Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

(ML) SANDY SILT, FLD. EST: 70% Low Plasticity Clay+Silt Fines, 25% Fine to Coarse Sand, 5% Fine Gravel; Yellowish Brown (10YR 5/6); Stiff; Moist; [Fill].

(Rig Chatter)

(GC) CLAYEY GRAVEL WITH SAND, FLD. EST: 25% Low Plasticity Clay+Silt Fines, 35% Fine to Coarse Sand, 40% Fine to Coarse Angular Gravel with a few Cobbles and Boulders; Strong Brown (7.5YR 5/6) Soil with Bluish Gray (Gley 2 5/1) Rock; Medium Dense, Moist, [Fill].

(OM) ORGANIC SOIL, FLD. EST: 30% Low Plasticity Clay+Silt Fines, 70% Organics (Sawdust and Bark); Black; Stiff; Damp to Moist. [Fill]

(ML) SILT WITH SAND, FLD. EST: 80% Silt, 20% Fine to Coarse Sand; Mottled Dark Brown (7.5YR 3/2); Stiff to Very Stiff; Moist; [Fill Bottom/Native Floodplan Deposit].

Boring Terminated at 18.0 Feet BGS, NFWE

NOTES:
SSA - Solid Stem Augers
SPT - Standard Penetration Test
2.5SS - 2.5" Split Spoon Sampler
2.0SS - 2.0" Split Spoon Sampler
BGS - Below Ground Surface
NFWE - No Free Water Encountered
BOH - Bottom of Hole

EXPLORATORY BORING LOG

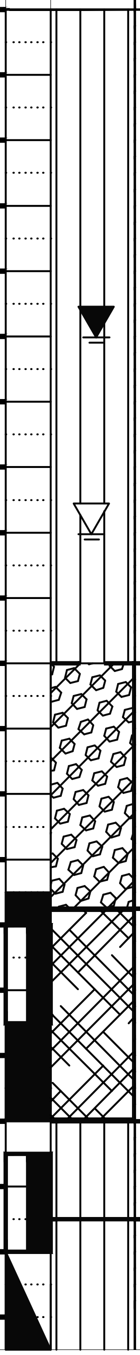
48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. B19-5
Sheet: 1 Of 1

Project Name: Idaho-Maryland Gold - Brunswick Industrial Site
Project No.: 5279.02
Task: 01
Start Date: 9-30-19
Location: Brunswick Drive, Grass Valley, California
Estimated Ground Surface Elevation (Ft. AMSL):
Finish Date: 9-30-19
Logged By: Daniel A. Vieira
Drilling Cmpny: H1 Drilling Company
Drill Rig Type: CME-75
Driller: Will Taber
Drilling Method: Solid Stem Augers (SSA)
Hammer Type: 140 Pound Auto Trip Hammer
Boring Dia. (In.): 4.0
Total Depth (Ft.): 20.5
Backfill or Well Casing: Grout to Surface

Ground Water Information

Date	9-30-19	9-30-19		
Time (24 Hour)	14:45	17:00		
Depth (Ft.)	8.0	5.0		



Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Soil And/Or Rock Material Descriptions
14:53			SSA			0	(ML) SANDY SILT, FLD. EST: 70% Low Plasticity Clay+Silt Fines, 25% Fine to Coarse Sand, 5% Fine Gravel; Yellowish Brown (10YR 5/6); Stiff, Moist, Few Rootlets, [Fill].
						1	
						2	
						3	
						4	
						5	
						6	
						7	
14:45						8	[Piece of Plastic/PVC on Auger]
						9	(Rig Chatter)
						10	(GC) CLAYEY GRAVEL WITH SAND, FLD. EST: 25% Clay, 35% Fine to Coarse Sand, 40% Fine to Coarse Angular Gravel, few Cobbles and Boulders; Strong Brown (7.5YR 5/6) to Grayish Brown (10YR 5/2) Soil with Bluish Gray (Gley 2 5/1) Crushed Mine Rock; Medium Dense, Moist, [Fill].
						11	
						12	
15:03		2	2.5SS				
		2				13	
		4		0.8/1.5	B19-5-L1		
		2	2.0SS			14	(OM) ORGANIC SOIL, FLD. EST: 10-30% Low Plasticity Clay+Silt Fines, 70-90% Organics (Sawdust); Black; Soft; Damp. [Fill]
		3			B19-5-B1		
		4		1.5/1.5		15	
		3	SPT				
		3			B19-5-B2	16	[Become 90% Well Preserved Sawdust]
		4		0.4/1.5			
			SSA			17	(ML) SILT WITH SAND, FLD. EST: 80% Low Plasticity Clay+Silt Fines, 20% Fine to Coarse Sand; Occasional Sawdust and Bark; Mottled Dark Brown (7.5YR 3/2); Soft; Wet; [Bottom of Fill].
		1	2.0SS	1.5/1.5			
		1			B19-5-B3	18	
		2		0.4/1.5			(ML) SILT, FLD. EST: 90% Low Plasticity Clay+Silt Fines, 10% Fine to Medium Sand; Very Dark Grayish Brown (2.5Y 3/2); Medium Stiff; Moist to Wet.
15:23		1	SPT			19	
		2			B19-5-B4		(Native/Floodplain Deposits)
		3				20	

NOTES: SSA - Solid Stem Augers
SPT - Standard Penetration Test
BOH - Bottom of Hole
BGS - Below Ground Surface
NFWE - No Free Water Encountered
2.5SS - Split Spoon Sampler

Boring Terminated at 20.5 Feet BGS, Groundwater First Encountered at 8 Feet BGS, Stabilized to 5 Feet BGS.

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. B19-6
Sheet: 1 Of 1

Project Name: Idaho-Maryland Gold - Brunswick Industrial Site
Project No.: 5279.02
Task: 01
Start Date: 9-30-19
Location: Brunswick Drive, Grass Valley, California
Estimated Ground Surface Elevation (Ft. AMSL):
Finish Date: 9-30-19
Logged By: Daniel A. Vieira
Drilling Cmpny: H1 Drilling Company
Drill Rig Type: CME-75
Driller: Will Taber
Drilling Method: Solid Stem Augers (SSA)
Hammer Type: 140 Pound Auto Trip Hammer
Boring Dia. (In.): 4.0
Total Depth (Ft.): 17.5
Backfill or Well Casing: Grout to Surface

Ground Water Information

Date				
Time (24 Hour)				
Depth (Ft.)				

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Graphic Log	Soil And/Or Rock Material Descriptions
			SSA			0		(ML) SANDY SILT, FLD. EST: 70% Low Plasticity Clay+Silt Fines, 25% Fine to Coarse Sand, 5% Fine Gravel; Yellowish Brown (10YR 5/6); Stiff, Moist, Few Rootlets, [Fill].
						1		
						2		
						3		
						4		
						5		
						6		
						7		
						8		
						9		(Grinding/Rig Chatter)
						10		(GC) CLAYEY GRAVEL WITH SAND, FLD. EST: 25% Clay, 35% Fine to Coarse Sand, 40% Fine to Coarse Angular Gravel, few Cobbles and Boulders; Strong Brown (7.5YR 5/6) to Grayish Brown (10YR 5/2) Soil with Bluish Gray (Gley 2 5/1) Crushed Mine Rock; Medium Dense, Moist, [Fill].
						11		
						12		(OM) ORGANIC SOIL, FLD. EST: 10% Low Plasticity Clay+Silt Fines, 90% Organics (Sawdust); Black; Medium Stiff; Damp. [Fill]
15:46		2	2.5SS		B19-6-L1	13		
		4			B19-6-L2			
		5		1.5/1.5	B19-6-L3	14		
		3	2.0SS					
		2			B19-6-B1	15		
		4		1.5/1.5				
		1	SPT			16		(ML) SILT, FLD. EST: 80% Low Plasticity Clay+Silt Fines, 20% Fine to Coarse Sand; Very Dark Grayish Brown (2.5Y 3/2); Soft; Moist to Wet. (Native/Floodplain Deposits)
		1			B19-6-B2	17		
15:54		3		1.3/1.5				
						18	BOH	Boring Terminated at 17.5 Feet BGS, NFWE
						19		
						20		

NOTES:

SSA - Solid Stem Augers	2.0SS - 2.0" Split Spood Sampler	BOH - Bottom of Hole
SPT - Standard Penetration Test	BGS - Below Ground Surface	
2.5SS - Split Spoon Sampler	NFWE - No Free Water Encountered	

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. B19-7

Sheet: 1 Of 1

Project Name: Idaho-Maryland Gold - Brunswick Industrial Site	Project No.: 5279.02	Task: 01	Start Date: 9-30-19	
Location: Brunswick Drive, Grass Valley, California	Estimated Ground Surface Elevation (Ft. AMSL):		Finish Date: 9-30-19	
Logged By: Daniel A. Vieira	Drilling Cmpny: H1 Drilling Company		Drill Rig Type: CME-75	
Driller: Will Taber	Drilling Method: Solid Stem Augers (SSA)		Hammer Type: 140 Pound Auto Trip Hammer	
Boring Dia. (In.): 4.0	Total Depth (Ft.): 19.0	Backfill or Well Casing: Grout to Surface		

Ground Water Information

Date				
Time (24 Hour)				
Depth (Ft.)				

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

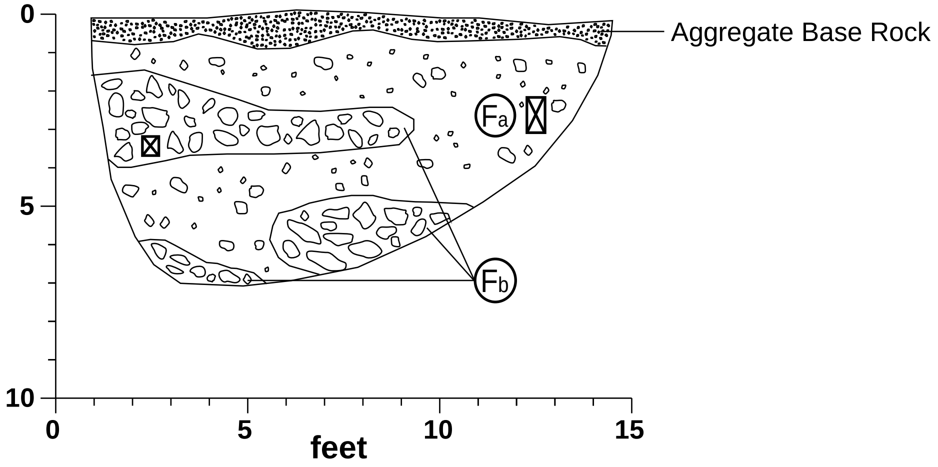
24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Sample Interval And Symbol	Graphic Log	Soil And/Or Rock Material Descriptions
16:05			SSA			0			(ML) SANDY SILT, FLD. EST: 70% Low Plasticity Clay+Silt Fines, 25% Fine to Coarse Sand, 5% Fine Gravel; Yellowish Brown (10YR 5/6); Stiff, Moist, Few Rootlets, [Fill].
					B19-7-BK1	1			
						2			
						3			
						4			
						5			
						6			
						7			
						8			
						9			(Becomes Medium Stiff)
16:09		3	2.0SS			10			
		4							
		4		1.0/1.5	B19-7-B1	11			
16:13			SSA						
						12			
						13			
						14			(CL) SANDY CLAY, FLD. EST: 60% Low Plasticity Clay+Silt Fines, 35% Fine to Coarse Sand, 5% Fine Gravel; Strong Brown (5YR 4/4) to Mottled Olive Gray (5Y 5/2); Medium Stiff; Moist.
16:16		2	SPT			15			
		2		1.5/1.5					
		3			B19-7-B2	16			
16:22			SSA						[Bottom of Fill]
						17			(CL) CLAY WITH SAND, FLD. EST: 85% Low Plasticity Clay+Silt Fines, 15% Fine to Coarse Sand; Reddish Brown (5YR 4/4); Medium Stiff; Moist; Few Rootlets.
16:26		3	2.0SS						
		5				18			[Residual Soil]
16:30		5		1.5/1.5	B19-7-B3				
						19		BOH	Boring Terminated at 19.0 Feet BGS, NFWE
						20			

NOTES:

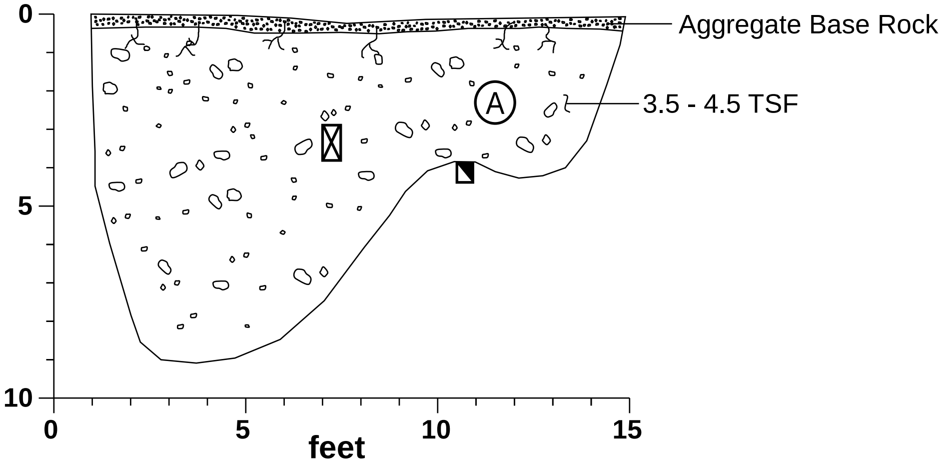
SSA - Solid Stem Augers	2.0SS - 2.0" Split Spoon Sampler	BOH - Bottom of Hole
SPT - Standard Penetration Test	BGS - Below Ground Surface	
2.5SS - Split Spoon Sampler	NFWE - No Free Water Encountered	

LOG OF TRENCH T19-1 & T19-2

T19-1



T19-2



Unit	Sample		Soil and/or Rock Material Description
	Depth (ft)	No.	
Fa	2.0-3.0	T19-1-B1	(SC); CLAYEY SAND WITH GRAVEL; Field Estimate: 40% clay, 35% fine to coarse sand, 25% fine to coarse gravel; occasional cobbles up to 8" Ø; yellowish red (5YR 5/8); medium dense; damp; contains trace amounts of debris (e.g., metal, concrete). [Heterogenous Fill]
Fb	3.0-3.5	T19-1-B2	(GW); WELL GRADED GRAVEL WITH SAND; Field Estimate: 5% fines, 25% fine to coarse sand, 70% fine to coarse gravel; abundant angular cobbles and boulders up to 18" Ø; light gray (10YR 7/2); medium dense; damp. [Crushed Mine Rock; Fill]
A	3.0-4.0 4.0	T19-2-B1 T19-2-L1	(SC); CLAYEY SAND WITH GRAVEL; Field Estimate: 40% clay, 35% fine to coarse sand, 25% fine to coarse gravel; yellowish red (5YR 5/8); medium dense to dense; damp. [Native]

⌇—— Pocket Penetrometer, Tons Per Square Foot (TSF)

Scale: 1" = 5'

792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

Project Name: Idaho-Maryland Gold Project - Brunswick Site
Location: Grass Valley, California
Excavation Co: Siller & Siller (Gavin)
Excavator: Hyundai 60CR-9A
Excavation Method: 24-inch Bucket

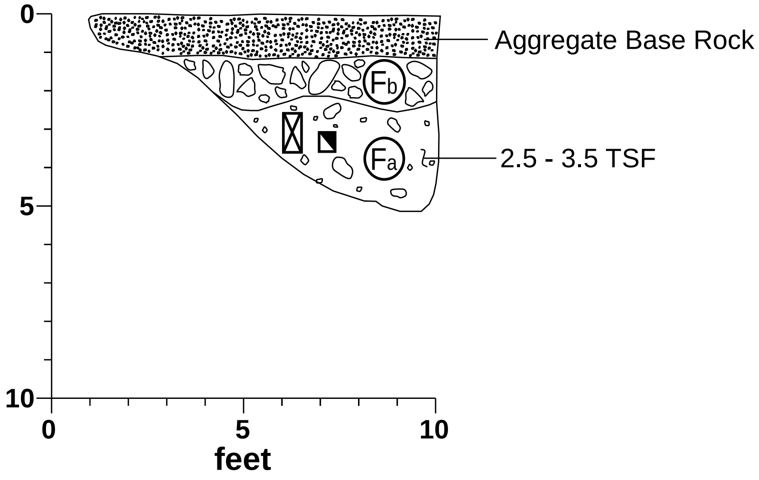
Project No.: 5270.02
Date: 9/25/19
Logged By: DAV
Sampling Method: Hand / Bulk

LOG OF TRENCH

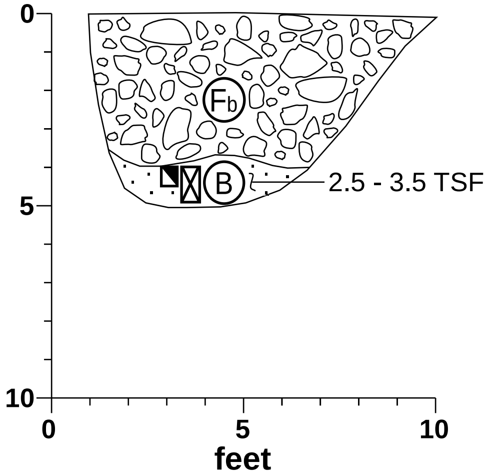
T19-1
T19-2

LOG OF TRENCH T19-3 & T19-4

T19-3



T19-4



Unit	Sample		Soil and/or Rock Material Description
	Depth (ft)	No.	
Ⓕ$_a$	3.0 2.5-3.5	T19-3-L1 T19-3-B1	(SC); CLAYEY SAND WITH GRAVEL; Field Estimate: 40% clay, 35% fine to coarse sand, 25% fine to coarse gravel; occasional cobbles up to 8" Ø; yellowish red (5YR 5/8); medium dense to dense; damp; contains trace amounts of debris (e.g., metal, concrete). [Heterogenous Fill]
Ⓕ$_b$			(GW); WELL GRADED GRAVEL WITH SAND; Field Estimate: 5% fines, 25% fine to coarse sand, 70% fine to coarse gravel; abundant angular cobbles and boulders up to 24" Ø; light gray (10YR 7/2); dense; damp; some debris. [Crushed Mine Rock; Fill]
Ⓑ	4.0 4.0-5.0	T19-4-L1 T19-4-B1	(MH); SANDY SILT; Field Estimate: 65% silt, 30% fine to coarse sand, 5% fine gravel; dark reddish brown (5YR 3/3); very stiff; moist. [Residual Soil]

⌇—— Pocket Penetrometer, Tons Per Square Foot (TSF)

Scale: 1" = 5'

NV5

792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

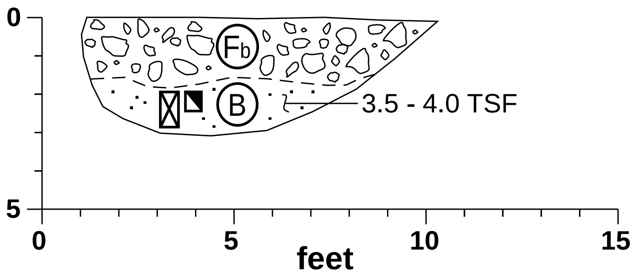
Project Name: Idaho-Maryland Gold Project - Brunswick Site
Location: Grass Valley, California
Project No.: 5270.02
Excavation Co: Siller & Siller (Gavin)
Date: 9/25/19
Excavator: Hyundai 60CR-9A
Logged By: DAV
Excavation Method: 24-inch Bucket
Sampling Method: Hand / Bulk

LOG OF TRENCH

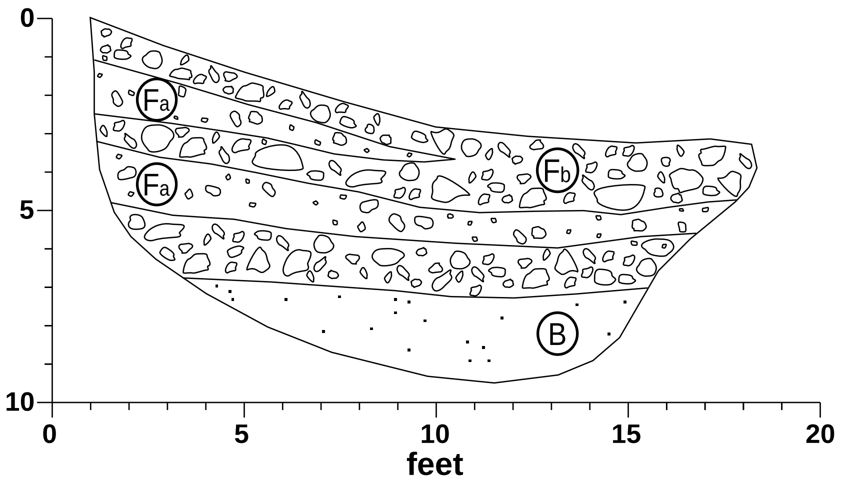
T19-3
T19-4

LOG OF TRENCH T19-5 & T19-6

T19-5



T19-6



Unit	Sample		Soil and/or Rock Material Description
	Depth (ft)	No.	
F_a			(SC); CLAYEY SAND WITH GRAVEL; Field Estimate: 40% clay, 35% fine to coarse sand, 25% fine to coarse gravel; occasional cobbles up to 8" Ø; yellowish red (5YR 5/8); medium dense to dense; damp; contains trace amounts of debris (e.g., metal, concrete). [Heterogenous Fill]
F_b			(GW); WELL GRADED GRAVEL WITH SAND; Field Estimate: 5% fines, 25% fine to coarse sand, 70% fine to coarse gravel; abundant angular cobbles up to 15" Ø; light gray (10YR 7/2); dense; damp; some tailings. [Crushed Mine Rock; Fill]
B	2.0-3.0 2.0	T19-5-B1 T19-5-L1	(MH); SILT WITH SAND; Field Estimate: 75% silt, 20% fine to coarse sand, 5% fine gravel; dark reddish brown (5YR 4/3); very stiff; moist. [Residual Soil]

⊸ Pocket Penetrometer, Tons Per Square Foot (TSF)

Scale: 1" = 5'

N|V|5

792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

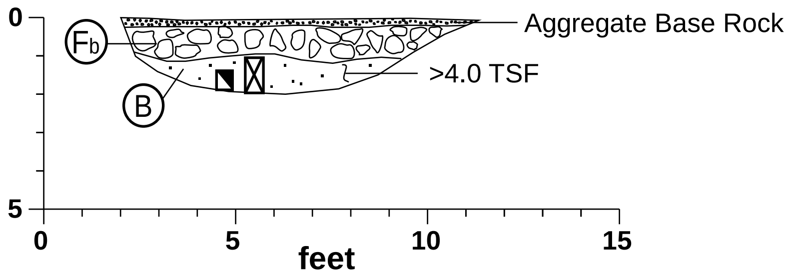
Project Name: Idaho-Maryland Gold Project - Brunswick Site
Location: Grass Valley, California
Project No.: 5270.02
Excavation Co: Siller & Siller (Gavin)
Date: 9/25/19
Excavator: Hyundai 60CR-9A
Logged By: DAV
Excavation Method: 24-inch Bucket
Sampling Method: Hand / Bulk

LOG OF TRENCH

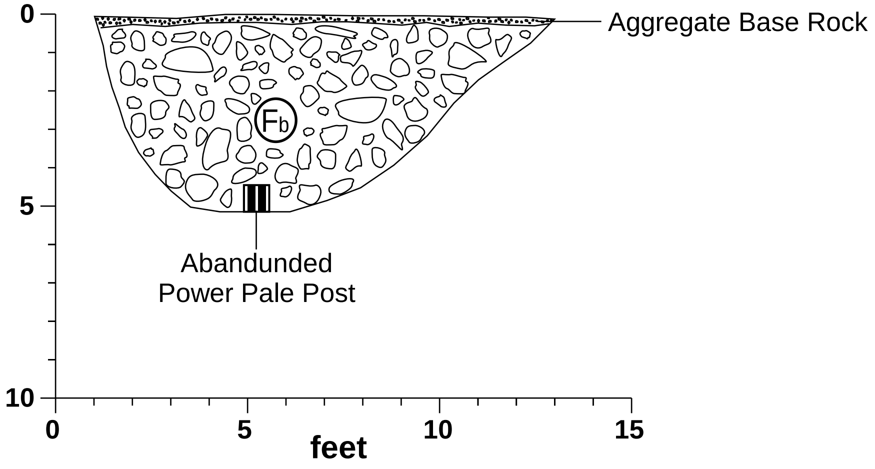
T19-5
T19-6

LOG OF TRENCH T19-7 & T19-8

T19-7



T19-8



Unit	Sample		Soil and/or Rock Material Description
	Depth (ft)	No.	
F$_b$			(GW); WELL GRADED GRAVEL WITH SAND; Field Estimate: 5% fines, 25% fine to coarse sand, 70% fine to coarse gravel; abundant angular cobbles and boulders up to 8" Ø (in T19-7) and 24" Ø (in T19-8); light gray (10YR 7/2); dense; damp. [Crushed Mine Rock; Fill]
B	1.0-2.0 1.5	T19-7-B1 T19-7-L1	(MH); SILT WITH SAND; Field Estimate: 70% silt, 25% fine to coarse sand, 5% fine gravel; dark reddish brown (5YR 4/3); hard; moist. [Residual Soil]

Pocket Penetrometer, Tons Per Square Foot (TSF)

Scale: 1" = 5'

NV5

792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

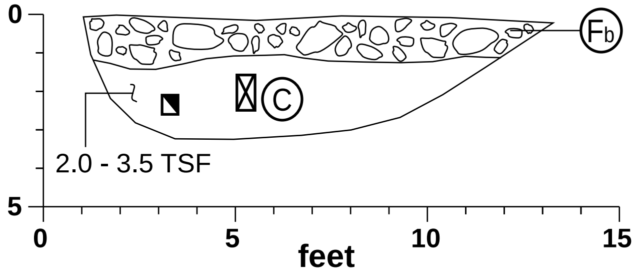
Project Name: Idaho-Maryland Gold Project - Brunswick Site
Location: Grass Valley, California
Project No.: 5270.02
Excavation Co: Siller & Siller (Gavin)
Date: 9/25/19
Excavator: Hyundai 60CR-9A
Logged By: DAV
Excavation Method: 24-inch Bucket
Sampling Method: Hand / Bulk

LOG OF TRENCH

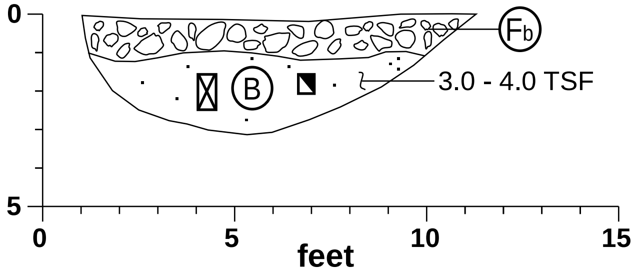
T-19-7
T19-8

LOG OF TRENCH T19-9, T19-10 & T19-11

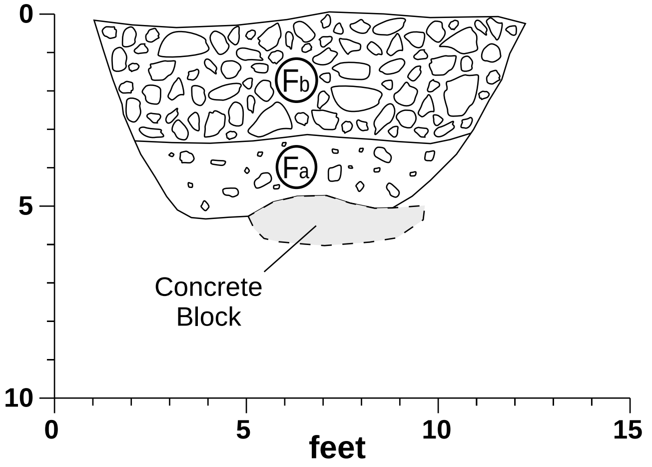
T19-9



T19-10



T19-11



Unit	Sample		Soil and/or Rock Material Description
	Depth (ft)	No.	
F_a			(SC); CLAYEY SAND WITH GRAVEL; Field Estimate: 40% clay, 35% fine to coarse sand, 25% fine to coarse gravel; occasional cobbles up to 8" Ø; yellowish red (5YR 5/8); medium dense to dense; damp; contains trace amounts of debris (e.g., metal, concrete). [Heterogenous Fill]
F_b			(GW); WELL GRADED GRAVEL WITH SAND; Field Estimate: 5% fines, 25% fine to coarse sand, 70% fine to coarse gravel; abundant angular cobbles and boulders up to 18" Ø (in T19-9) and12" Ø (in T19-11); light gray (10YR 7/2); dense; damp; some mine tailings and asphalt/concrete debris. [Crushed Mine Rock; Fill]
B	1.5-2.5 1.5	T19-10-B1 T19-10-L1	(MH); SILT WITH SAND; Field Estimate: 70% silt, 25% fine to coarse sand, 5% fine gravel; dark reddish brown (5YR 4/3); very stiff; moist. [Residual Soil]
C	1.5-2.5 2.0	T19-9-B1 T19-9-L1	(MH); HIGH PLASTICITY SILT WITH SAND; Field Estimate: 80% elastic silt, 15% fine to coarse sand, 5% fine gravel; some angular cobbles and boulders up to 18" Ø; mottled pale yellow (5YR 8/3) to brownish yellow (10YR 6/6); very stiff; moist. [Residual Soil]

⌇— Pocket Penetrometer, Tons Per Square Foot (TSF)

Scale: 1" = 5'

NV5

792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

Project Name: Idaho-Maryland Gold Project - Brunswick Site
Location: Grass Valley, California
Excavation Co: Siller & Siller (Gavin)
Excavator: Hyundai 60CR-9A
Excavation Method: 24-inch Bucket

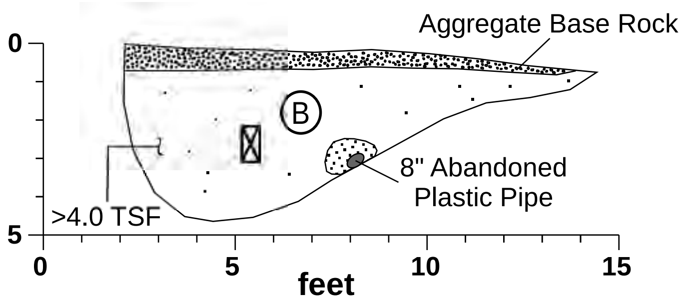
Project No.: 5270.02
Date: 9/25/19
Logged By: DAV
Sampling Method: Hand / Bulk

LOG OF TRENCH

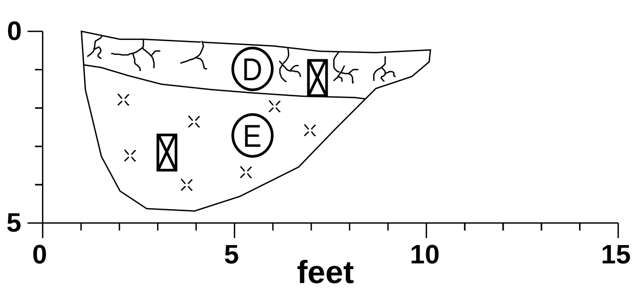
T19-9
T19-10
T19-11

LOG OF TRENCH T19-12, T19-13 & T19-14

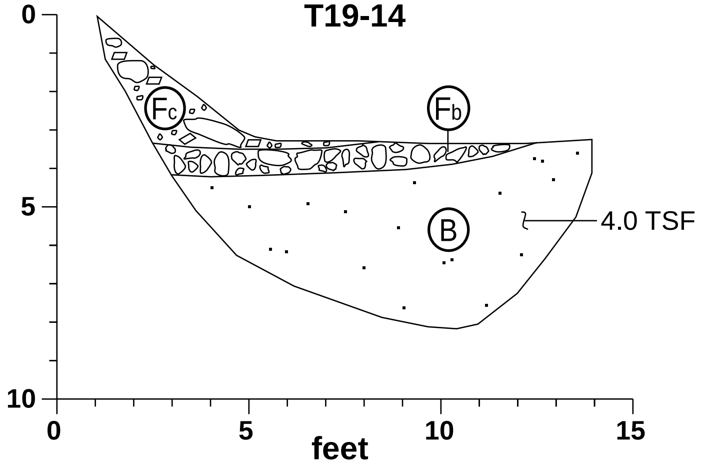
T19-12



T19-13



T19-14



Unit	Sample		Soil and/or Rock Material Description
	Depth (ft)	No.	
Fb			(GW); WELL GRADED GRAVEL WITH SAND; Field Estimate: 5% fines, 25% fine to coarse sand, 70% fine to coarse gravel; abundant angular cobbles and boulders up to 18" Ø; light gray (10YR 7/2); dense; damp. [Crushed Mine Rock; Fill]
Fc			(SC); CLAYEY SAND WITH GRAVEL; Field Estimate: 40% clay, 35% fine to coarse sand, 25% fine to coarse gravel; boulders up to 24" Ø; yellowish red (5YR 5/8); medium dense; damp; contains some bark and metal wire. [Heterogenous Fill with Wood Debris]
B	2.0-3.0	T19-12-B1	(MH); SANDY SILT; Field Estimate: 65% silt, 30% fine to coarse sand, 5% fine gravel; dark reddish brown (5YR 3/3); very stiff; moist. [Residual Soil]
D	0.5-1.5	T19-13-B1	(ML); SANDY SILT WITH GRAVEL; Field Estimate: 60% silt, 30% fine to coarse sand, 10% fine gravel; reddish brown (5YR 4/4); soft to firm; moist; weak and porous with roots/rootlets up to 1" Ø. [Residual Soil]
E	2.5-3.5	T19-13-B2	(Rx); METAVOLCANIC ROCK; weakly to moderately cemented; highly to completely weathered; friable to weak; damp. [Completely Weathered Rock]

⌇—— Pocket Penetrometer, Tons Per Square Foot (TSF)

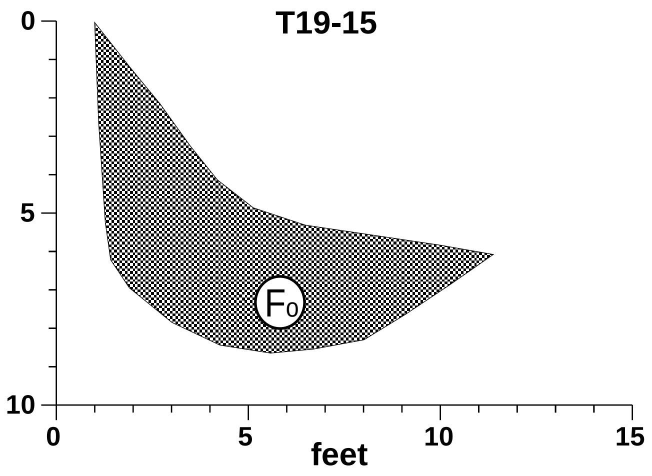
Scale: 1" = 5'

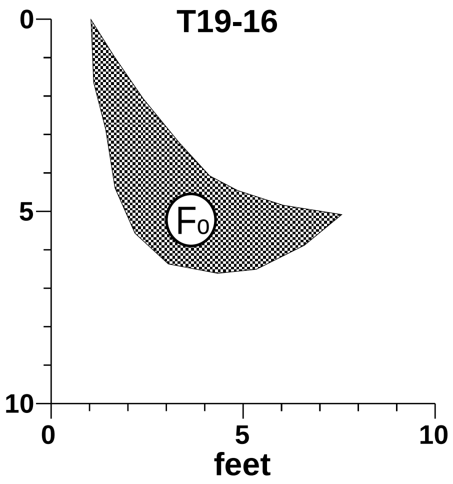
NV5
792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

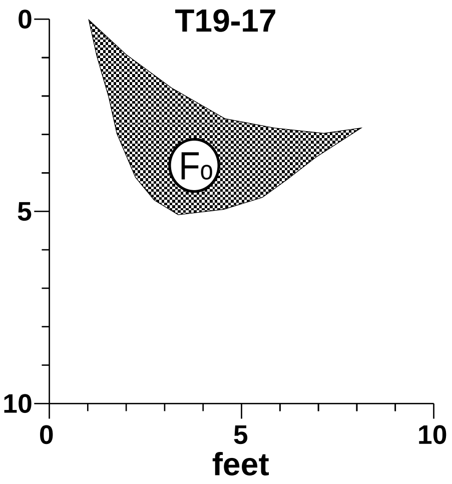
Project Name: Idaho-Maryland Gold Project - Brunswick Site
Location: Grass Valley, California
Project No.: 5270.02
Excavation Co: Siller & Siller (Gavin)
Date: 9/25/19
Excavator: Hyundai 60CR-9A
Logged By: DAV
Excavation Method: 24-inch Bucket
Sampling Method: Hand / Bulk

LOG OF TRENCH
T19-12
T19-13
T19-14

LOG OF TRENCH T19-15, T19-16 & T19-17







Unit	Sample		Soil and/or Rock Material Description
	Depth (ft)	No.	
F_o			(OM); ORGANIC SOIL; Field Estimate: 5% fines, 95% organics (decayed organics and bark); black to brown (7.5YR 3/2); very soft to medium stiff; damp; weak, porous and compressible. [Fill]

Pocket Penetrometer, Tons Per Square Foot (TSF)

Scale: 1" = 5'

792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

Project Name: Idaho-Maryland Gold Project - Brunswick Site
Location: Grass Valley, California
Project No.: 5270.02
Excavation Co: Siller & Siller (Gavin)
Date: 9/25/19
Excavator: Hyundai 60CR-9A
Logged By: DAV
Excavation Method: 24-inch Bucket
Sampling Method: Hand / Bulk

LOG OF TRENCH

T19-15
T19-16
T19-17

APPENDIX D

Laboratory Test Data

NV5

UNCONFINED COMPRESSION

ASTM D2166

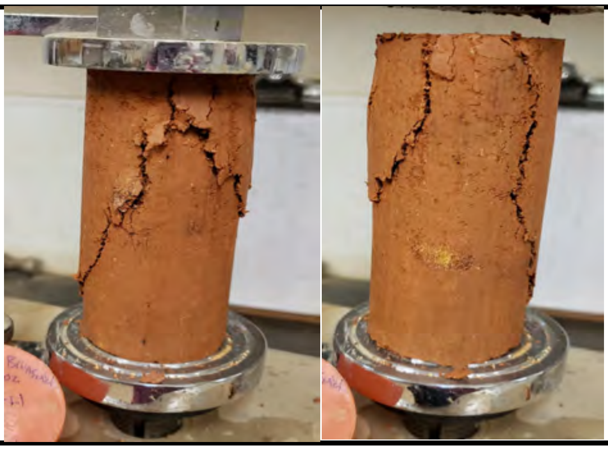
DSA File #:

DSA Appl #:

Project No.:	5279	Project Name:	Idaho Maryland Mine			Date:	11/1/2019
Sample No.:	T19-2-L1	Boring/Trench No.:	T19-2	Depth (ft.)	4	Tested By:	MLH
Soil Description:	Yellowish Red (5YR 4/6) Sandy Silt					Check By:	MLH
Sample Location:						Lab No.:	15-19-546

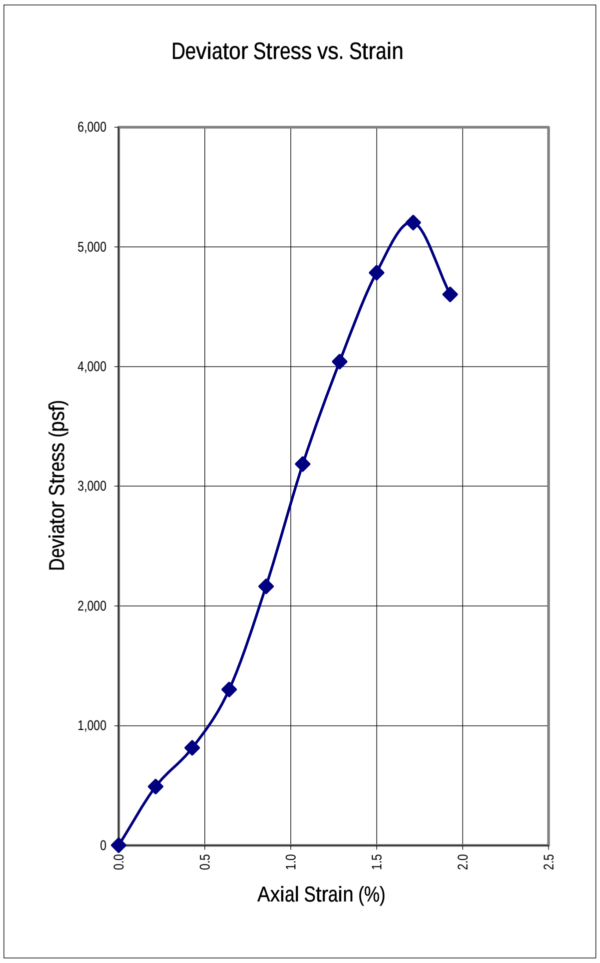
Sample Data		
Tare Tube Number	I.D.	CTP
Tare Weight	(gm)	409.09
Wet Soil + Tare	(gm)	776.05
Dry Soil + Tare	(gm)	684.25
Weight of Water	(gm)	91.80
Dry Soil Weight	(gm)	275.16
Moisture Content	(%)	33.36
Soil Height	(cm)	11.86
Sample Diameter	(cm)	4.90
Wet Unit Weight	(pcf)	102.44
Dry Unit Weight	(pcf)	76.81
Specific Gravity	(dim)	2.70
Saturation	(%)	75.48
Strain Rate	(%)	1.22
Proving Ring Constant	(lbs/unit)	1.108

Sample Sketch At Failure



Unconfined Shear Strength = 2,602.1 psf

Elapsed Time (Minutes)	Strain Units (0.001in/unit)	Strain Percent (%)	Area (cm^2)	Load Dial (units)	Load Force (lbs)	Deviator Stress (psf)
12:00:00	0	0.00	0.00	0	0.00	0.00
12:00:10	10	0.21	18.90	9	9.97	490.23
12:00:20	20	0.43	18.94	15	16.62	815.29
12:00:30	30	0.64	18.98	24	26.59	1301.67
12:00:40	40	0.86	19.02	40	44.32	2164.77
12:00:50	50	1.07	19.06	59	65.37	3186.13
12:01:00	60	1.28	19.10	75	83.10	4041.40
12:01	70	1.50	19.14	89	98.61	4785.39
12:01:20	80	1.71	19.19	97	107.48	5204.20
12:01:30	90	1.93	19.23	86	95.29	4603.98



NV5

UNCONFINED COMPRESSION

ASTM D2166

DSA File #:

DSA Appl #:

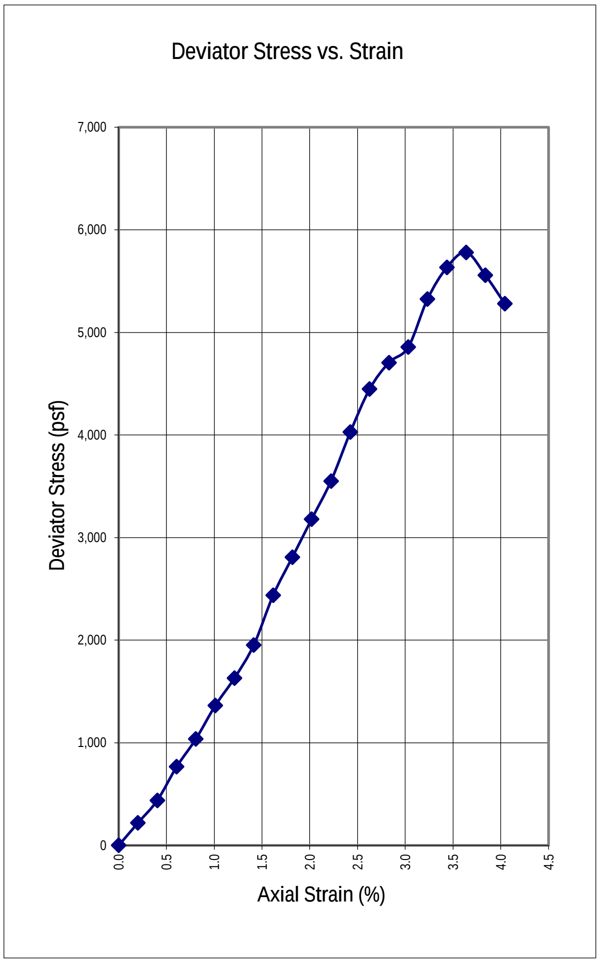
Project No.:	5279	Project Name:	Idaho Maryland Mine	Date:	11/1/2019
Sample No.:	T19-4-L1	Boring/Trench No.: T19-4	Depth (ft.) 4	Tested By:	MLH
Soil Description:	Dark Reddish Brown (2.5YR 2.5/4) Sandy Silt			Check By:	MLH
Sample Location:				Lab No.:	15-19-546

Sample Data		
Tare Tube Number	I.D.	2
Tare Weight	(gm)	87.76
Wet Soil + Tare	(gm)	535.72
Dry Soil + Tare	(gm)	444.13
Weight of Water	(gm)	91.59
Dry Soil Weight	(gm)	356.37
Moisture Content	(%)	25.70
Soil Height	(cm)	12.57
Sample Diameter	(cm)	4.88
Wet Unit Weight	(pcf)	118.96
Dry Unit Weight	(pcf)	94.63
Specific Gravity	(dim)	2.70
Saturation	(%)	88.93
Strain Rate	(%)	1.21
Proving Ring Constant	(lbs/unit)	1.108

Sample Sketch At Failure

Unconfined Shear Strength = 2,890.3 psf

Elapsed Time (Minutes)	Strain Units (0.001in/unit)	Strain Percent (%)	Area (cm^2)	Load Dial (units)	Load Force (lbs)	Deviator Stress (psf)
12:00:00	0	0.00	0.00	0	0.00	0.00
12:00:10	10	0.20	18.74	4	4.43	219.70
12:00:20	20	0.40	18.78	8	8.86	438.50
12:00:30	30	0.61	18.82	14	15.51	765.82
12:00:40	40	0.81	18.86	19	21.05	1037.22
12:00:50	50	1.01	18.89	25	27.70	1361.98
12:01:00	60	1.21	18.93	30	33.24	1631.04
12:01:10	70	1.41	18.97	36	39.89	1953.24
12:01:20	80	1.62	19.01	45	49.86	2436.55
12:01:30	90	1.82	19.05	52	57.62	2809.78
12:01:40	100	2.02	19.09	59	65.37	3181.46
12:01:50	110	2.22	19.13	66	73.13	3551.58
12:02:00	120	2.42	19.17	75	83.10	4027.55
12:02:10	130	2.63	19.21	83	91.96	4447.92
12:02:20	140	2.8289578	19.248313	88	97.50	4706.0842
12:02:30	150	3.0310263	19.288423	91	100.83	4856.3989
12:02:40	160	3.2330947	19.328701	100	110.80	5325.58122
12:02:50	170	3.4351631	19.369148	106	117.45	5633.32798
12:03:00	180	3.6372315	19.409764	109	120.77	5780.64004
12:03:10	190	3.8392999	19.450551	105	116.34	5556.82956
12:03:20	200	4.0413683	19.49151	100	110.80	5281.09776



UNCONFINED COMPRESSION

ASTM D2166

DSA File #:

DSA Appl #:

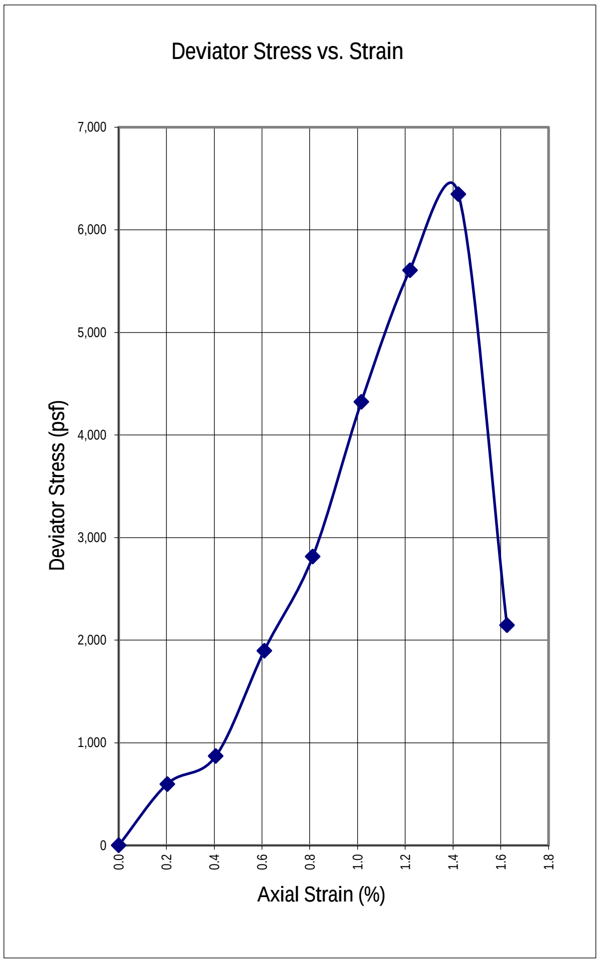
Project No.:	5279	Project Name:	Idaho Maryland Mine			Date:	11/1/2019
Sample No.:	T19-10-L1	Boring/Trench No.:	T19-10	Depth (ft.)	1.5	Tested By:	MLH
Soil Description:	Yellowish Red (5YR 4/6) Sandy Silt					Check By:	MLH
Sample Location:						Lab No.:	15-19-546

Sample Data		
Tare Tube Number	I.D.	TB
Tare Weight	(gm)	396.88
Wet Soil + Tare	(gm)	758.08
Dry Soil + Tare	(gm)	658.47
Weight of Water	(gm)	99.61
Dry Soil Weight	(gm)	261.59
Moisture Content	(%)	38.08
Soil Height	(cm)	12.50
Sample Diameter	(cm)	4.90
Wet Unit Weight	(pcf)	95.67
Dry Unit Weight	(pcf)	69.29
Specific Gravity	(dim)	2.70
Saturation	(%)	71.81
Strain Rate	(%)	0.96
Proving Ring Constant	(lbs/unit)	1.108

Sample Sketch At Failure

Unconfined Shear Strength = 3,174.8 psf

Elapsed Time (Minutes)	Strain Units (0.001in/unit)	Strain Percent (%)	Area (cm^2)	Load Dial (units)	Load Force (lbs)	Deviator Stress (psf)
12:00:00	0	0.00	0.00	0	0.00	0.00
12:00:10	10	0.20	18.90	11	12.19	599.23
12:00:20	20	0.41	18.93	16	17.73	869.84
12:00:30	30	0.61	18.97	35	38.78	1898.89
12:00:40	40	0.81	19.01	52	57.62	2815.44
12:00:50	50	1.02	19.05	80	88.64	4322.58
12:01:00	60	1.22	19.09	104	115.23	5607.81
12:01	70	1.42	19.13	118	130.74	6349.62
12:01:20	80	1.63	19.17	40	44.32	2147.98



UNCONFINED COMPRESSION

ASTM D2166

DSA File #:

DSA Appl #:

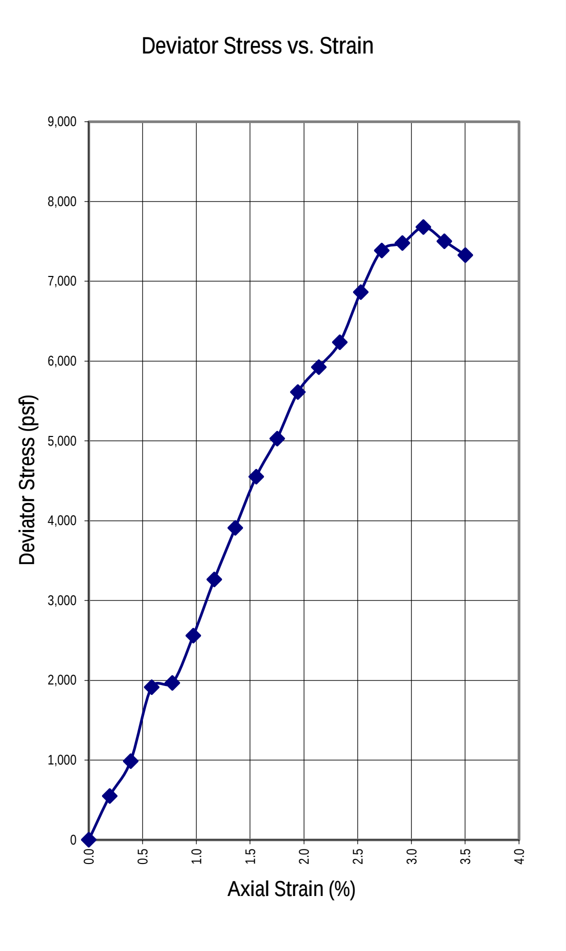
Project No.:	5279	Project Name:	Idaho Maryland Mine			Date:	11/1/2019
Sample No.:	T19-7-L1	Boring/Trench No.:	T19-7	Depth (ft.)	1.5	Tested By:	MLH
Soil Description:	Yellowish Red (5YR 5/8) Sandy Silt					Check By:	MLH
Sample Location:						Lab No.:	15-19-546

Sample Data		
Tare Tube Number	I.D.	11
Tare Weight	(gm)	253.26
Wet Soil + Tare	(gm)	696.41
Dry Soil + Tare	(gm)	578.02
Weight of Water	(gm)	118.39
Dry Soil Weight	(gm)	324.76
Moisture Content	(%)	36.45
Soil Height	(cm)	13.06
Sample Diameter	(cm)	4.88
Wet Unit Weight	(pcf)	113.26
Dry Unit Weight	(pcf)	83.00
Specific Gravity	(dim)	2.70
Saturation	(%)	95.58
Strain Rate	(%)	1.17
Proving Ring Constant	(lbs/unit)	1.108

Sample Sketch At Failure

Unconfined Shear Strength = 3,839.2 psf

Elapsed Time (Minutes)	Strain Units (0.001in/unit)	Strain Percent (%)	Area (cm^2)	Load Dial (units)	Load Force (lbs)	Deviator Stress (psf)
12:00:00	0	0.00	0.00	0	0.00	0.00
12:00:10	10	0.19	18.74	10	11.08	549.28
12:00:20	20	0.39	18.78	18	19.94	986.78
12:00:30	30	0.58	18.81	35	38.78	1914.99
12:00:40	40	0.78	18.85	36	39.89	1965.85
12:00:50	50	0.97	18.89	47	52.08	2561.50
12:01:00	60	1.17	18.92	60	66.48	3263.58
12:01	70	1.36	18.96	72	79.78	3908.58
12:01:20	80	1.56	19.00	84	93.07	4551.02
12:01:30	90	1.75	19.04	93	103.04	5028.68
12:01	100	1.94	19.07	104	115.23	5612.34
12:01:50	110	2.14	19.11	110	121.88	5924.35
12:02:00	120	2.33	19.15	116	128.53	6235.08
12:02	130	2.53	19.19	128	141.82	6866.39
12:02:20	140	2.7228178	19.227311	138	152.90	7388.05686
12:02:30	150	2.9173047	19.265829	140	155.12	7480.14507
12:02	160	3.1117917	19.304502	144	159.55	7678.45029
12:02:50	170	3.3062787	19.34333	141	156.23	7503.39047
12:03:00	180	3.5007657	19.382315	138	152.90	7328.97287



UNCONFINED COMPRESSION

ASTM D2166

DSA File #:

DSA Appl #:

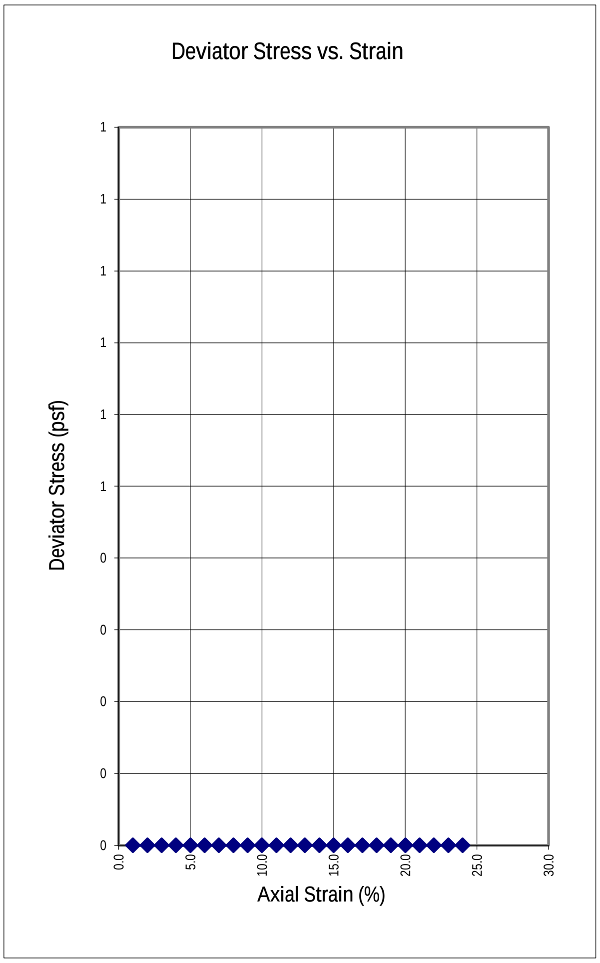
Project No.:	5279	Project Name:	Idaho Maryland Mine			Date:	11/1/2019
Sample No.:	A	Boring/Trench No.:	B17-01	Depth (ft.)	49	Tested By:	0
Soil Description:	TOO Rocky					Check By:	0
Sample Location:						Lab No.:	15-19-546

Sample Data		
Tare Tube Number	I.D.	
Tare Weight	(gm)	
Wet Soil + Tare	(gm)	
Dry Soil + Tare	(gm)	
Weight of Water	(gm)	0.00
Dry Soil Weight	(gm)	0.00
Moisture Content	(%)	
Soil Height	(cm)	
Sample Diameter	(cm)	
Wet Unit Weight	(pcf)	
Dry Unit Weight	(pcf)	
Specific Gravity	(dim)	2.70
Saturation	(%)	
Strain Rate	(%)	0.01
Proving Ring Constant	(lbs/unit)	1.108

Sample Sketch At Failure

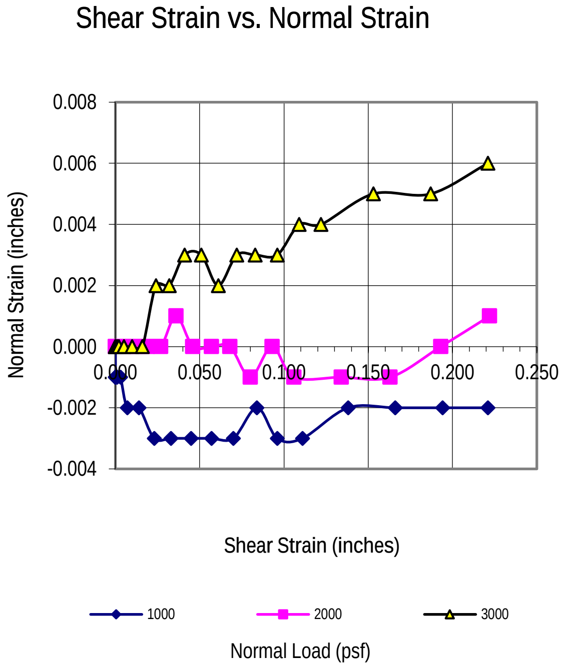
Unconfined Shear Strength = 0.0 psf

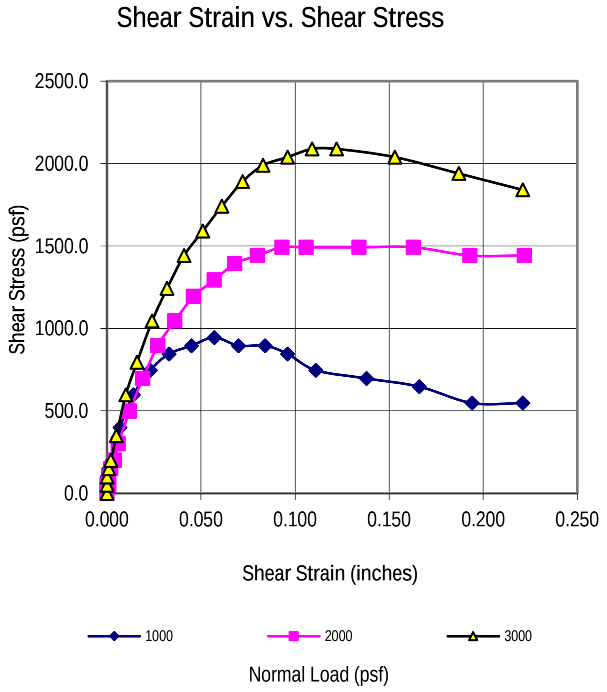
Elapsed Time (Minutes)	Strain Units (0.001in/unit)	Strain Percent (%)	Area (cm^2)	Load Dial (units)	Load Force (lbs)	Deviator Stress (psf)
12:00:00					0.00	
12:00:30					0.00	
12:01:00					0.00	
12:01:30					0.00	
12:02:00					0.00	
12:02:30					0.00	
12:03:00					0.00	
12:03					0.00	
12:04:00					0.00	
12:04:30					0.00	
12:05:00					0.00	
12:05:30					0.00	
12:06:00					0.00	
12:06:30					0.00	
12:07					0.00	
12:07:30					0.00	
12:08:00					0.00	
12:08:30					0.00	
12:09:00					0.00	
12:09:30					0.00	
12:10:00					0.00	
12:10:30					0.00	
12:11:00					0.00	
12:11:30					0.00	

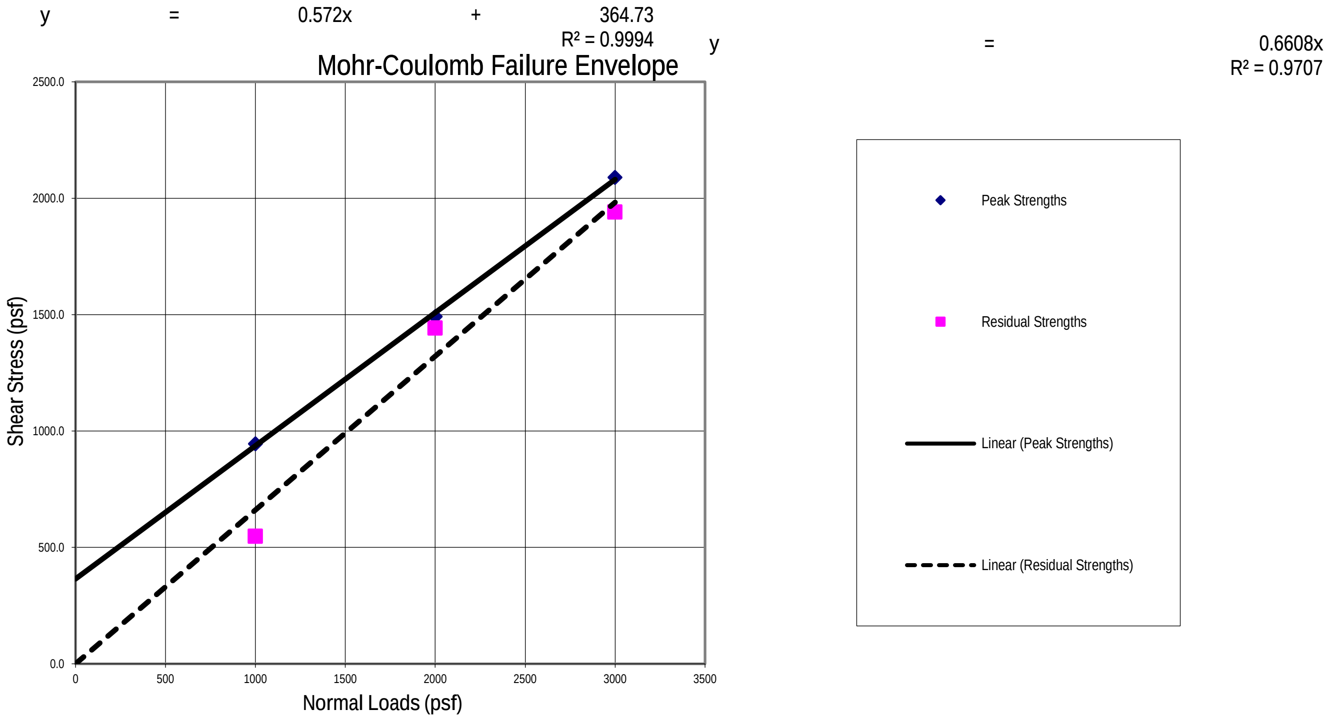


DIRECT SHEAR

ASTM D3080







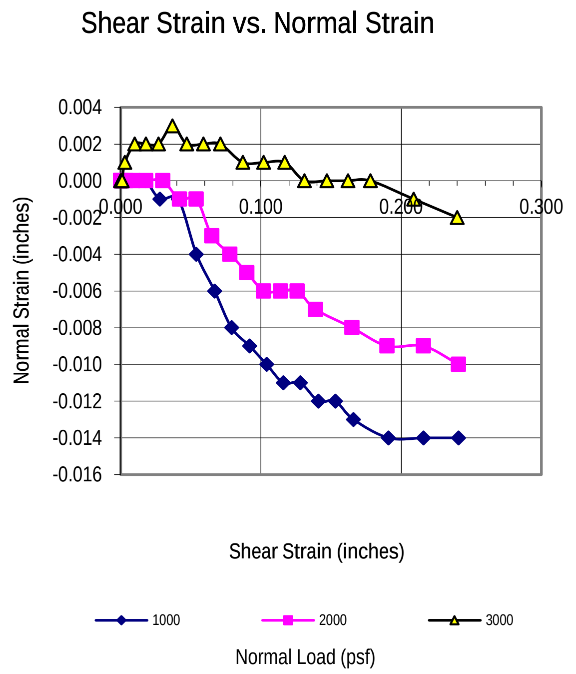
SHEAR STRENGTH TEST RESULTS		
PARAMETERS	**PEAK STRENGTH:**	**RESIDUAL STRENGTH:**
FRICTION ANGLE, (Degree)	29.8	33.5
COHESION, (psf)	365.0	0.0

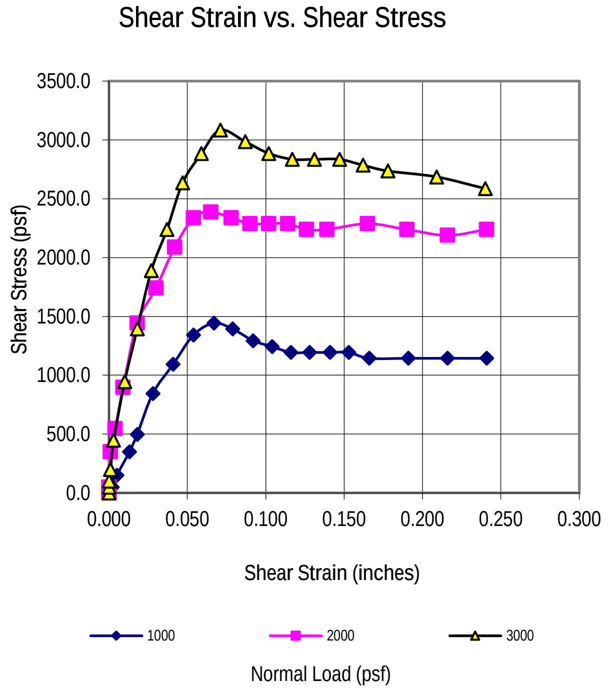
PROJECT NAME: Idaho Maryland Mine
PROJECT NO.: 5279.02
BORING / TRENCH NO.: T19-9
SAMPLE NO.: T19-9-L1
DESCRIPTION: Pale Yellow (2.5Y 7/4) Elastic Silt with Sand

DATE: 11/1/2019
LAB NO.: 15-19-546
SAMPLE DEPTH (ft.): 0.0

DIRECT SHEAR

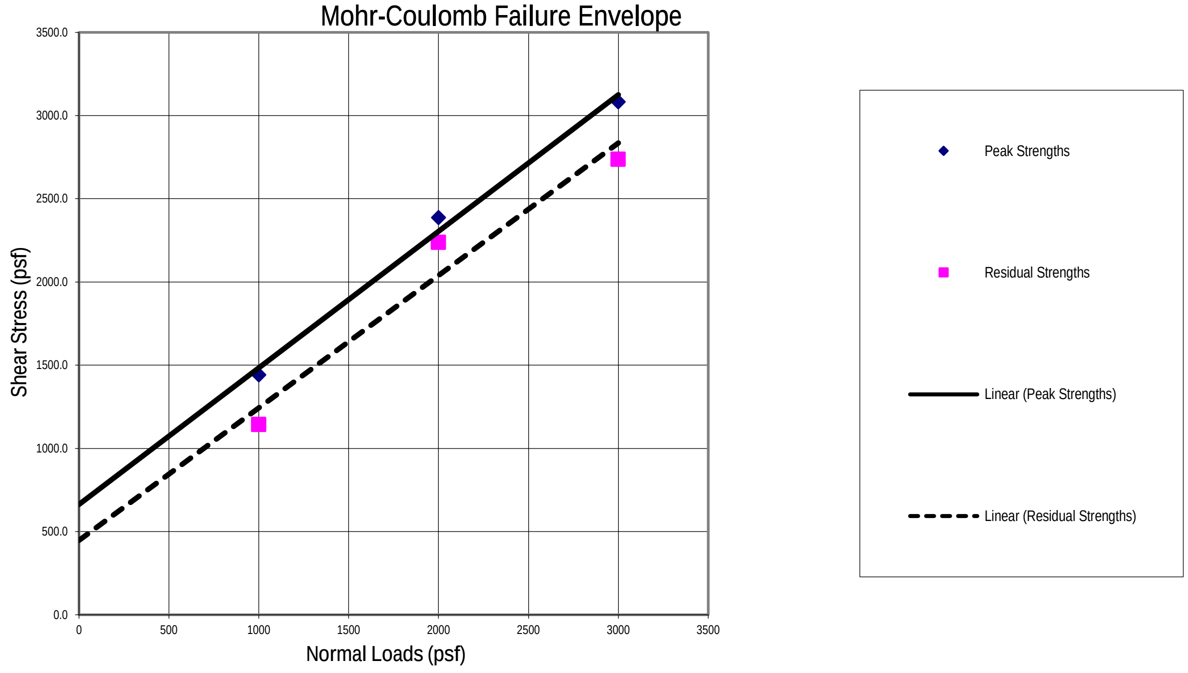
ASTM D3080





y = 0.8206x + 663.15
R² = 0.9924

y = 0.796x + 447.33
R² = 0.9554



SHEAR STRENGTH TEST RESULTS		
PARAMETERS	PEAK STRENGTH:	RESIDUAL STRENGTH:
FRICTION ANGLE, (Degree)	39.4	38.5
COHESION, (psf)	663.0	447.0

PROJECT NAME: Idaho Maryland Mine
PROJECT NO.: 5279.02
BORING / TRENCH NO.: T19-3
SAMPLE NO.: T19-3-L1
DESCRIPTION: Dark Reddish Brown (2.5YR 2.5/4) Sandy Silt

DATE: 11/1/2019
LAB NO.: 15-19-546
SAMPLE DEPTH (ft.): 3

MOISTURE & DENSITY

ASTM D2216 & D2937

DSA File #:
DSA Appl #:

Project No.: 5279.02 Project Name: Idaho Maryland Mine Date: 11/1/2019
Lab No.: 15-19-546 Performed By: MLH Checked By: MLH

SAMPLE LOCATION DATA										
Boring/Trench No.	Units	T19-2	T19-3	T19-4	T19-7	T19-9	T19-10			
Sample No.		T19-2-	T19-3-L1	T19-4-L1	T19-7-L1	T19-9-L1	T19-10-L1			
Depth Interval	(ft.)	4	3	4	1.5	2	1.5			
Sample Description		Yellowish Red (5YR 4/6) Sandy Silt	Dark Reddish Brown (2.5YR 2.5/4) Sandy Silt	Dark Reddish Brown (2.5YR 2.5/4) Sandy Silt	Yellowish Red (5YR 5/8) Sandy Silt	Pale Yellow (2.5Y 7/4) Elastic Silt with Sand	Yellowish Red (5YR 4/6) Sandy Silt			
USCS Symbol										
SAMPLE DIMENSION AND WEIGHT DATA										
Sample Length	(in)	5.220	5.290	5.940	5.140	5.910	5.440			
Sample Diameter	(in)	1.930	1.930	1.920	1.920	1.910	1.930			
Sample Volume	(cf)	0.0088	0.0090	0.0100	0.0086	0.0098	0.0092			
Wet Soil + Tube Wt.	(gr)	575.43	659.61	702.47	605.85	656.60	582.91			
Tube Wt.	(gr)	161.24	178.56	159.85	161.39	162.01	165.15			
Wet Soil Wt.	(gr)	414.19	481.05	542.62	444.46	494.59	417.76			
MOISTURE CONTENT DATA										
Tare No.		CTP	TA	2	11	MT	TB			
Tare Wt.	(gr)	409.09	50.75	87.76	253.26	50.99	396.88			
Wet Soil + Tare Wt.	(gr)	776.05	142.54	535.72	696.41	134.05	758.08			
Dry Soil + Tare Wt.	(gr)	684.25	122.68	444.13	578.02	109.79	658.47			
Water Wt.	(gr)	91.80	19.86	91.59	118.39	24.26	99.61			
Dry Soil Wt.	(gr)	275.16	71.93	356.37	324.76	58.80	261.59			
Moisture Content	(%)	33.4	27.6	25.7	36.5	41.3	38.1			
TEST RESULTS										
Wet Unit Wt.	(pcf)	103.3	118.4	120.2	113.8	111.3	100.0			
Moisture Content	(%)	33.4	27.6	25.7	36.5	41.3	38.1			
Dry Unit Wt.	(pcf)	77.5	92.8	95.6	83.4	78.8	72.4			
MOISTURE CORRECTION DATA										
Gauge Moisture	(%)									
K Value Correction Factor										
COMPACTION CURVE DATA (ASTM D698, ASTM D1557, or CAL216)										
Test Method										
Curve No.										
Max Wet Unit Wt.	(pcf)									
Max Dry Unit Wt.	(pcf)									
Optimum Moisture	(%)									
Wet Relative Comp.	(%)									
Dry Relative Comp.	(%)									

ATTERBERG INDICES

ASTM D4318

DSA File #:

DSA Appl #:

Project No.:	**5279.02**	Project Name:	**Idaho Maryland Mine**			Date:	**11/1/2019**
Sample No.:	**T19-9-B1**	Boring/Trench:	**T19-9**	Depth, (ft.):	**1.5-2.5**	Tested By:	**SJS**
Description:	**Pale Yellow (2.5Y 7/4) Elastic Silt with Sand**					Checked By:	**MLH**
Sample Location:						Lab. No.:	**15-19-546**

Estimated % of Sample Retained on No. 40 Sieve: 10

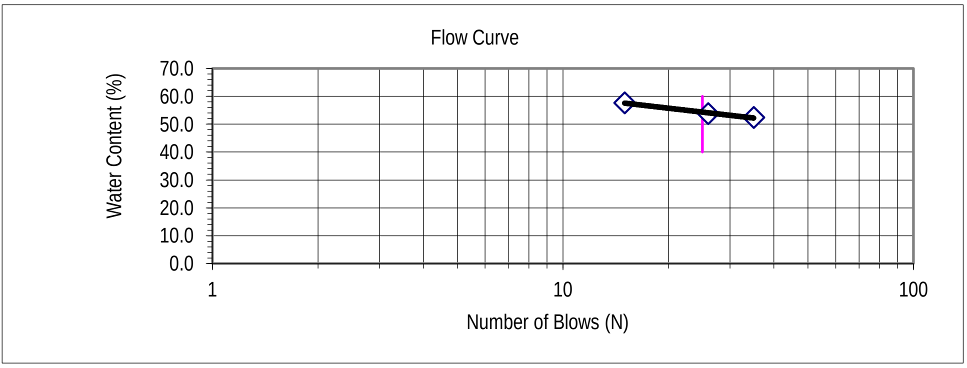
Sample Air Dried: yes

Test Method A or B: A

	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	45	36	32			40	22	
Wt. Pan (gr)	21.16	21.54	21.17			21.60	21.93	
Wt. Wet Soil + Pan (gr)	35.09	32.12	32.68			28.22	28.06	
Wt. Dry Soil + Pan (gr)	30.30	28.42	28.47			26.02	26.49	
Wt. Water (gr)	4.79	3.70	4.21			2.20	1.57	
Wt. Dry Soil (gr)	9.14	6.88	7.30			4.42	4.56	
Water Content (%)	52.4	53.8	57.7			49.8	34.4	
Number of Blows, N	35	26	15					

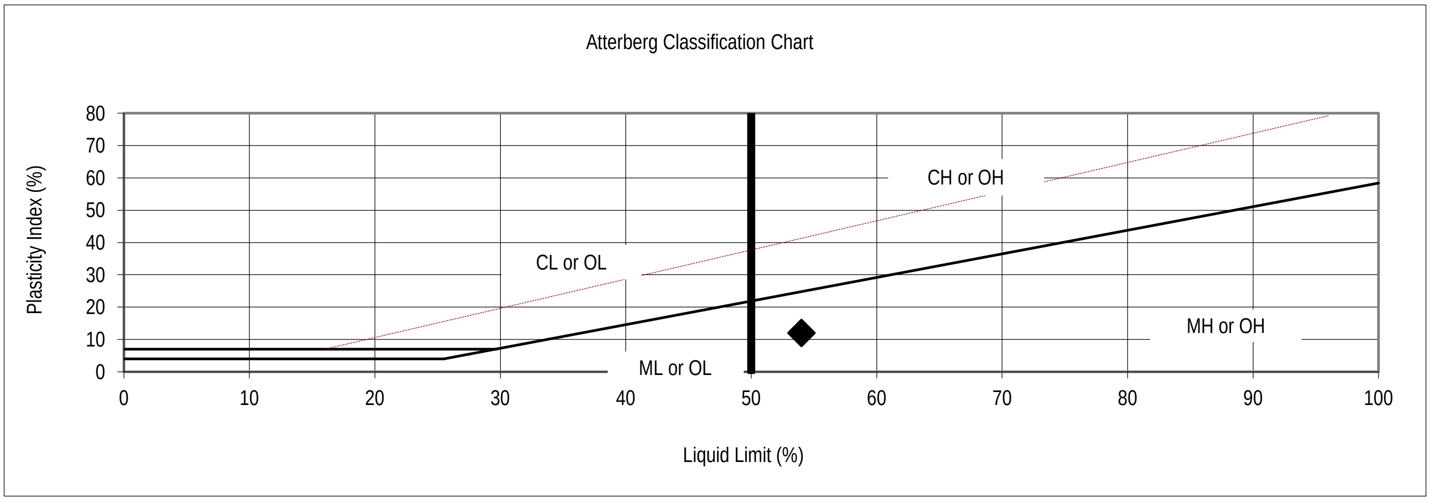
LIQUID LIMIT = 54

PLASTIC LIMIT = 42



Plasticity Index = 12

Group Symbol = MH



ATTERBERG INDICES

ASTM D4318

DSA File #:

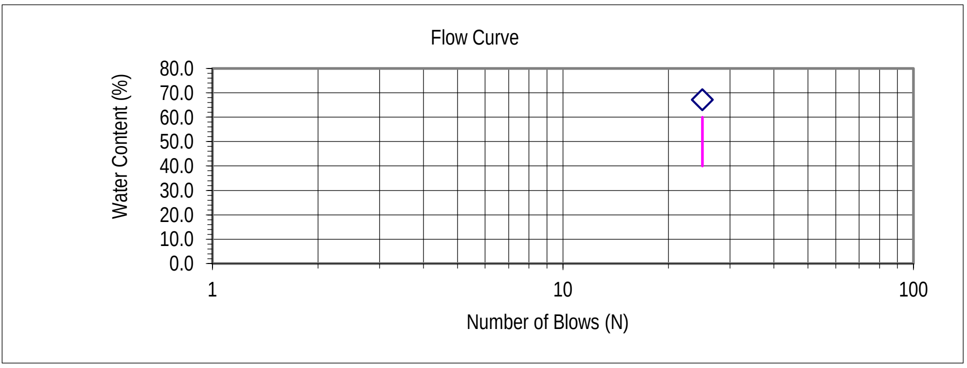
DSA Appl #:

Project No.: **5279.02** Project Name: **Idaho Maryland Mine** Date: **11/1/2019**
Sample No.: **T19-10-B1** Boring/Trench: **T19-10** Depth, (ft.): **1.5-2.5** Tested By: **SJS**
Description: **Dark Reddish Brown (2.5YR 2.5/4) Elastic Silt with Sand** Checked By: **MLH**
Sample Location: Lab. No.: **15-19-546**

Estimated % of Sample Retained on No. 40 Sieve: 10
Sample Air Dried: yes
Test Method A or B: A

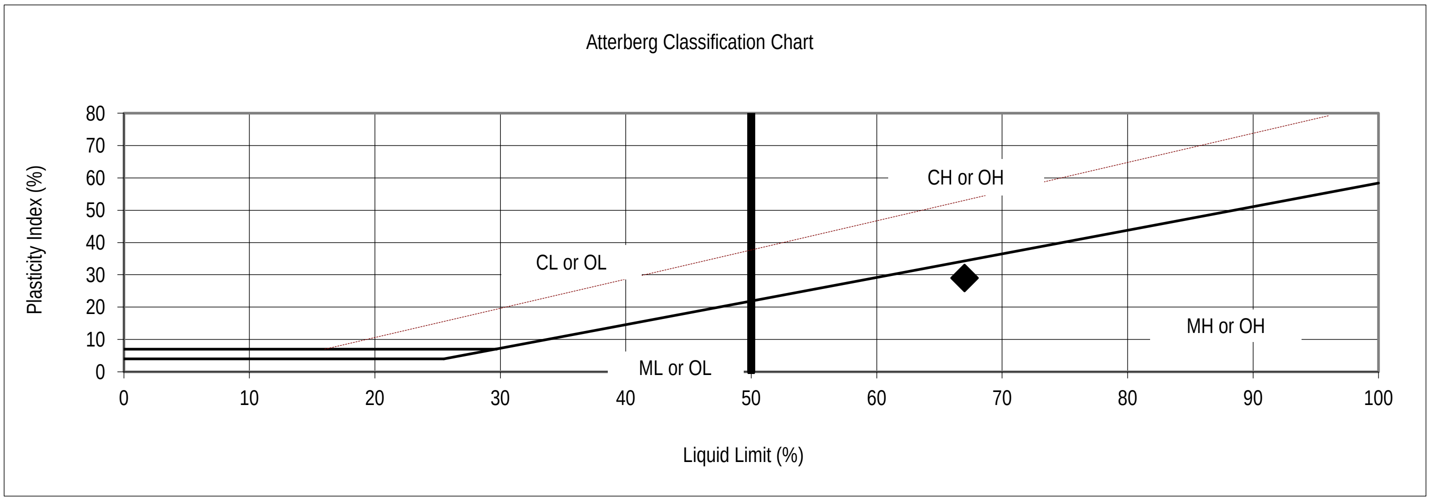
	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	38					11	19	
Wt. Pan (gr)	21.25					29.99	22.25	
Wt. Wet Soil + Pan (gr)	35.04					36.31	29.09	
Wt. Dry Soil + Pan (gr)	29.50					34.55	27.21	
Wt. Water (gr)	5.54					1.76	1.88	
Wt. Dry Soil (gr)	8.25					4.56	4.96	
Water Content (%)	67.2					38.6	37.9	
Number of Blows, N	25							

LIQUID LIMIT = 67
PLASTIC LIMIT = 38



Plasticity Index = 29

Group Symbol = MH



EXPANSION INDEX/SWELL

ASTM D4829

DSA File #:
DSA Appl #:

Project No.: 5279.02 Project Name: Idaho Maryland Mine Date: 11/1/2019
Sample No.: T19-9-B1 Boring/Trench No.: T19-9 Depth (ft.) 1.5-2.5 Tested By: MLH
Soil Description: Pale Yellow (2.5Y 7/4) Elastic Silt with Sand Checked By: MLH
Estimated % of sample retained on #4: 0 Notes: Lab. No.: 15-19-546

Specimen Type: Undisturbed: Disturbed: Remolded to: ASTM Guidelines
Tube Dia. (Inch) = Ring Dia. (Inch) = 4 Ring Height (Inch) = 1.00

FIELD DATA Tube Sample Moisture & Density		
Tare Tube Number		
Tare Weight	(gr)	
Wet Soil + Tare	(gr)	
Dry Soil + Tare	(gr)	
Weight of Water	(gr)	0.00
Dry Soil Weight	(gr)	0.00
Moisture Content	(%)	0.00
Soil Height	(In.)	
Wet Unit Weight	(pcf)	
Dry Unit Weight	(pcf)	
Specific Gravity		2.7

LAB DATA	Test wt.	144	Test wt.		Test wt.	
	Initial	Final	Initial	Final	Initial	Final
Tare Number	B10					
Tare Ring Weight (gr)	368.56	368.56				
Tare Pan Weight (gr)	0.00	270.72				
Wet Soil + Tare (gr)	710.74	1046.64				
Dry Soil + Tare (gr)	658.25	928.97	0.00		0.00	
Weight of Water (gr)	52.49	117.67	0.00	0.00	0.00	0.00
Dry Soil Weight (gr)	289.69	289.69	0.00	0.00	0.00	0.00
Moisture Content (%)	18.12	40.62	0.00	0.00	0.00	0.00
Wet Unit Weight (pcf)	103.75	112.30				
Dry Unit Weight (pcf)	87.83	79.86				
Sample Height (Inches)	1.00	1.100				
Percent Saturation	53.28	98.83				

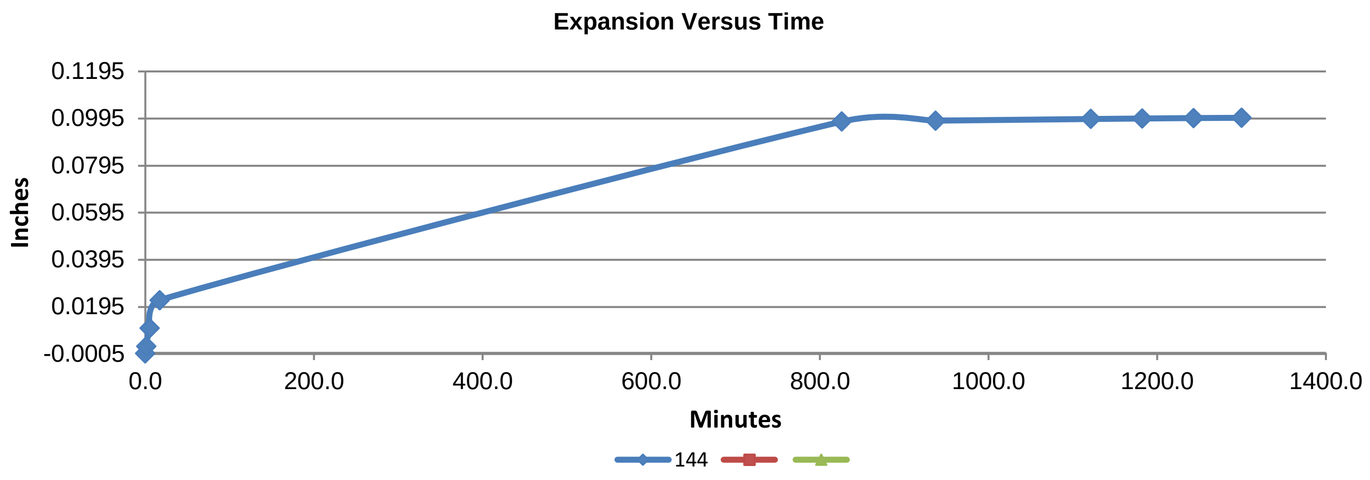
Expansion Index Number

Surcharge (psf)		Uncorrected	Corrected to 50% Saturation
Test wt.	144	100	103
Test wt.			
Test wt.			

Elapsed Time (m:s)	Change in Height (Inches)	Elapsed Time (m:s)	Change in Height (Inches)	Elapsed Time (m:s)	Change in Height (Inches)
0.0	-0.0002				
1.0	0.0027				
5.0	0.0105				
17.0	0.0224				
826.0	0.0982				
937.0	0.0986				
1121.0	0.0993				
1182.0	0.0995				
1243.0	0.0997				
1300.0	0.0998				

Expansion Index Values and Descriptions

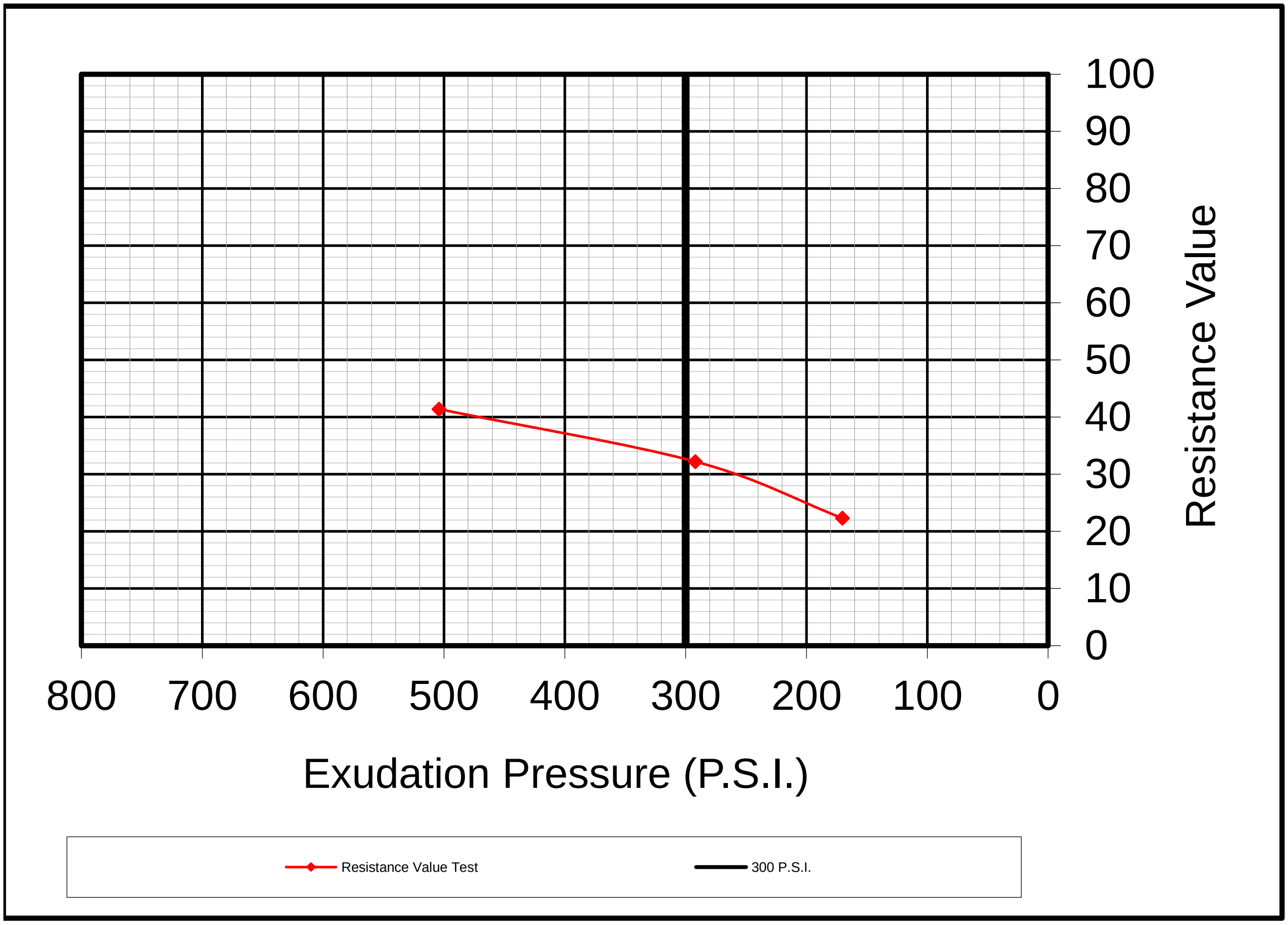
Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High



RESISTANCE (R) VALUE TEST

ASTM D 2844

PEI Laboratory No.:	L192670	**H & K Project Name:**	Idaho Maryland Mine
PEI Client:	NV5 (Holdrege & Kull)	**H & K Project No.:**	5279.02
PEI Project Name:	2019 Laboratory Testing	**H & K Date Sampled:**	10-11-2019
PEI Project No.:	190129-01	**H & K Office:**	Nevada City
Report Date:	October 15, 2019	**H & K Engineer:**	John Atkinson
Sample Description:	Reddish Silty Clay	**H & K PO No.:**	15-19-485
		H & K Sample ID:	0-3'



Specimen No.	7	8	9
Moisture Content (%)	21.7	23.1	20.9
Dry Density (PCF)	104.7	102.9	106.4
Resistance Value (R)	32	22	41
Exudation Pressure (PSI)	292	170	504
Expansion Pressure	13	4	26
As Received Moisture Content (%)	21.7		

RESISTANCE VALUE AT 300 P.S.I. 33

AASHTO R18

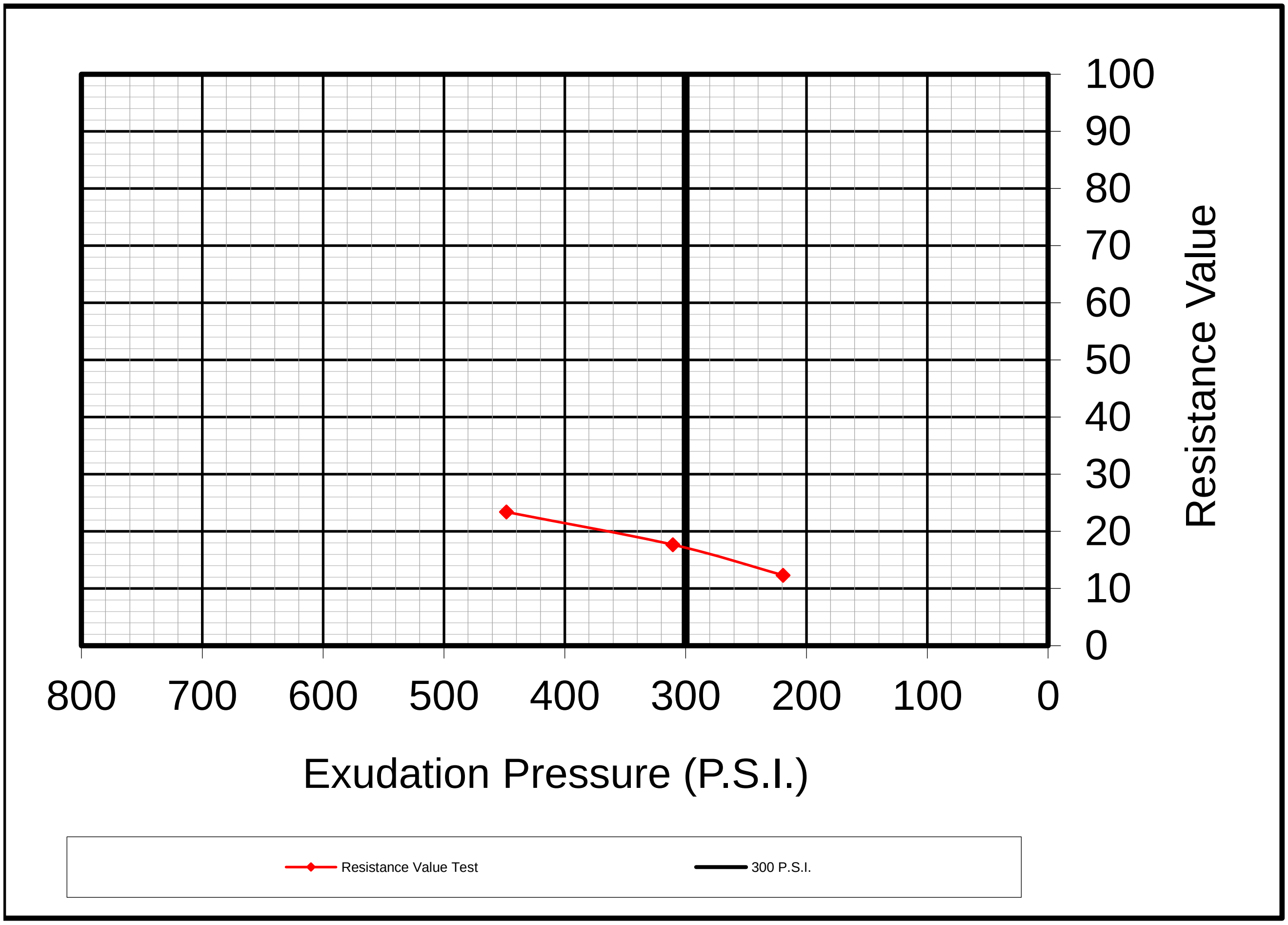
Reviewed By:

Brandon Rodebaugh
Materials Engineer

RESISTANCE (R) VALUE TEST

ASTM D 2844

PEI Laboratory No.:	L192670	**H & K Project Name:**	Idaho Maryland Mine
PEI Client:	NV5 (Holdrege & Kull)	**H & K Project No.:**	5279.02
PEI Project Name:	2019 Laboratory Testing	**H & K Date Sampled:**	10-11-2019
PEI Project No.:	190129-01	**H & K Office:**	Nevada City
Report Date:	October 16, 2019	**H & K Engineer:**	John Atkinson
Sample Description:	Brown Sandy Clay	**H & K PO No.:**	15-19-485
		H & K Sample ID:	BK-2, 0-5'



Specimen No.	10	11	12
Moisture Content (%)	15.5	14.7	15.0
Dry Density (PCF)	117.7	120.0	119.6
Resistance Value (R)	12	23	18
Exudation Pressure (PSI)	219	448	311
Expansion Pressure	0	52	30
As Received Moisture Content (%)	9.4		

RESISTANCE VALUE AT 300 P.S.I. 17

AASHTO R18

Reviewed By:

Brandon Rodebaugh
Materials Engineer

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/18/2019
Date Submitted 10/15/2019

To: Michelle Holub
Holdrege & Kull
792 Searls Ave.
Nevada City, CA 95959

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 5279.02 Site ID : BK-2@0-3FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 80775-168787.

EVALUATION FOR SOIL CORROSION

Soil pH	4.78	
Minimum Resistivity	5.36 ohm-cm (x1000)	
Chloride	7.5 ppm	00.00075 %
Sulfate	26.2 ppm	00.00262 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/18/2019
Date Submitted 10/15/2019

To: Michelle Holub
Holdrege & Kull
792 Searls Ave.
Nevada City, CA 95959

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 5279.02 Site ID : T19-12-B1@2-3F.
Thank you for your business.

* For future reference to this analysis please use SUN # 80775-168788.

EVALUATION FOR SOIL CORROSION

Soil pH	5.63		
Minimum Resistivity	4.02 ohm-cm (x1000)		
Chloride	1.7 ppm	00.00017	%
Sulfate	3.0 ppm	00.00030	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

APPENDIX E

Previous Geological/Geotechnical Reports at the Site

MGW 11/15/88

VECTOR ENGINEERING, INC.

GEOLOGIC AND SUBSOIL INVESTIGATION
AND
PROPOSED GROUNDWATER MONITORING SYSTEM
FOR THE GRASS VALLEY LUMBER MILL RECYCLE POND
GRASS VALLEY, CALIFORNIA

November 3, 1988

Prepared for:

Bohemia, Inc.
P.O. Box 1819
Eugene, OR 97440

Capital Project No. 9619

RECEIVED
SACRAMENTO
CVRWQCB
88 NOV -8 AM 11:47

Prepared By:

VECTOR ENGINEERING, INC.
12438 Loma Rica Drive, Suite C
Grass Valley, CA 95945
(916) 272-2448

DUPLICATE
Driller's Copy

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

WATER WELL DRILLERS REPORT

Do not fill in

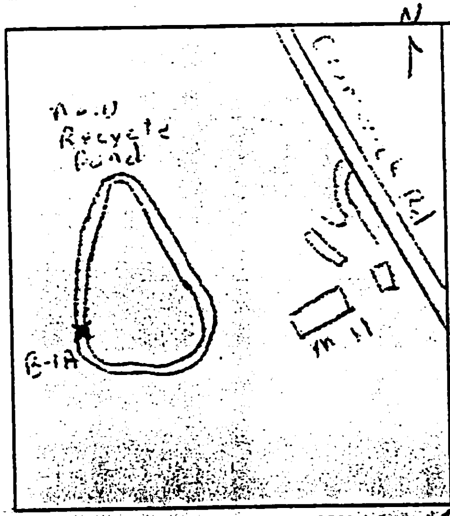
No. 339761

Notice of Intent No. ______
Local Permit No. or Date ______

State Well No. ______
Other Well No. E-1A

(1) OWNER: Name [illegible] Inc
Address P.O. Box 12[illegible]
City Eugene, OR ZIP 9744[illegible]

(2) LOCATION OF WELL (See instructions):
County Nevada Owner's Well Number E-1A
Well address if different from above Gr. Valley Lumber Mill
Township ______ Range ______ Section ______
Distance from cities, roads, railroads, fences, etc. ______



WELL LOCATION SKETCH

(3) TYPE OF WORK:
New Well ☒ Deepening ☐
Reconstruction ☐
Reconditioning ☐
Horizontal Well ☐
Destruction ☐ (Describe destruction materials and procedures in Item 12)

(4) PROPOSED USE:
Domestic ☐
Irrigation ☐
Industrial ☐
Test Well ☐
Municipal ☐
Other Monitoring ☒
(Describe)

(5) EQUIPMENT:
Rotary ☐ Reverse ☐
Cable ☐ Air ☐
Other ☒ Bucket ☐
auger

(6) GRAVEL PACK:
Yes ☒ No ☐ Size #3
Diameter of bore 8"
Packed from 9' to 33 ft.

(7) CASING INSTALLED:
Steel ☐ Plastic ☒ Concrete ☐

(8) PERFORATIONS:
Type of perforation or size of screen

From ft.	To ft.	Dia. in.	Gage or Wall	From ft.	To ft.	Slot size
0	13	2	sch 40	13	33	.02

(9) WELL SEAL:
Was surface sanitary seal provided? Yes ☒ No ☐ If yes, to depth 9 ft.
Were strata sealed against pollution? Yes ☒ No ☐ Interval 2 ft.
Method of sealing cement/bentonite grout

(10) WATER LEVELS:
Depth of first water, if known 25 ft.
Standing level after well completion ______ ft.

(11) WELL TESTS:
Was well test made? Yes ☐ No ☒ If yes, by whom? ______
Type of test Pump ☐ Bailer ☐ Air lift ☐
Depth to water at start of test ______ ft. At end of test ______ ft.
Discharge ______ gal/min after ______ hours Water temperature ______
Chemical analysis made? Yes ☐ No ☐ If yes, by whom? ______
Was electric log made Yes ☐ No ☒ If yes, attach copy to this report

(12) WELL LOG: Total depth 33 ft. Completed depth 33 ft.

from ft.	to ft.	Formation (Describe by color, character, size or material)
0	7	yellow-brown, stiff, sandy clay with some shrub organic material slightly moist Fill material
7	18.7	brown sandy clay with gravel to cobble sized angular Tailing material, moist, medium dense
18.7	20	grey-brown clayey sand, Medium dense, moist.
20	25	Dark brown clay with (minor fine to medium sand) and thin sand lenses, stiff, moist.
25	33	Red-brown sandy clay. Very moist to wet. Increasing gray-green angular gravel to cobble-sized weathered bedrock.

Work started 1-24 1990 Completed 1-24 1990

WELL DRILLER'S STATEMENT:

This well was drilled under my jurisdiction and this report is true to the best of my knowledge and belief.

Signed ______ (Well Driller)
NAME Hogate Drilling (Person, firm, or corporation) (Typed or printed)
Address 8403 Big Oak Drive
City Citrus Heights ZIP 95610
License No. 401530 Date of this report 1-30-90

DWR 188 (REV. 12-86) IF ADDITIONAL SPACE IS NEEDED, USE NEXT CONSECUTIVELY NUMBERED FORM 86 96355

VECTOR ENGINEERING, INC.
12438 Loma Rica Drive, Suite C
Grass Valley, CA 95945 (916) 272-2448

November 3, 1988
Vector Job No. 88052.1
Capital Project #9619
Bohemia.rep

Mr. Hal McCall
Bohemia, Inc.
P.O. Box 1819
Eugene, Oregon 97440

RE: Geologic and Subsoil Investigation and Proposed Ground Water Monitoring System, Grass Valley Lumber Mill Recycle Pond, Grass Valley, California

Dear Hal:

INTRODUCTION

This report presents the results of our investigation of the geologic, hydrogeologic and subsoil conditions and proposed groundwater monitoring system for the Grass Valley Lumber Mill Recycle Pond. We understand the pond, presently under construction, is to be utilized for containment of water applied to stacked timber located southeast of the pond.

The scope of our work was described in our proposal dated 28 September, 1988, and was authorized verbally by Hal McCall on 29 September under Purchase Order # 45-45412.

FIELD AND LABORATORY EVALUATION

Our investigation of the site took place on 4 and 5 October, 1988. Exploratory borings B-1, B-2 and B-3 were drilled to depths of 25, 40 and 60 feet, respectively using a truck-mounted Mobile B-40 auger drilling rig. Monitor wells were installed in each boring to the detail shown on the logs. In addition to the borings, the site surface conditions were observed by our geologist. Locations of the drill holes and attitudes of noted geologic features are presented on the enclosed Site Plan. Available literature including previous reports performed for Bohemia by other consultants for other plant projects, the NID report addressing the slope failure along Brunswick Road and the Geologic Map of the Grass Valley Quadrangle, 1939 edition, were also reviewed during the course of our evaluation.

The drill holes were logged, and samples of each soil or rock type encountered were obtained. Samples were obtained using a 2-inch diameter split spoon sampler by Standard Penetration Test (SPT) methods. The method used to classify the soils is described on Plate 1. Lithologic symbols for rock types are presented on Plate 2. Logs of the drill holes are presented on Plates 3 through 5.

Selected soil samples were tested in the laboratory to determine the particle size, Atterberg Limits, and moisture content. The results are presented on Figures GS-1 through GS-5 and on the drill hole logs.

SITE CONDITIONS

Surface Conditions

At the time of our investigation the Recycle Pond embankments had been fully constructed and efforts were underway to enlarge the storage area within the pond. Several rock outcrops were observed in the center of the pond excavation area as well as along the east slope of the embankment and along the exposed cutbank to the east of the pond where excavations were being performed to provide clay materials for the embankment fill. The western half of the pond bottom had been covered with clay prior to our investigation.

Groundwater movement was observed at several locations along the northeast slope of the pond at or near the contact between the overlying soils and bedrock. Groundwater movement appeared to be in a west to southwesterly direction flowing towards the South Fork of Wolf Creek drainage. The elevations and locations of these observed groundwater levels and of borings B-1, B-2, B-3 and MW-3 were determined by Vector personnel by transit level survey and pacing and are referenced to B-1 at elevation 2727.0 as shown on the site plan. The site plan was prepared for Bohemia by EMCON. The location and elevation of the borings should be considered accurate only to the degree implied by the method used. A cross-section showing the observed groundwater contacts along the surface, the installed monitor wells and the pond embankment is shown on Plate 6.

Surface water runoff from plant operations was also observed flowing from a single point into the pond along the northeast embankment. This water was being pumped back into the present smaller recycle pond to the east. The existing culvert which diverts the South Fork of Wolf Creek crosses underneath the southeast portion of the pond and was observed to be operational and flowing at the time of our investigation.

Subsurface Conditions and Geology

The Grass Valley area is located between strands of the Foothills Fault Zone, a major north northwest trending structural feature of the Sierra Foothills. Paleozoic and Mesozoic rocks in this region and those present at the project site exhibit varying degrees of low grade metamorphism and weathering.

The pond is underlain by sandy clays and silts, coarse mine tailings, lumber mill effluent (sawdust) and bedrock. Subsoils encountered in Boring B-1 along the northwest embankment consisted of 7 feet of recently placed yellowish brown sandy clay fill overlying 9 feet of coarse mine tailings, 7 feet of slightly moist, very dense grayish-brown residual clayey sand directly overlying hard unaltered grayish-green amphibolite schist to the depth investigated, 25 feet. No free water was encountered in this boring.

East of B-1 along the north and northeast pond embankment, fill materials were present to varying depths to a level just above bedrock. The fill consisted of moist reddish-brown sandy silts and clays overlying a thin layer of organic soil which may have been the original ground surface prior to any mining or lumber mill activities in the area. Bedrock elevation dips at an angle of two (2) to four (4) degrees across the site to a point approximately 100 feet north of the south embankment. At this point the bedrock elevation abruptly drops approximately ten (10) feet or more. The bedrock observed within the pond excavations and directly beneath the organic layer mentioned above consisted of grayish-green weathered to slightly altered and fractured serpentinite. Much of the serpentinite had been weathered to clay and much of the crystalline structure was lacking.

The subsoils encountered in boring B-2, located along the south embankment, consisted of 9 feet of recently placed yellowish-brown sandy clay embankment fill, overlying 8 feet of grayish-green moist sandy silt with organic matter and wet sawdust possibly altered during earlier mining and/or lumber milling activities, overlying 21 feet of soft, wet reddish-brown residual sandy silt, overlying moist, very hard mottled residual sandy silt derived from the weathering of serpentinite. Groundwater level was measured at 30.1 feet → switched w/ B-3 (elevation 2752) on 17 October.
?
2727 - 30.1 = 2697

From the pond area to the east along the exposed cutbank towards boring B-3 southeast of the timber storage area, the soil and/or rock observed consisted of yellowish to reddish-brown residual sandy silts and clays derived from the weathering of metasediments of the Paleozoic Calaveras Formation (W. Lindgren, 1939). Evidence of faulting was observed within these sediments at the location shown on the Site Plan. Strike measurements of the exposed fault varied from N40W to N25W at a 45 degree dip. The fault surface was curved along the fault plane and exhibited curved striations along the face. Further to the east, approximately 100 feet west of boring B-3, serpentinite, amphibolite schist and metasediments of the Calaveras Formation were observed adjacent to one another. The soils to the north of the schist were yellowish-brown in color at the surface and the soils to the south of the serpentinite were reddish-brown in color indicating the possibility of groundwater oxidation zones within the soils at this location, suggesting fault control. An attitude of the dip at the rock contact was difficult to estimate as the outcrop exposure was minimal and the rock highly fractured.

The subsoils encountered in boring B-3 consisted of 34 feet of yellowish-brown, stiff to very hard, moist sandy silt-clay grading to a silty sand with increased moisture content from 34 to 48 feet and to a grayish-green, very hard moist sandy clay from 47 to 59 feet, the depth investigated. Groundwater level was measured at 19.3 feet (elevation 2703.7) on 17 October.

NO => B-2 switched ?

GROUNDWATER MONITORING SYSTEM

Monitor wells were installed in each of the three borings upon completion of drilling. Slope-Indicator brand flush-jointed two inch diameter schedule 40 PVC MonitorPipe was installed as casing. Perforated sections were of slot size .020 inch. Number 3 sieve Monterey gravel pack was used for the filter packs. A bentonite seal was provided directly above the filter pack using 3/8 - inch diameter pellets. A four (4) sack cement grout was placed over the bentonite seal to the surface. Eight-inch diameter by twelve-inch high monitor well covers were installed within the cement grout surface seal. The grout was tapered away from the monitor well covers at the surface to prevent ponding above the well. Details of the monitor wells are indicated on the boring logs. Additionally, Bohemia has existing monitor wells at the Grass Valley location. EMCON monitor well MW-3 is shown on the site plan. The groundwater level in MW-3 was measured at 15.6 feet (elevation 2742.3) on 17 October.

Selection of the monitor wells installed was based upon the knowledge of the near surface geology and inferred subsurface contacts as determined throughout our field investigation. The proposed groundwater monitoring system for the Recycle Pond would consist of monitor wells B-1 through B-3, installed by Vector, and monitor well MW-3 installed by EMCON. Monitor well B-1, downgradient, on the west side of the pond, although dry at the present time, would act to intercept potential seepage in that area through the pond liner. Present indications are that groundwater is diverted along the top of bedrock contact beneath the pond to the main drainage of the South Fork of Wolf Creek to the south of the pond. After the pond is filled, however, any seepage that may occur through the clay liner could potentially flow through the pervious coarse mine waste located beneath the west side of the pond. Monitor well B-2, downgradient, on the south side of the pond would act to intercept any seepage from the pond at that point. In addition, monitor well B-2 would act to intercept other

no!

potential contaminants that may enter the near surface groundwater without first being contained by the Recycle Pond. Monitor well B-3, upgradient, located southeast of the timber stacking and spraying area would act to intercept groundwater not affected by the logging operation, and would provide background levels for groundwater quality in the area. The EMCON monitor well, MW-3, would act to intercept any subsurface groundwater before it reaches the Recycle Pond area. This well would act to intercept potential subsurface flow after application of water to the logs stacked upgradient from this location. An approximate groundwater profile is shown on Plate 6. This interpretation uses groundwater levels obtained within the borings as well as observations of groundwater seepage noted along the bedrock contact. It must be noted that seasonal variations in the groundwater levels may occur across the site.

Y?

LIMITATIONS

The findings presented in this report are based upon our field observations from several borings and site reconnaissance, laboratory testing and review of appropriate published literature. Variations across the site are anticipated and may differ from our reported findings. The scope of this work did not include the identification of toxic or hazardous materials.

This report was prepared for the sole use of the client, Bohemia, Inc., for specific application to this project, in accordance with generally accepted geological and engineering practices. Vector Engineering, Inc., makes no other warranties, either expressed or implied, as to the professional findings provided under the terms of this agreement, and as described in this report.

Regards,
VECTOR ENGINEERING, INC.

Rick Kiel

Rick Kiel
Geological Engineer/Project Manager

Reviewed by Ralph Phillips, C.E.G.
Certified Engineering Geologist - No. 758

RALPH D. PHILLIPS
No. 758
CERTIFIED

encl. Site Plan

/rk

MAJOR DIVISIONS					TYPICAL NAMES
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	▩	WELL GRADED GRAVELS, GRAVEL - SAND MIXTURES
			GP	▩	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	▩	SILTY GRAVELS, POORLY GRADED GRAVEL - SAND - SILT MIXTURES
			GC	▩	CLAYEY GRAVELS, POORLY GRADED GRAVEL - SAND - CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	⁘	WELL GRADED SANDS, GRAVELLY SANDS
			SP	⁘	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	▦	SILTY SANDS, POORLY GRADED SAND - SILT MIXTURES
			SC	▩	CLAYEY SANDS, POORLY GRADED SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	▥	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	▨	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	▥	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	▥	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	▨	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	▨	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	☰	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

SAMPLE DESIGNATIONS

▮ Modified Calif. Sample (2.0 or 2.5 in I.D.)

◩ Other tube sample

☒ Bulk or classification sample

KEY TO SYMBOLS

▽ Observed water level

NFWE No Free Water Encountered

TD Total Depth of excavation (ft)

Tv Torve Vane (field) (tsf)

Pp Pocket Penetrometer (field)

q_u Unconfined compressive strength

FDT Field Density (compaction) Test

NR No Recovery

SPT Standard Penetration Test (blows per foot)

KEY TO TEST DATA

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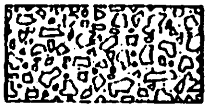
Grass Valley, CA 95945 (916) 272-2448

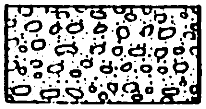
Job No.________ Appr: EKK Date 11-3-88

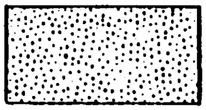
SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

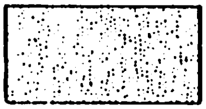
PLATE 1

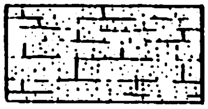
APPENDIX 5. LITHOLOGIC SYMBOLS FOR CROSS SECTIONS AND COLUMNAR SECTIONS

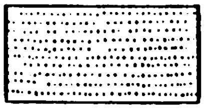

1. Breccia

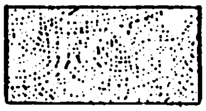

2. Conglomerate

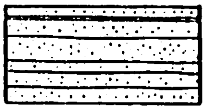

3. Massive sandstone, coarse-grained

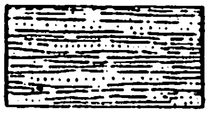

4. Massive sandstone, fine-grained

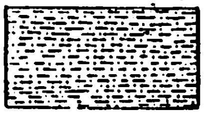

5. Calcareous sandstone

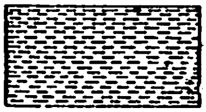

6. Bedded sandstone

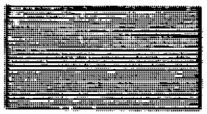

7. Cross-bedded sandstone

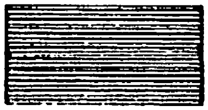

8. Sandstone beds with shale partings

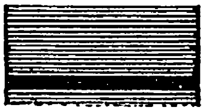

9. Sandstone lenses in shale

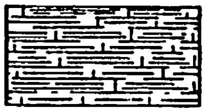

10. Siltstone

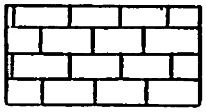

11. Mudstone or massive claystone

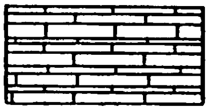

12. Shale

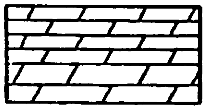

13. Oil shale

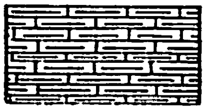

14. Carbonaceous shale with coal bed

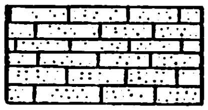

15. Calcareous shale

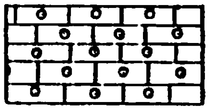

16. Massive limestone

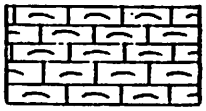

17. Bedded limestone

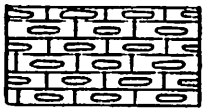

18. Dolomite

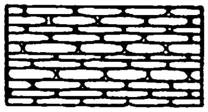

19. Argillaceous limestone

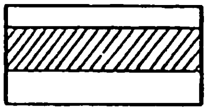

20. Sandy limestone

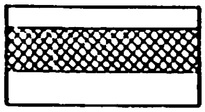

21. Oölitic limestone

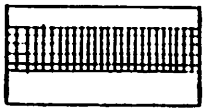

22. Shelly limestone

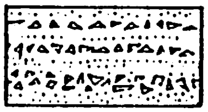

23. Cherty limestone

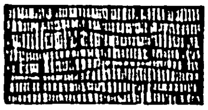

24. Bedded chert

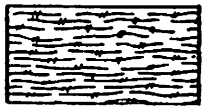

25. Gypsum

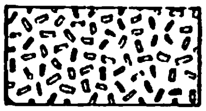

26. Anhydrite

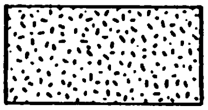

27. Salt


28. Tuff and tuff-breccia

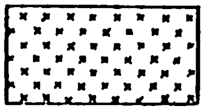

29. Basic lava flows

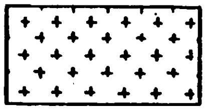

30. Other lava flows

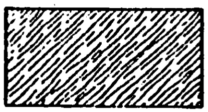

31. Porphyritic igneous rock

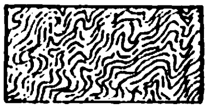

32. Granitic rock

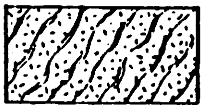

33. Serpentine

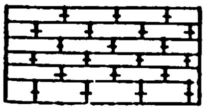

34. Massive igneous rock

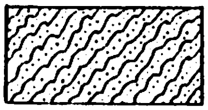

35. Massive igneous rock


36. Schist


37. Folded schist


38. Gneiss


39. Marble


40. Quartzite

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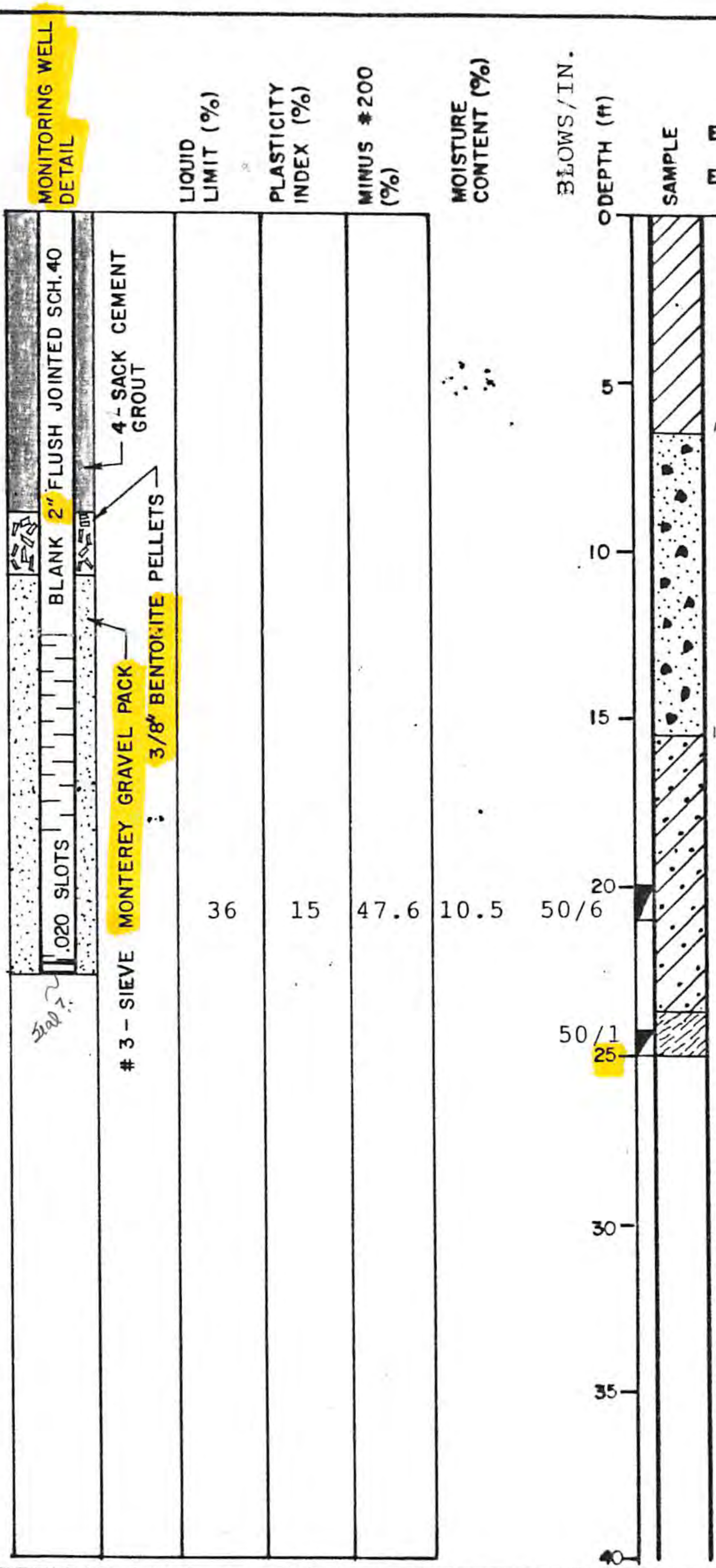
JOB NO. ______ APPR. PEK DATE 11-3-88

PLATE
2

LOG OF B - 1

EQUIPMENT Mobile B-40

ELEVATION 2727.0 **DATE** 10-4-88



DEPTH (ft)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	MINUS #200 (%)	MOISTURE CONTENT (%)	BLOWS/IN.
20	36	15	47.6	10.5	50/6
25					50/1

EMBANKMENT FILL

Yellowish brown sandy CLAY (CL) very stiff, slightly moist

COARSE MINE TAILINGS FILL

Remnant gravel to cobble size angular tailings from Brunswick mine operations

Grayish brown clayey SAND (SC) very dense, slightly moist

Grayish green AMPHIBOLITE SCHIST

TD @ 25 feet

No Free Water Encountered

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JOB NO. 88052.1 **APPR:** RLK **DATE** 11-3-88

LOG OF B - 1

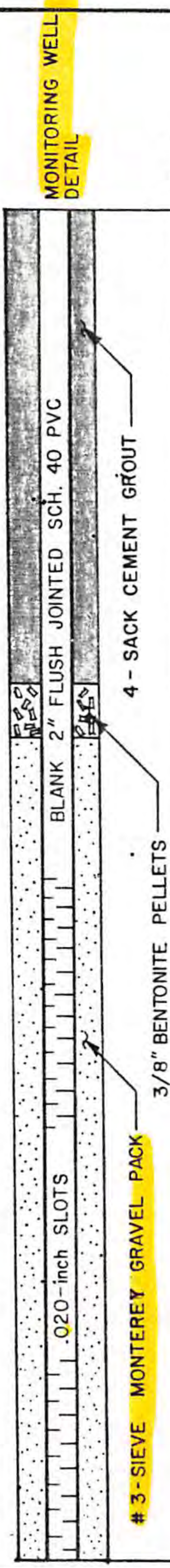
BOHEMIA, INC.
GRASS VALLEY LUMBER MILL
NEVADA COUNTY, CA

PLATE
3

LOG OF B - 2

EQUIPMENT Mobile B-40

ELEVATION 2723.0 **DATE** 10-4-88



LIQUID LIMIT (%)	PLASTICITY INDEX (%)	MINUS #200 (%)	MOISTURE CONTENT (%)	BLOWS/IN.	DEPTH (ft)	SAMPLE	
				2724	0		Yellowish brown sandy CLAY (CL) stiff to very stiff, slightly moist EMBANKMENT FILL
					5		
				14/12	10		9 Grayish green sandy SILT (ML) stiff, moist, previously disturbed or altered, organic matter present
				2712 3/12	15 16		4-inch sawdust layer will contaminants bind to sawdust layer (bottom of pond at 2715)
				19.6' ▽			Reddish brown sandy SILT (ML) soft, wet, residual soils derived from the weathering of bedrock
43	14	74	37.6	4/12	20		
				5/12	25		
				3/12	30		Minor structure becoming more evident
				12/12	35		
40	10	60	29.2	65/12			Mottled sandy SILT (ML) very hard, moist, strong structural features evident
					40		

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JOB NO. 88052.1 **APPR:** RCK **DATE** 11-3 88

LOG OF B - 2

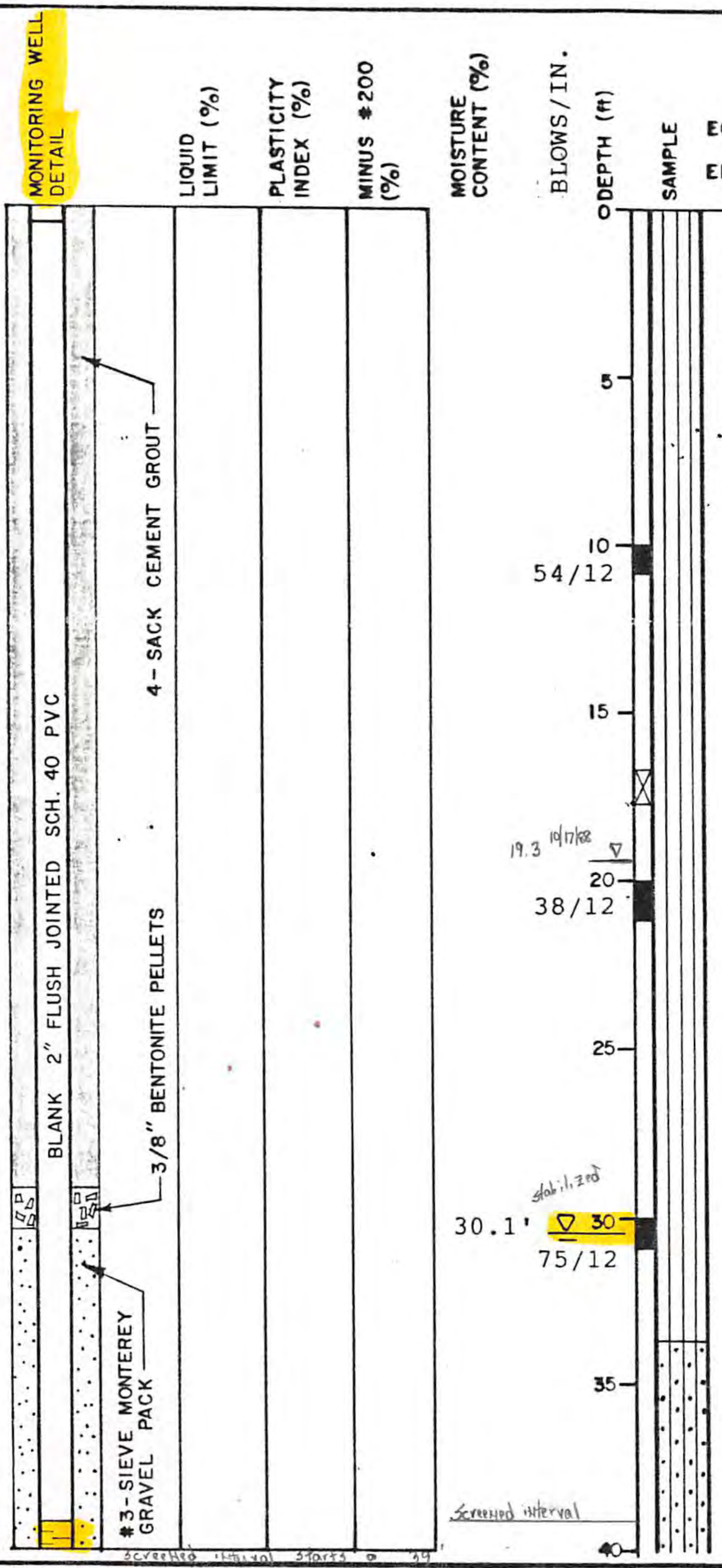
BOHEMIA, INC.
GRASS VALLEY LUMBER MILL
NEVADA COUNTY, CA

PLATE 4

LOG OF B - 3 upgradient well

EQUIPMENT Mobile B-40

ELEVATION 2782 **DATE** 10-5-88



Yellowish brown sandy SILT-CLAY (CL-ML) stiff to very hard, moist, traces of organic matter near surface to 15 feet, soils are residual derived from the weathering of the Calaveras formation metasediments

Minor strucural features evident

Yellowish brown silty SAND (SM) very hard, moist to very moist

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JOB NO. 88052.1 APPR: Rkh DATE 11-3-88

LOG OF B - 3

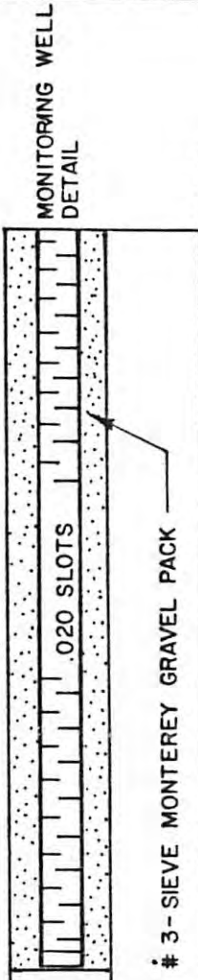
BOHEMIA, INC.
GRASS VALLEY LUMBER MILL
NEVADA COUNTY, CA

PLATE
5

LOG OF B - 3 Page 2

EQUIPMENT Mobile B-40

ELEVATION 2782 **DATE** 10-5-88



LIQUID LIMIT (%)	PLASTICITY INDEX (%)	MINUS #200 (%)	MOISTURE CONTENT (%)	BLOWS/IN.	DEPTH (ft)	
36	8	44.6	20.7	75/12		SAND (SM) cont.
					45	∇ water level during drilling
35	15	60.7	19.5	50/4		Grayish green sandy CLAY (CL) very hard, moist
					50	
					55	
				50/2	60	TD @ 59 feet

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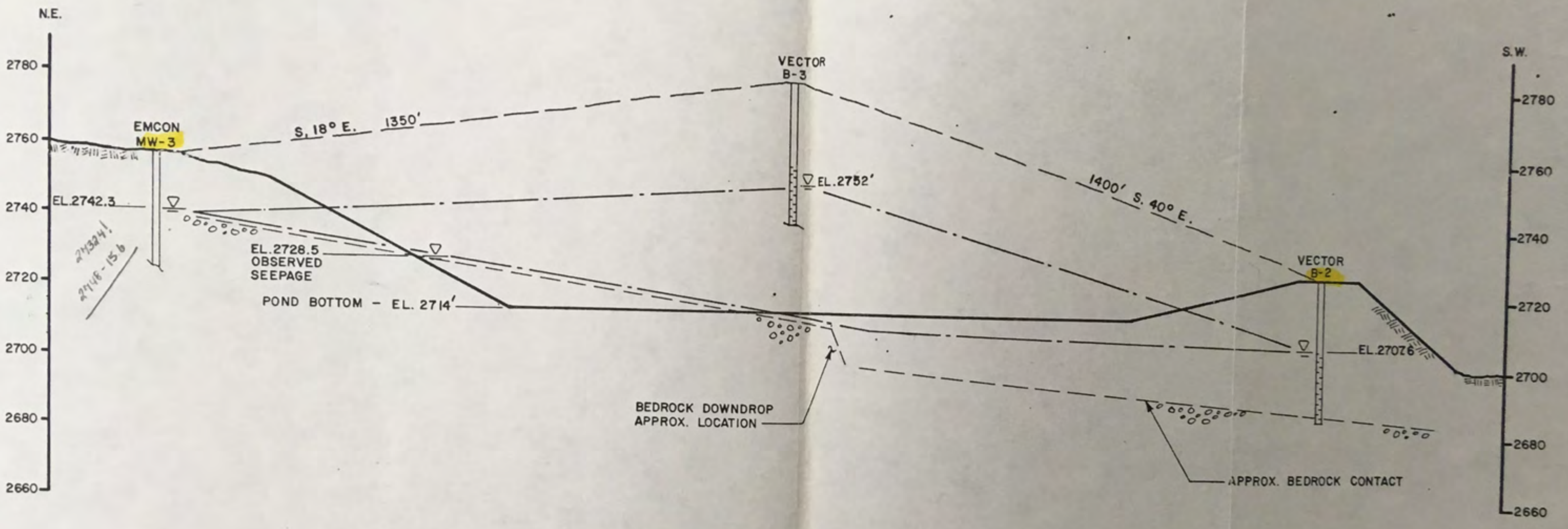
Grass Valley, CA 95945 (916) 272-2448

JOB NO. 88052.1 **APPR** RLh **DATE** 11-3-88

LOG OF B - 3

BOHEMIA, INC.
GRASS VALLEY LUMBER MILL
NEVADA COUNTY, CA

PLATE
5A



SECTION A-A

AND TRIANGULATION (NTS) TO B–3

HORIZONTAL SCALE : 1"=50'
VERTICAL SCALE : 1"=30'

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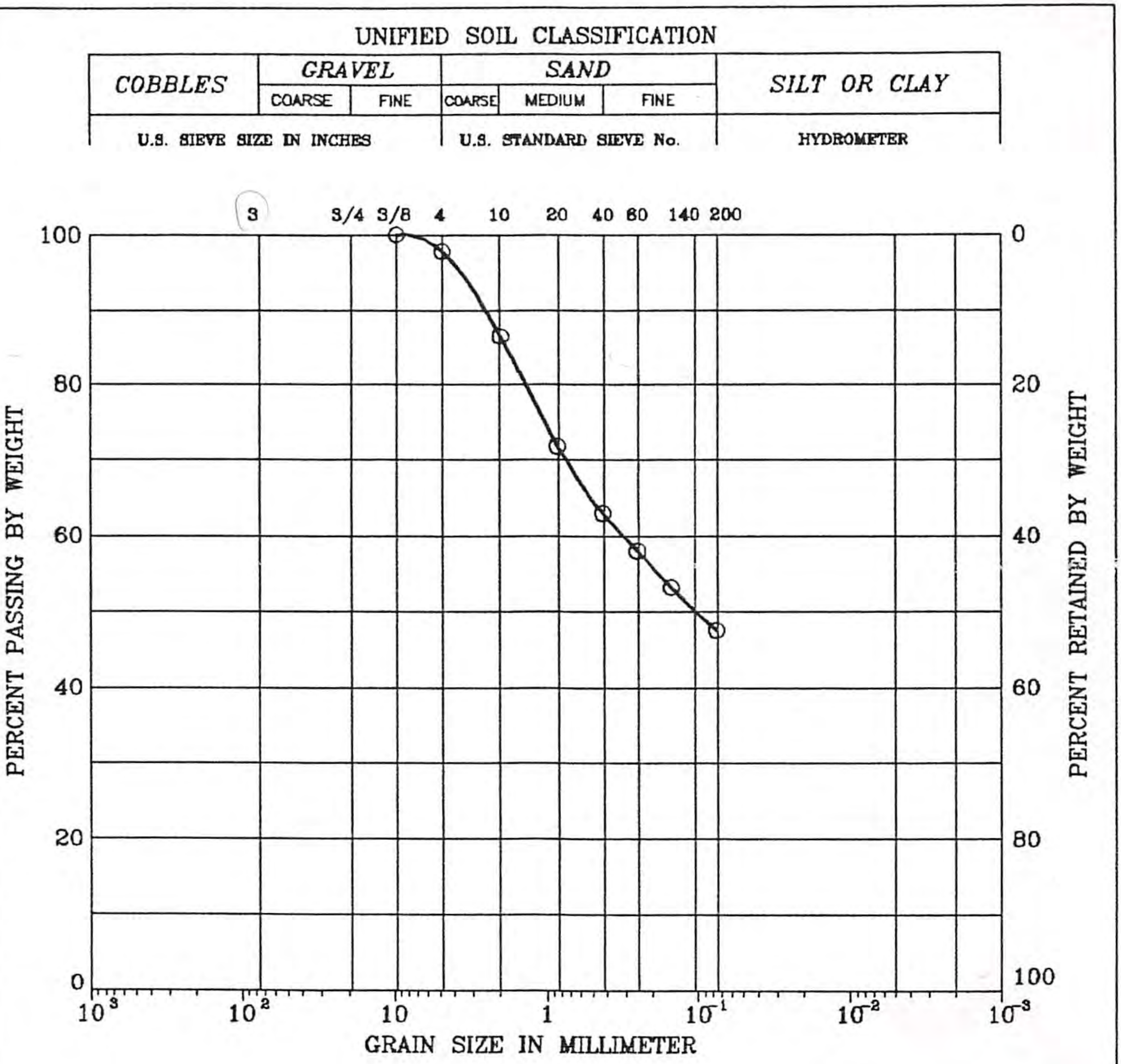
Grass Valley, CA 95945 (916) 272-2448

JOB NO. 88052.1 APPR. RLK DATE 11-3-88

GROUND WATER PROFILE
RECYCLE POND

BOHEMIA MILL GRASS VALLEY, CA

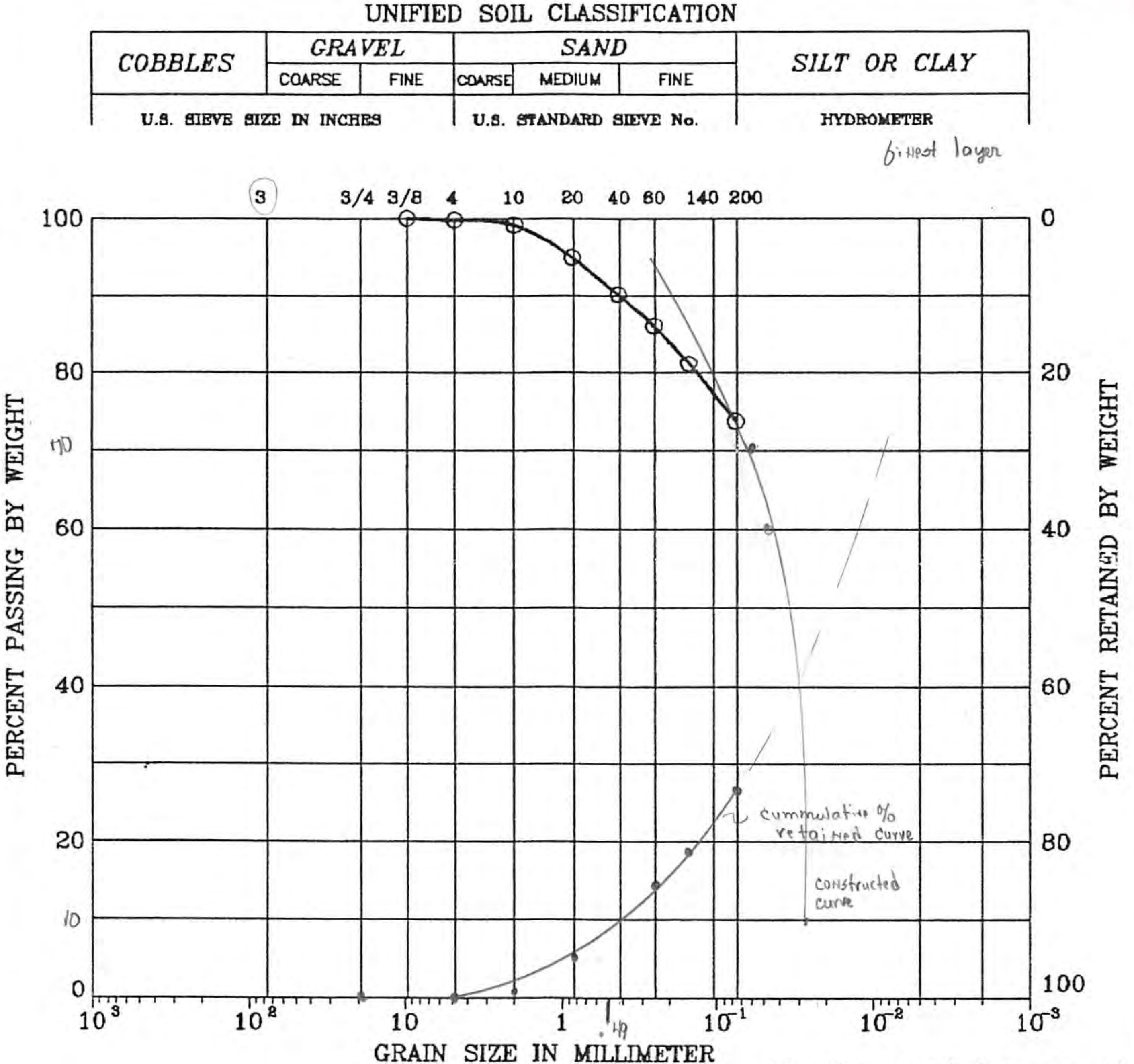
PLATE
6



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	B-1	20	36	15	GRAY-BROWN CLAYEY SAND (SC)

Remark :

Job No. 88052.1	Bohemia Monitor Wells
Vector Engineering	GRAIN SIZE DISTRIBUTION Figure No.GS-1



finest layer

cummulative % retained curve

constructed curve

(too Fine to select gravel pack)
by normal method ⇒ extrapolate
↳ 70 % retained × multiplier

SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	B-2	20	43	14	BROWN-RED SILT (ML)

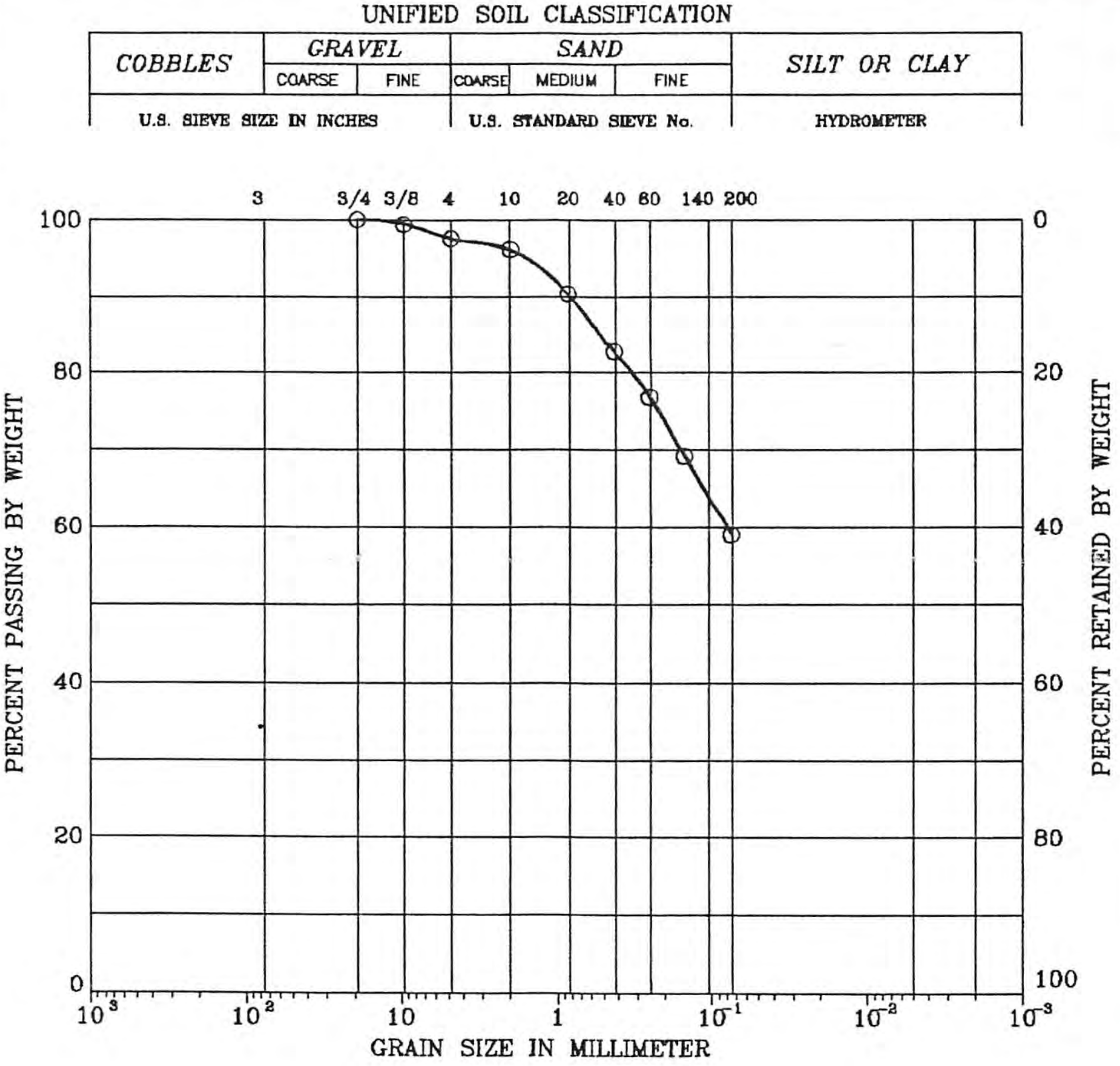
slot size:
1 IN = 25.4cm
.02 IN = .49mm

Remark :

Job No. 88052.1 | Bohemia Monitor Wells

Vector Engineering

GRAIN SIZE DISTRIBUTION Figure No.GS-2



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	B-2	40	40	10	GRAY-BROWN SILT (ML)

Remark :

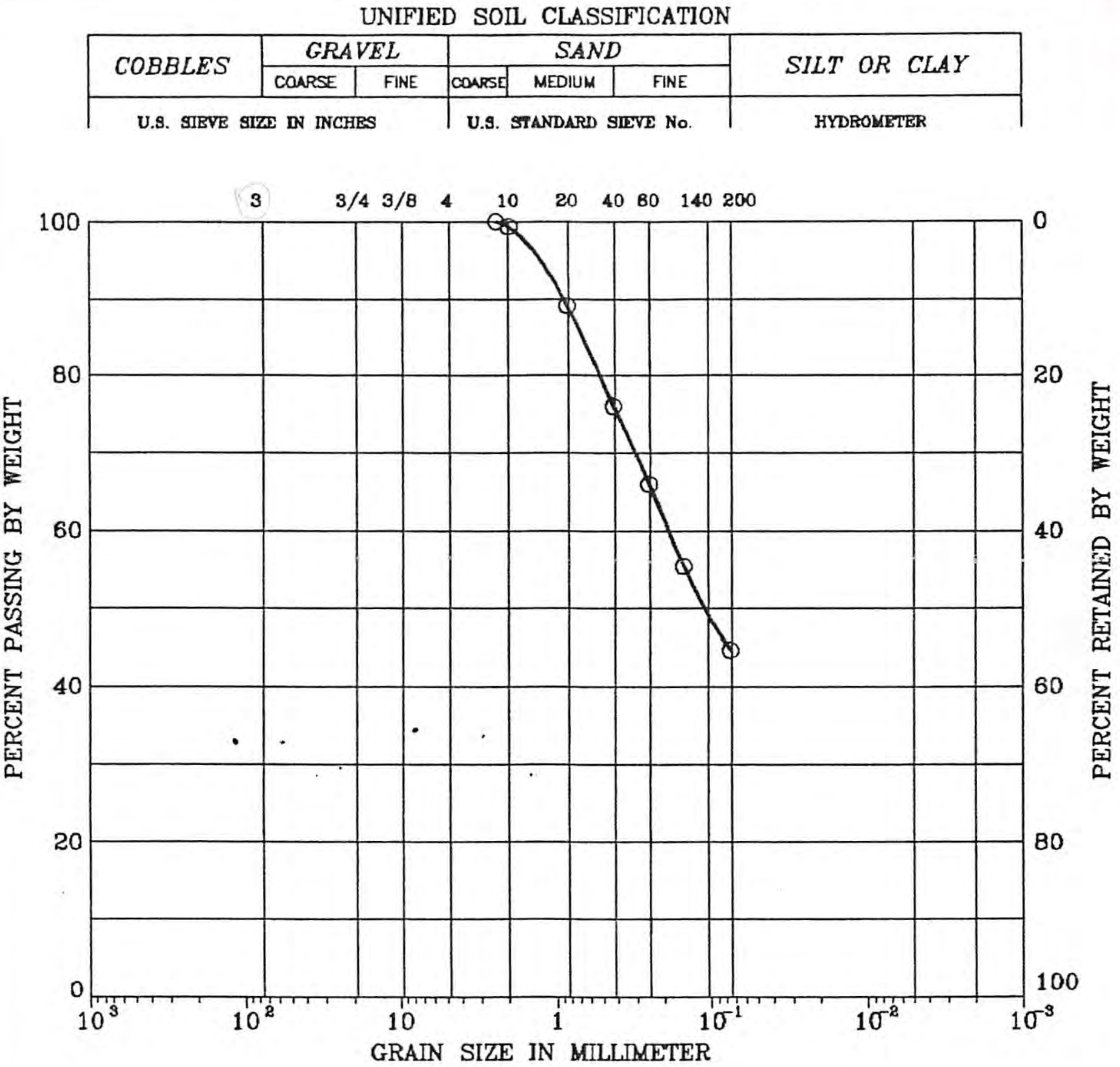
Job No. 88052.1

Bohemia Monitor Wells

Vector Engineering

GRAIN SIZE DISTRIBUTION

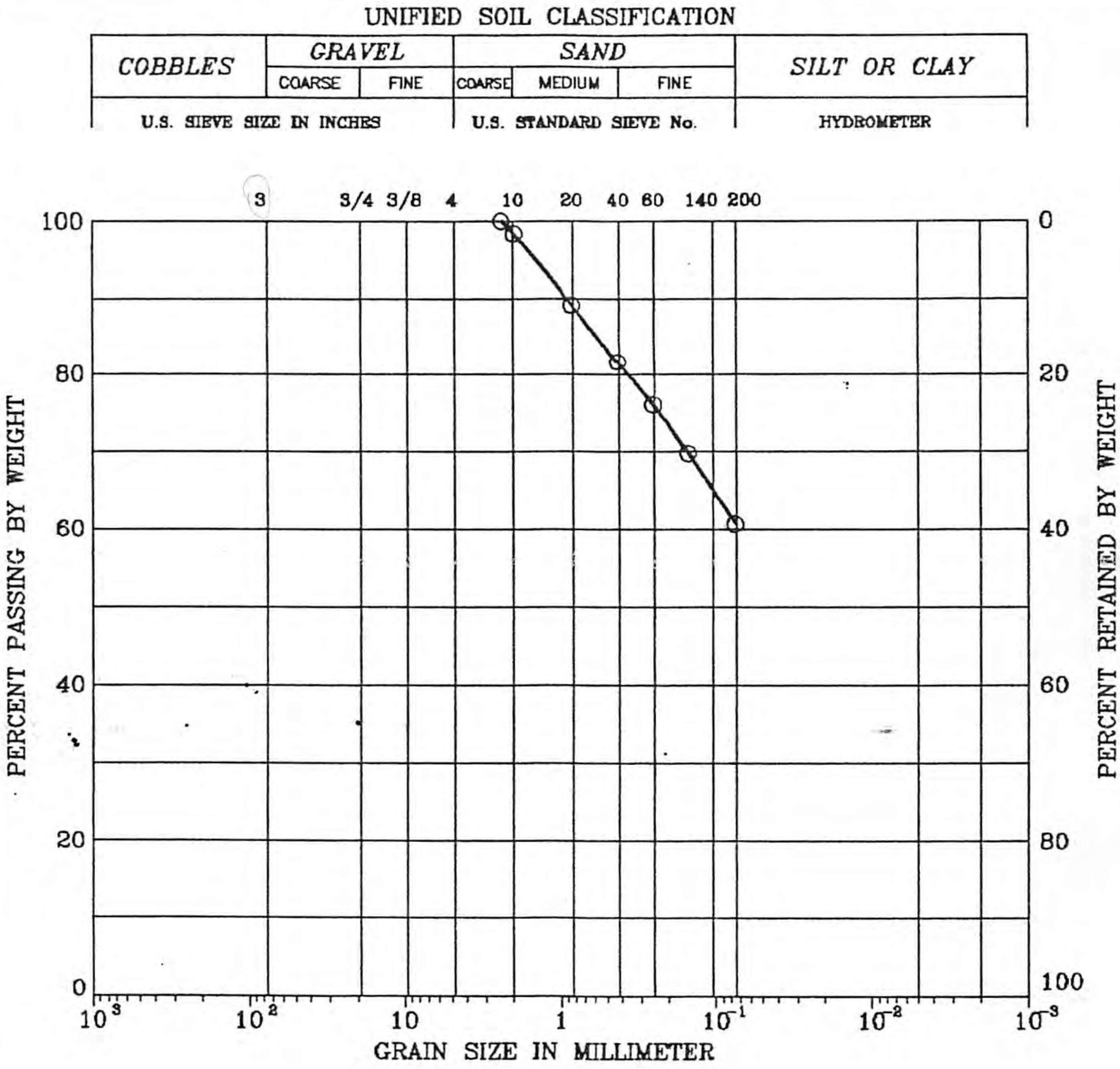
Figure No.GS-3



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	B-3	40	36	8	YELLOW-BROWN SILTY SAND (SM)

Remark :

Job No. 88052.1	Bohemia Monitor Wells
Vector Engineering	**GRAIN SIZE DISTRIBUTION** Figure No. GS-4



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	B-3	50	35	15	GREEN-BROWN CLAY (CL)

Remark :

Job No. 88052.1

Bohemia Monitor Wells

Vector Engineering

GRAIN SIZE DISTRIBUTION

Figure No. GS-5

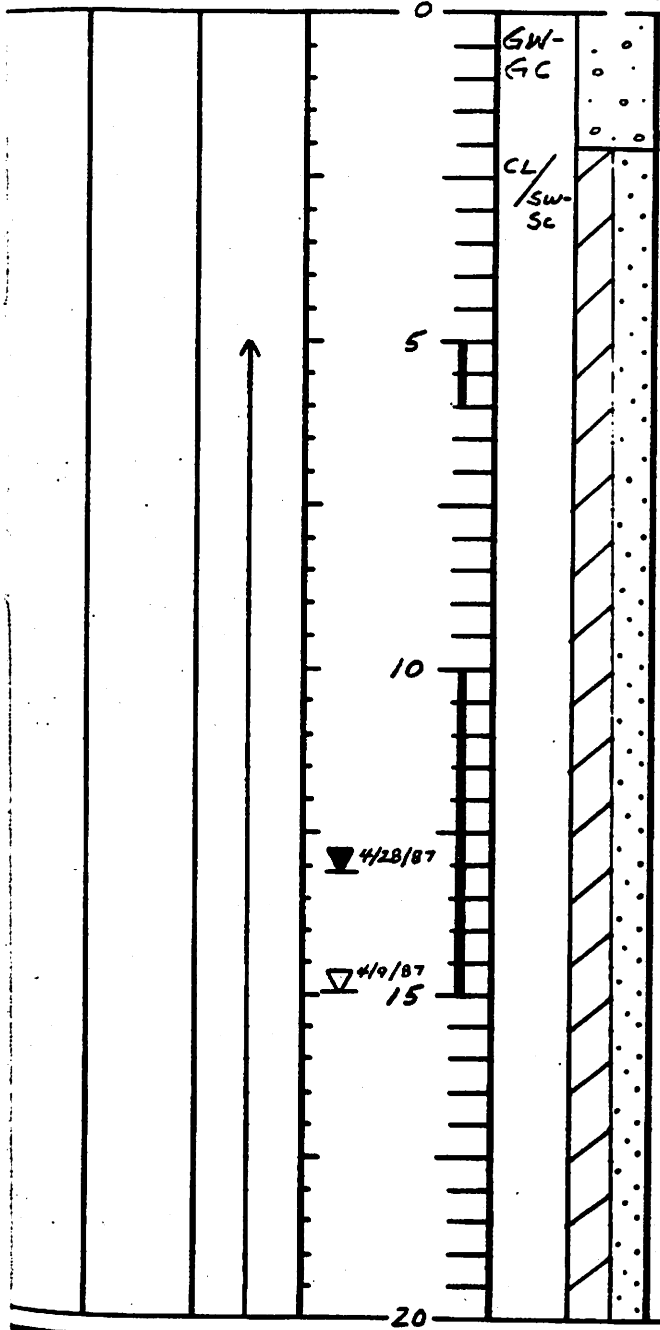
LOG OF EXPLORATORY BORING

PROJECT NUMBER 878-02.01 BORING NO. MW-3

PROJECT NAME Bohemia Inc., Grass Valley Lumber Mill PAGE 1 OF 2

BY SK DATE 4/9/87 SURFACE ELEV. 2748.40

ORVANE (TSF)	POCKET PENETROMETER (TSF)	Cored Interval	GROUND WATER LEVELS	DEPTH IN FT.	SAMPLES	LITHOGRAPHIC COLUMN	DESCRIPTION



SANDY GRAVEL TO CLAYEY GRAVEL-FILL; dark reddish brown (5Yr, 3/3); 5-15% low-plasticity fines, 30% fine to coarse sand, 55-60% fine and coarse gravel, 5% cobble-size clasts; 1-2% concrete fragments; dense; damp.

SANDY CLAY AND GRAVELLY SAND TO CLAYEY SAND-FILL; weak red (10Yr, 5/3) and dark reddish gray (10R, 3/1); sandy clay; 75% low-to moderate-plasticity fines, 10-20% fine to coarse sand, 5-15% fine gravel; mottled; firm; damp. gravelly sand to clayey sand: 5-15% low-plasticity fines, 55% fine to coarse sand, 25-35% fine to coarse gravel, 5% organics; medium dense; damp.

@ 10½': water as film on clasts.

@ 13½': black and white mottling.

@ 14': wood fragments

@ 15-18': increase in rate of penetration (soft zone).

@ 19-21': sandy gravel layer; cobbly; very hard; wet

REMARKS Boring drilled with air-rotary drilling equipment. Core samples obtained with 3-inch I.D. core barrel. Boring converted to a 2-inch monitoring well as shown on Well Details.

↑ – denotes cored interval ▌ denotes

LOG OF EXPLORATORY BORING

PROJECT NUMBER 878-02.01

BORING NO. 1

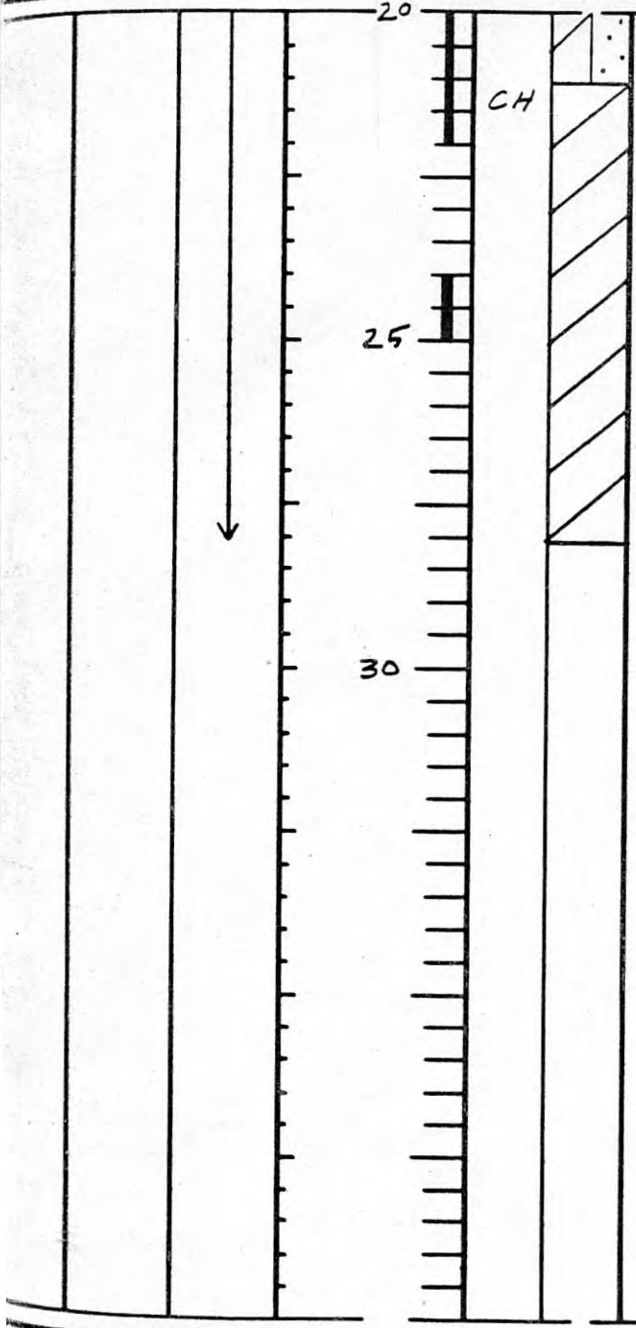
PROJECT NAME Bohemia Inc., Grass Valley Lumber Mill

PAGE 2 OF 2

BY SK DATE 4/9/87

SURFACE ELEV. 2748.40

VANE (TSF)	POCKET PENETROMETER (TSF)	Cored Interval	GROUND WATER LEVELS	DEPTH IN FT.	SAMPLES	LITHOGRAPHIC COLUMN	DESCRIPTION



SILTY CLAY: dark brown (7.5Yr, 3/2); 95% high plasticity fines, very silty <5% fine sand, 1-2% organics, roots; firm; moist

@24': white (7.5Yr, N8); slightly silty; moist.

BOTTOM OF BORING AT 28 FEET.

REMARKS

EMCON ASSOCIATES

PLATE

POND

Surface Area[1]	4.5 Ac.
Average Depth[2]	12.5 Ft.
Storage Capacity	39.6 Ac-Ft.

NOTES:

1. Top of maximum water elevation (2' below top of berm elevation)
2. Average depth from top of berm elevation 2727

SCALE: 1" = 100'

FAULT, LOCATION APPROX., SHOWING DIP & RELATIVE MOVEMENT

MONITORING WELL

NOTE: ORIGINAL PLAN BY EMCON

STATE USE ONLY

DESIGNED
REK 10/26/88
DRAWN
BAS 10/26/88
CHECKED
APP'D
APP'D

NO.	REVISION	DATE	VECTOR

VECTOR ENGINEERING, INC.
GRASS VALLEY, CA
RENO, NV

BOHEMIA, INC
GRASS VALLEY LUMBER MILL

RECYCLE POND

SITE PLAN

T.16 N. R.9 E. SEC. 31
NEVADA COUNTY, CA

SHEET 1 OF 1

Extra copy

VECTOR
ENGINEERING, INC.

12438 Loma Rica Drive, Suite C, Grass Valley, CA 95945
(916) 272-2448 FAX (916) 272-8533

July 20, 1989
Job No. 88052.3
Capital Project No. 9619

Mr. Mike Waggoner
State of California
Regional Water Quality Control Board
Central Valley Region
3443 Routier Road
Sacramento, CA 95827-3098

RE: Report of In-situ Permeability Testing and
Quality Control of the Clay Liner System
Submittal of As-Built Drawings
Bohemia Grass Valley Lumber Mill Recycle Pond
Grass Valley, California

Mike:

This letter presents the results of the in-situ and laboratory permeability tests and quality control provided by Vector Engineering, Inc. for the clay liner at the Bohemia Grass Valley Lumber Mill Recycle Pond in Grass Valley, California. Construction of the clay liner was performed by Robinson Construction, beginning in mid-May and completed in mid-June. The construction was managed by Steve Perry, with Perry Construction, and quality control services provided during construction by Vector Engineering, Inc. (Vector).

Vector monitored placement of the clay liner on a periodic basis and determined the in-place densities of the compacted clay liner using a nuclear density gauge by ASTM Method D2922. Moisture of the clay liner was also verified in the laboratory by ASTM Method D2216. Since no compaction specification existed at the onset of the project, laboratory permeability tests were performed on relatively undisturbed samples of the clay liner obtained from the pond bottom during placement of the first lift at the location of in-place density test numbers 7 and 10. A review of the moisture values determined for the clay indicated that the clay was being obtained from the borrow source and placed on the pond bottom at moisture contents approximately 2 to 8 percent above the optimum moisture content as determined by ASTM Method D1557 for moisture-density determination. These two preliminary permeability tests indicated that compaction of the clay liner to between 85 and 90 percent of the Laboratory Maximum Dry Density at these moisture contents would result in permeabilities on the order of 1 x 10^{-6} cm/sec or less. Since the soil moisture was varying somewhat and to allow for some factor of safety, we elected to establish a project compaction requirement of 90 percent of the ASTM D1557 Laboratory Maximum Dry Density and to control field densities at or above that value. The results of the field compaction tests performed and the construction inspection reports prepared by Vector's engineering technician are attached for your review.

Further laboratory permeability tests were performed on undisturbed samples of the clay liner obtained during the remainder of the construction at locations adjacent to in-place density tests numbers 23, 26, 27 and 29. The laboratory tests performed were fixed wall falling head permeability tests performed in 1.94 inch diameter brass tubes and are similar to the SoilTest, Inc. permeability methods for use with their compaction permeameter equipment for models K-610 and K-612. The SoilTest permeameter operating instructions are attached for your review. One of these tests (Test # 1) experienced a piping failure in the laboratory before the test was complete.

Field permeability tests were performed on the clay liner at four locations across the pond bottom using a Boutwell Permeameter following the methods described by Boutwell and Derick (1986). Four Boutwell permeability tests were performed at various depths in the clay liner. Three of the four Boutwell field permeability tests experienced mechanical piping failures during Stage II of the test. This prohibited calculation of the permeability in the horizontal direction in these cases since the Stage I portion of the Boutwell test measures vertical permeability. A summary of the test results for the Boutwell field permeabilities and the laboratory permeabilities is shown on Table I, below.

The clay liner system was constructed in substantial accordance with the State Water Resources Control Board Title 23, Subchapter 15, Section 2532 regarding construction of the natural liner system for Class II Designated Waste Management Units. If you have any questions regarding these test results or procedures, please call.

An as-built plan of the constructed pond was prepared by Nevada City Engineering, Inc. (NCE) and is also included herein.

Regards,
VECTOR ENGINEERING, INC.

Robert Young
for Rick Kiel
Geological Engineer

Mark E Smith
Mark E. Smith, P.E.
Civil Engineer - No. 35469
Geotechnical Engineer - No. 2082

Encl.- Field Construction Inspection Reports #1-8
Field Density Test Summary
SoilTest Permeameter Operating Instructions
As-built drawings (NCE)
cc: Tom Arlent, Bohemia, Inc. Eugene, Oregon
Jack Hill, Bohemia, Inc. Grass Valley

\rk\boqc89.rep

TABLE I

SUMMARY OF PERMEABILITY TESTS

Test #/Type	Location	Permeability Coeff. (K)
1/Lab	100'N,160'E of SW Cor. (FDT # 7)	5.6×10^{-6} cm/sec Piping failure prior to completion
2/Lab	180'N,170'E of SW Cor. (FDT # 10)	8.7×10^{-8} cm/sec
3/Lab	225'N,200'E of SW Cor. (FDT # 23)	2.9×10^{-8} cm/sec
4/Lab	10'S,150'E of SW Cor. (FDT # 26)	1.4×10^{-7} cm/sec
5/Lab	10'N,50'E of SW Cor. (FDT # 27)	3.1×10^{-8} cm/sec
6/Lab	175'N,10'E of SW Cor. (FDT # 29)	2.2×10^{-7} cm/sec
7/Field	25'N,180'E of SW Cor.	Kv = 2.5×10^{-7} cm/sec
8/Field	180'N,160'E of SW Cor.	Kv = 9.5×10^{-8} cm/sec
9/Field	190'N,25'E of SW Cor.	Kh = 1.4×10^{-7} cm/sec Kv = 1.2×10^{-8} cm/sec
10/Field	35'N,35'E of SW Cor.	Kv = 2.7×10^{-8} cm/sec

NOTES: Kv denotes coefficient of vertical permeability
Kh denotes coefficient of horizontal permeability
FDT denotes in-place field density test
Locations referenced to the SW Corner of the Pond
Lab permeabilities are all Kv.

VECTOR ENGINEERING, INC.

FIELD CONSTRUCTION INSPECTION REPORT

Project: Bohemia
Job No.: 88052.3

Date 5-31 through 6-14-89
Report No. #1-8
Page 1 of 1

Equipment in Use: Various, see below.

Description of Work

Report #1 May 31,1989
Equipment in Use: D-8 with sheepsfoot and 2 scrapers

Met with Steve Perry, Bohemia Construction Consultant, and performed a visual inspection. Robinson Construction was placing fill in the South West Corner of the Recycle Pond bottom. It was agreed that I would return later in the afternoon to perform field density test's in the South West corner. Returned to the project site and met with Bob Dorris who was operating the D-8 pulled sheepsfoot. We observed the fill area and two areas in the South West corner were determined for field density test's. Field density test's #1 and 2 indicated that compaction was low according to the project guidelines. Having notified Bob Dorris of the results it was determined that no more water would be added, an additional dozer would be brought in and I would return on 3-31 at 9:00am for retests. Moisture samples were taken to the lab for accurate moisture calculations.

Report #2 May 31,1989

Equipment in Use: 2 Dozers with sheepsfoot compactors;
and 2 scrapers

Traveled to the project site and met with Steve Perry, discussed the results of 3-30 field density test's and observed fill lift that had been placed over the bottom of the pond. Prior to my arrival clay liner material had been rolled several times with the D-8 and sheepsfoot. Two field density test's would be performed in the South East corner. Tests # 3 and 4 failed to meet project guidelines of 95% relative compaction by ASTM 1557. Mr. Perry and I agreed to meet again in the afternoon with Rich Kiel for further examination and opinion on the liner. It was also agreed that Robinson Construction would continue compaction with the D-8 and sheepsfoot and dozer and sheepsfoot without adding additional water.

Returned to the project site with Rick Kiel and met with Bob Dorris. Four passes with the D-8 with sheepsfoot had been made over entire liner area. Performed field density test's #5 and #6. The relative compaction still did not meet project guideline. Retests #6A and #6B were performed in the same area after being rolled 12 times to determine if the D-8 with sheepsfoot was adequate for the compaction specifications. The results were improved. Field density test's #7 and #8 were performed and an undisturbed sample of the clay liner was obtained from #7 for a laboratory permeability test and from #8 for moisture density test. Steve Perry was notified of the field density test results.

Report #3 June 1,1989

Equipment in Use: 2 dozers with sheepsfoot compactors, 2
scrapers

Traveled to the project site and met with Steve Perry of Bohemia and discussed the project and observed areas to receive fill. The results of the moisture determinations from samples obtained to date indicate that the clay liner is being placed at moistures ranging from 2 to 8% above the optimum moisture content. Subsequently, the compactions achieved in the field have been in the 85-90 percent range. Preliminary permeability tests indicate that if this moisture range is maintained, compaction above 85 percent as compared to ASTM D1557 will achieve permeabilities on the order of 1×10^{-3} cm/sec or less. Since moisture content is variable in these soil types, we will attempt to maintain 90 percent compaction and 2 percent over optimum for the duration of this project. Prior to my arrival the contractor had been compacting liner fill area on the East side of the pond. Field density test's #9-11 were performed and met the modified project specifications. It was agreed the area around #10 would be rerolled along with the West side of the Pond. I would return later for retest #10 and the West side. Returned to project site and met with Steve Perry. Field density test #12-15 were performed and met project specifications. Steve Perry was notified of the results. Robinson Construction would begin placing the second lift for the clay liner.

Report #4 June 2,1989

Equipment in Use: D-8 with 5 X 5 sheepsfoot, D-6 with 3 X 5 sheepsfoot

Travelled to the project site and met with Steve Perry representing Bohemia. I observed sand excavation from SE corner of pond bottom and replacement with clay liner material. I also observed compaction of North Side of liner and advised Steve Perry to continue compaction and to bring moisture content down. Returned to project site and performed field density tests #16-#18. Tests #16 and #17 met project specifications and #18 failed. Steve Perry was notified of the results. A retest will be performed in this area after further compaction is performed.

Report #5

Equipment in Use: D-8 with 5X5 sheepsfoot, D-6 with 3X5 sheepsfoot, s 627 scrapers.

Travelled to the project site and met with Steve Perry. Observed the placement of clay liner final grade in the North East half of the pond bottom. Field density tests #19 through #22 were performed. Field density tests #20 was unsatisfactory. Steve Perry was notified of the results and stated that further compaction would be applied. I would return Monday, June 12 for further testing.

Report #6 June 12,1989

Equipment in Use: D-8 with sheepsfoot, Drum Vibratory Roller

Travelled to the project site and observed the final grade compaction taking place on the liner with the vibratory drum roller in the SE corner and side. I returned later and observed the compaction of the entire liner. I met with Steve Perry and it was agreed I would return in the morning for final field density tests and in place permeability sampling.

Report #7 June 13,1989

Equipment in Use: Drum Roller

Travelled to the project site and met with Steve Perry and observed final grade preparation of the pond liner. Field density tests #23 through #29 were performed. Test #23 was below project specifications. Tests #24 though #29 met project specifications. Steve Perry was notified of the results. In place samples were taken at field density tests locations 23, 26, and 29 for permeability tests to be performed in the laboratory by falling head test methods.

Report #8 June 14, 1989

At the request of Steve Perry, Rick Kiel and I travelled to the project site to discuss final grade preparation of the pond liner. Observed the drain in left in the South side which drains from under the mill. It was agreed to tie the drain in under the liner. Enabling the area to be filled and compacted in accordance with project specifications. It was also agreed to meet at a later date to discuss results of the laboratory perms and to discuss the installation of the field Boutwell permeability apparatus' in order to conform with the California RWQCB regulations.

Technician Ron Royston

Reviewed by ____________________

VECTOR ENGINEERING, INC.
12438 Loma Rica Dr., Ste. C
Grass Valley, CA 95945
(916) 272-2448

Project: Bohemia Recycle Pond
Job No.: 88052.3

FIELD DENSITY TEST SUMMARY

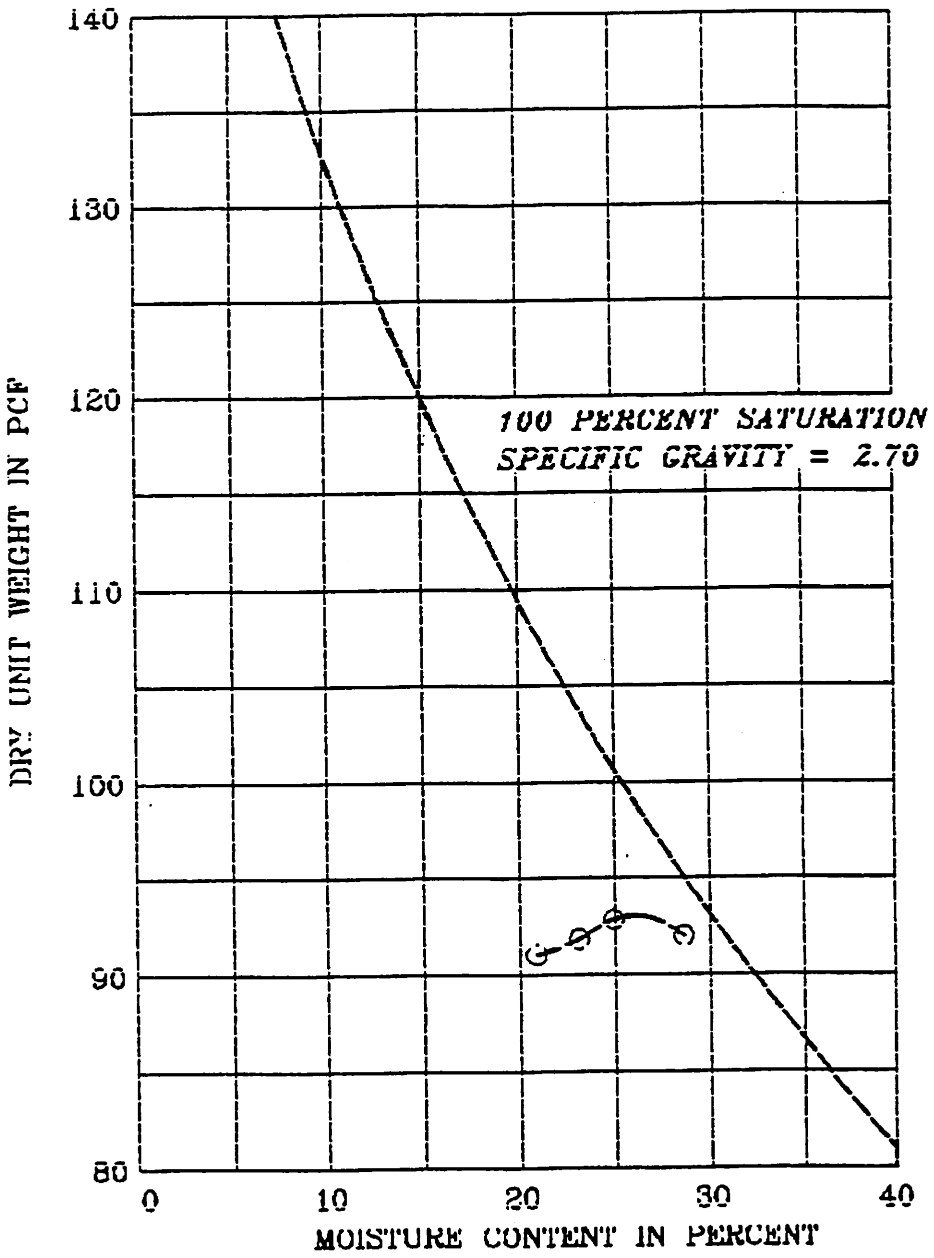
Test No. & Date	Location	Elev. (ft)	Moist. (%)	Dry Density (PCF)	Max. Dry Dens. (PCF)	Rel. Comp. (%)	Remarks
1 5/30/89	120’North, 15’East of SW corner	5"	27.5	76.1	93.9	81	Area reworked and Recompacted
2 5/30/89	200’North, 60’East of SW corner	6"	35.8	79.0	93.9	85	Area reworked and recompacted
3 5/30/89	120’North, 80’East of SW corner	6"	34.1	81.1	93.9	86	Area reworked and recompacted
4 5/30/89	110’North,155’East of SW corner	6"	38.5	81.6	93.9	87	Area reworked and recompacted
5 5/31/89	90’North, 160’East of SW corner	6"	30.8	85.2	93.9	91	
6 5/31/89	170’North,190’East of SW corner	6"	32.4	85.2	93.9	91	
6A 5/31/89	Retest of #6	6"	29.7	90.4	93.9	96	
7 5/31/89	100’North,160’East of SW corner	6"	34.4	83.9	93.9	89	
8 5/31/89	100’South,75’East of SW corner	6"	34.3	84.6	93.9	90	
9 6/1/89	170’East, 60’North of SW corner	12"	32.1	86.5	93.9	92	
10 6/1/89	170’East, 180’North	12"	36.8	82.2	93.9	87	

FIELD DENSITY TEST SUMMARY

Test No. & Date	Location	Elev. (ft)	Moist. (%)	Dry Density (PCF)	Max. Dry Dens. (PCF)	Rel. Comp. (%)	Remarks
11 6/1/89	110'East,170'North of SW corner	12"	35.4	84.3	93.9	90	
12 6/1/89	150'East,160'North of SW corner	12"	32.1	88.5	93.9	94	
13 6/1/89	40'East, 100'North of SW corner	12"	32.3	87.5	93.9	93	
14 6/1/89	100'East,100' South of SW corner	12"	32.6	86.8	93.9	92	
15 6/1/89	150'East,100'East of SW corner	12"	32.7	86.4	93.9	92	
16 6/1/89	200'North,100'East of SW corner	12"	29.4	92.8	93.9	99	
17 6/1/89	50'North,100'East of SW corner	12"	32.1	88.0	93.9	94	
18 6/1/89	20'North,90'East of SW corner	12"	31.7	84.4	93.9	90	
19 6/1/89	200'East,20'North of SW corner	12"	29.9	90.6	93.9	97	
20 6/1/89	210'East,120'North of SW corner	12"	42.7	75.7	93.9	81	
21 6/1/89	150'East,210'North of SW corner	12"	35.2	82.0	93.9	87	
22 6/1/89	75'East, 120'North of SW corner	FG	34.3	86.7	93.9	92	
23 6/1/89	200'East,225'North of SW corner	FG	38.7	81.3	93.9	88.7	
24 6/1/89	190'East,225'North of SW corner	FG	32.4	84.9	93.9	90	
25 6/1/89	200'East,150'North of SW corner	FG	33.8	85.4	93.9	91	
26 6/1/89	150'East, 10'South of SW corner	FG	36.4	84.8	93.9	90	

FIELD DENSITY TEST SUMMARY

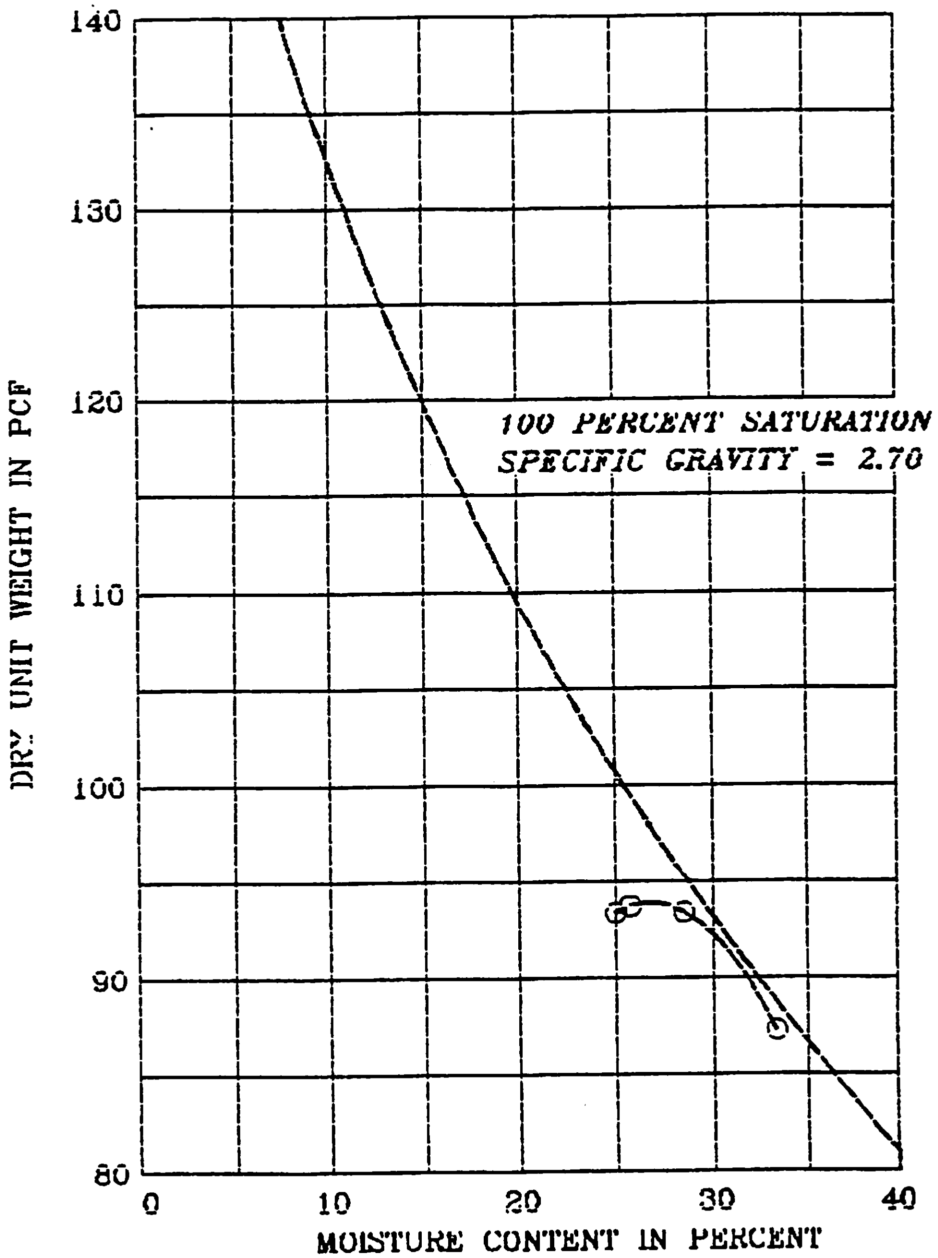
Test No. & Date	Location	Elev. (ft)	Moist. (%)	Dry Density (PCF)	Max. Dry Dens. (PCF)	Rel. Comp. (%)	Remarks
27 6/1/89	50'East, 10'North of SW corner	FG	30.7	84.8	93.9	90	
28 6/1/89	75'East, 100'North of SW corner	FG	30.5	85.6	93.9	91	
29 6/1/89	10'East, 175'North of SW corner	FG	30.9	84.3	93.9	90	



SYMBOL	SAMPLE LOCATION	DEPTH (ft)	DESCRIPTION	TEST METHOD	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
○		Surface	Yellow Brn Silty CLAY (CL)	ASTM D1557A	26.1	93.0

Remark : Pond Liner 200' N of SW Pond Interior Toe

Project No.88052.3	Bohemia Recycle Pond
Vector Engineering	COMPACTION TEST Figure No. MD-1



SYMBOL	SAMPLE LOCATION	DEPTH (ft)	DESCRIPTION	TEST METHOD	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
○		Surface	Yellow Brn Silty CLAY (CL)	ASTM D1557A	26.6	93.9

Remark : Pond Liner 100' N. 100' E of SW Pond Interior Toe

Project No.88052.3

Bohemia Recycle Pond

Vector Engineering

COMPACTION TEST

Figure No. MD-2

SOILTEST, INC.
2205 LEE STREET
EVANSTON, ILLINOIS 60202, USA
UBSIDIARY OF CENCO INCORPORATED • TELEPHONE 312/869-5500 • CABLE: SOILTEST, EVANST

OPERATING INSTRUCTIONS

MODEL	DESCRIPTION	DATE
K-610 K-612	COMPACTION PERMEAMETER	June 1978

1. General:

Direct methods for determining the coefficient of permeability of soil samples are the constant head permeability test and the following permeability test. The K-610 and K-612 Permeameters are designed for performing either constant head or falling head permeability tests on undisturbed, remolded or compaction soils. The units consist of a standard compaction mold and collar, mounting base with porous stone and brass pipe fitting, and head with air escape valve and fitting.

Constant head or falling head of only a few feet of water will be sufficient for materials of high coefficient of permeability greater than .01 cm/sec. For materials of low coefficient of permeability (below .01 cm/sec.), the equipment should have the capacity to apply large pressure gradients up to 100 psi (7 kg/sq.cm.). Also when the permeability of the material is very low, samples of larger cross-section may be used so that a measurable quantity of water may pass through the material under the pressure gradient applied.

Use of large size permeameters such as K-610 and K-612 is recommended for use with the K-670 High Pressure Permeameter set to test soils having coefficients of permeability nearly $.01 \times 10^{-4}$ cm/sec and lower. The specimen can be compacted in the permeameter itself. The miniature permeameter included in K-670 can be replaced with the K-610 or K-612 for conducting the test under a pressure of up to 100 psi.

2. Preparation of Specimen for Test:

2.1 Cohesionless soils:

For tests on cohesionless soils such as sands and gravels the soils are placed in the mold by spoon or any other desired method. To avoid entrapped air it is advisable to raise the water level in the cylinder as the soil is placed, maintaining about one inch of free water above the surface of the sample. If the sample contains fines, care must be exercised to avoid segregation. After the material has been placed and is at the proper voids ratio, the collar is removed and the test head and gasket are set in position and tightened.

2.2 Remolded Soils:

For tests on remolded, compacted soils such as silts, clays, the cylinder and the collar are mounted on a compaction test base and the sample is compacted as per the Standard Proctor Method or other method applicable to the desired test results. With the specimen properly compacted, the collar should be removed and the upper surface of the soil squared with the end of the cylinder. The cylinder and soil are then placed on the permeameter base, and the head fastened in place. Care must be taken to keep air out of the system at all stages of preparation for the test.

2.3 Undisturbed Soils:

For tests on undisturbed soil samples, the specimen is trimmed to fit the cylinder without air space between soil and the wall. If this is impossible, the sides of the sample should be sealed with paraffin and the sample fitted to the cylinder. The cylinder is then placed on the permeameter base and the head fastened in place.

2.4 Rock Samples:

For tests on rock samples (rock cores) the sides are coated with paraffin in order to fit tightly in the cylinder. With this completed, the cylinder is placed on the permeameter base and the head fastened in place.

Note: Cohesive soils can be compacted into the cylinder by the use of a small tamper. Cohesionless soils can be placed and tamped to a dense state by the application of pressure and vibration.

For compacted specimens, it has been found that the energy imparted to the soil, in foot-lbs. per cubic inch of compacted soil, is a reliable criterion of the compactive effort, independent of the size of the compaction cylinder and weight of hammer. Therefore, reproducible results should be obtained by using a known compactive effort. The energy per blow is simply taken as the product of the weight of the hammer times the height of the fall.

3. Operation:

3.1 Constant Head Test:

This test is generally used for determining permeability of high-permeability materials such as sand, gravel, etc.

The test can be performed using the K-670 or without using it as shown schematically in fig. 1.

If the assembly includes K-670 tank, during the period of collecting water (after reaching steady conditions), the level of water in the tank, indicated by the level in the standpipe, is practically unchanged since the level change in the 4" diameter tank, due to the small amount of water flowing out of the permeameter, is negligible. The level of water at the start and finish of collecting water may be noted if rigorous calculation is desired.

In order to use pressure gradients of more than 2 feet, as generally required, a constant level tank with long tubes has to be attached to the permeameter system. (see fig. 1).

3.1.1 Place a receiver for water at the outlet of valve #20A.

3.1.2 Close valves #20A and #23B and open #25A and #23A. This will allow water to rise slowly in the specimen and saturate it. Wait for sufficient time until the water level is seen slowly rising in the pipette.

3.1.3 When the water level in the pipette is a few inches high, open valve #20A and allow water to flow out. This will bring down the water level in the pipette. Regulate the flow by adjusting valve #20A so that the water level stands steady at a convenient height in the pipette.

3.1.4 Adjust the constant level reservoir and valve #20A for the desired pressure gradient and wait until the water level in the pipette stands steady.

3.1.5 Measure (in centimeters) the heights of water H_1 and H_2 from the level of the outlet as shown in fig. 1.

3.1.6 Using a stop watch and a clean receiver, collect the water flowing out of the permeameter for a known time. Select the time interval so as to collect about 10 cc of water.

3.1.7 Calculations: The coefficient of permeability is given by:

$$K=\frac{QL}{AH}$$

Where K = coefficient of permeability, cm/sec.
Q = rate of discharge cc/sec.
L = length of specimen, cm
H = pressure head, cm of water = $(H_1 - H_2)$
A = area of specimen (cross section in cm^2)

3.2 Falling Head Test with Small Pressure Head:

For soils with low permeability, the falling heat test with small pressure head is most applicable. The drain at the base is connected to a tank which will give the tail water elevation. The falling source is connected to the pipe fitting at the head and the valve opened to remove all air from the system; the valve is closed after air removal. If a small pressure head is sufficient, the elevation of the water in a pipette may be taken as the headwater elevation.

3.2.1 Fig. 2 shows the set up for this type of falling head test.

3.2.2 Calculations:

$$K= 2.3 \frac{aL}{AT} \log_{10} \frac{Ho}{H} \quad \text{or } K= \frac{aL}{AT} \text{Log}_e \frac{Ho}{H} \quad a=\frac{Q}{H-Ho}$$

K= coefficient of permeablity, cm/sec.

A= cross sectional area of permeameter, sq. cm

L= length of specimen, cm

a= cross sectional area of standpipe (pipette), sq. cm

T= time of the test, sec.

Ho= head at start of test, cm

H= head at end of test, cm

Q= discharge, cc

3.3 Falling Head Test with Large Pressure Head:

The falling head test, using relatively high pressure, is most applicable for cohesive type of soils with low permeability.

3.3.1 Set up the permeability set as shown in fig. 1, with the quick-disconnect coupler of the rubber tube attached to the bottom of the mold. Fill the water tank. Close all the valves.

3.3.2 Connect compressed air to the system as shown in fig. 1.

3.3.3 Open valve #25B and adjust the pressure regulator (#5) for the required pressure as read on the gauge #4.

3.3.4 Slowly open valve #25A and allow enough time for the specimen to get saturated with water. Wait until water level slowly rises and reaches nearly the middle of the pipette.

The time reuquired for this depends on the permeability of the material and the pressure applied.

3.3.5 Place a receiver for water at the outlet of valve #20A and open #20A partially until water level in the pipette slowly goes down. Also open valve #23B.

The flow may be very slow. Flow can be increased by increasing the air pressure (adjusting the regulator) to a maximum of 60 psi.

3.3.6 Allow the water flow to continue for about ½ hour to let all the air in the specimen escape.

3.3.7 Close valve #20A for a short time until water level rises a few inches in the pipette. Then open valve #20A and close valve #23A before the water level falls below the valve.

3.3.8 Regulate the flow of water,if necessary, by the valve #20A and adjust the inlet air pressure to get a pressure reading in gauge #38.

3.3.9 After the gauge readings become steady, note the reading of gauge #4. Then close valve #25A tightly, shut off air closing valve #25B, release pressure using the regulator #5 and open valve #23C. Measure the height h_1 (in cms.) of water level in the stand pipe from the centerline level of gauge #38 as shown in fig. 1.

3.3.10 Close valve #23C, open air inlet valve #25B and adjust the regulator #5 to get a reading on gauge #4 equal to the value noted in step 3.3.9.

3.3.11 When the readings on the two gauges become steady (in a few minutes) collect the water in a clean receiver for a known time using a stop watch, and determine the exact volume of the water in cubic centimeters.

3.3.12 Take the reading P_1 of gauge #4 and also reading P_2 of gauge #38 in psi.

3.3.13 Close valve #25A, shut off air supply and release air pressure completely with the regulator #5.

3.3.14 Open valve #23C and measure the height h_2 (in cms) of water level in the stand pipe from the level of gauge #38, as shown in fig. 1.

Find the average value h of h_1 and h_2 (cms).

3.3.15 Calculations: When a very high pressure head is used in a falling head test, it is sufficiently accurate to average the start and finish heads and to calculate the permeability as in the constant head test.

Also, since the water tank has a large cross section (4 inch diameter) and since the water flowing out in the period of real test is only a few cubic centimeters, the change in the level of water in the tank will be practically zero or negligible.

Therefore, permeability, $K = \frac{QL}{AH}$

Where K = coefficient of permeability, cm/sec.
Q = rate of discharge cc/sec.
L = length of specimen, cm
H = pressure head, cm of water
A = area of specimen, sq. cm

The value of H is calculated from $H = (P_1 - P_2)\ 70{\cdot}317 + h + L$:

where P_1 = test reading of gauge #4 in psi

P_2 = test reading of gauge #38 in psi

$h = \frac{h_1 + h_2}{2}$ cms

70·317 = conversion factor to convert psi to cms of water

NOTE: For tests in which it is desired to maintain a high hydrostatic pressure, but a small difference in pressure head (i.e. simulating underground low depth pressures) two pressure tanks, one at each end may be used. One would become head pressure, the other tail pressure.

Additions: (1) Fig. 1

(2) Fig. 2

(3) K-610 drawing

(4) K-612 drawing

50	Air and water tank
25B	Air inlet valve
4	Tank pressure gauge
5	Pressure regulator
39	Quick disconnect
25A	Pressure inlet valve
37	Cylinder mold
20A	Overflow valve
23B	Gauge shut-off valve
23A	Pipette shut-off valve
38	Outlet pressure gauge
23C	Waterlevel pipette valve
28A	Tank water level pipette
28B	Manometer pipette

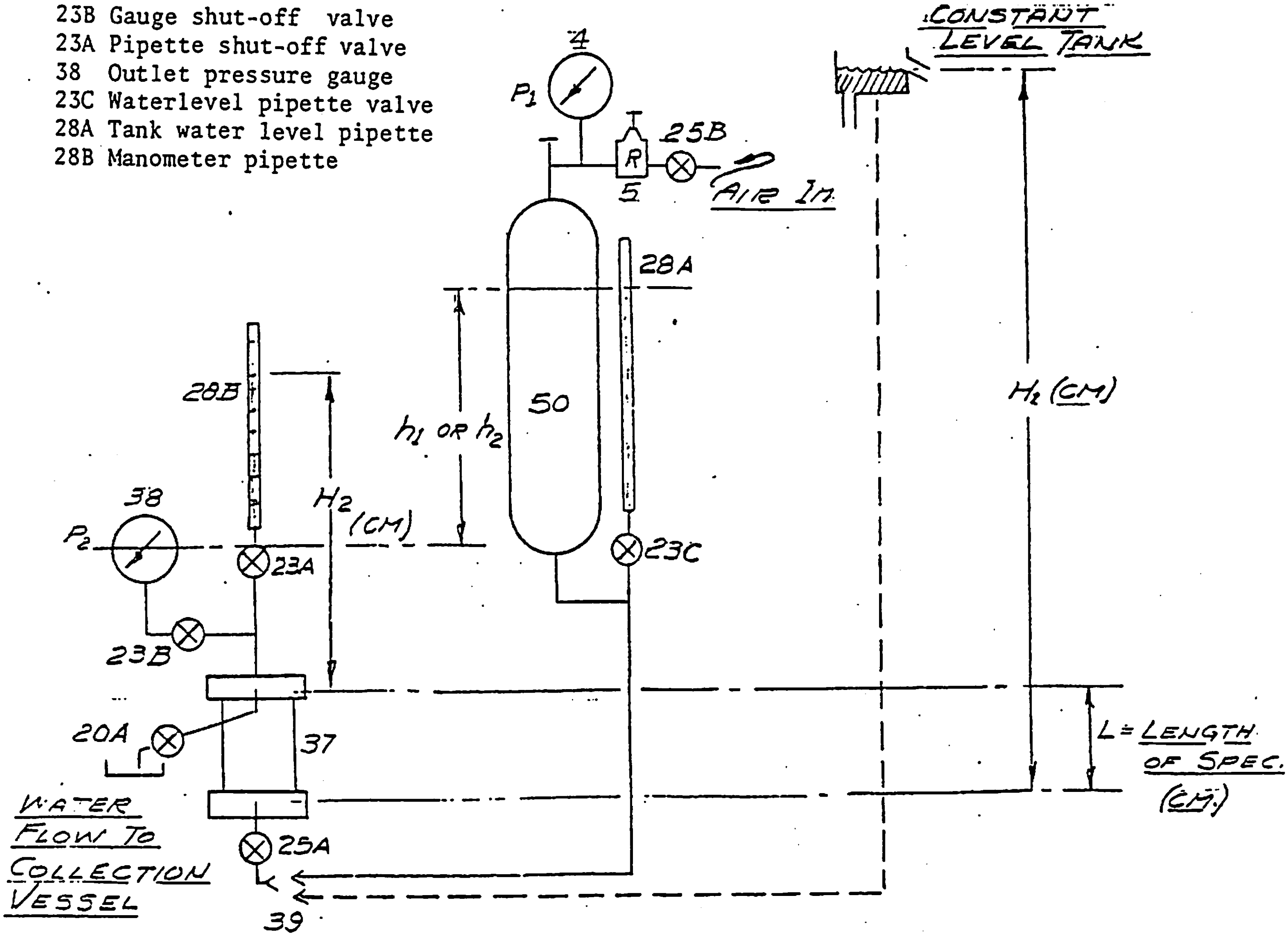
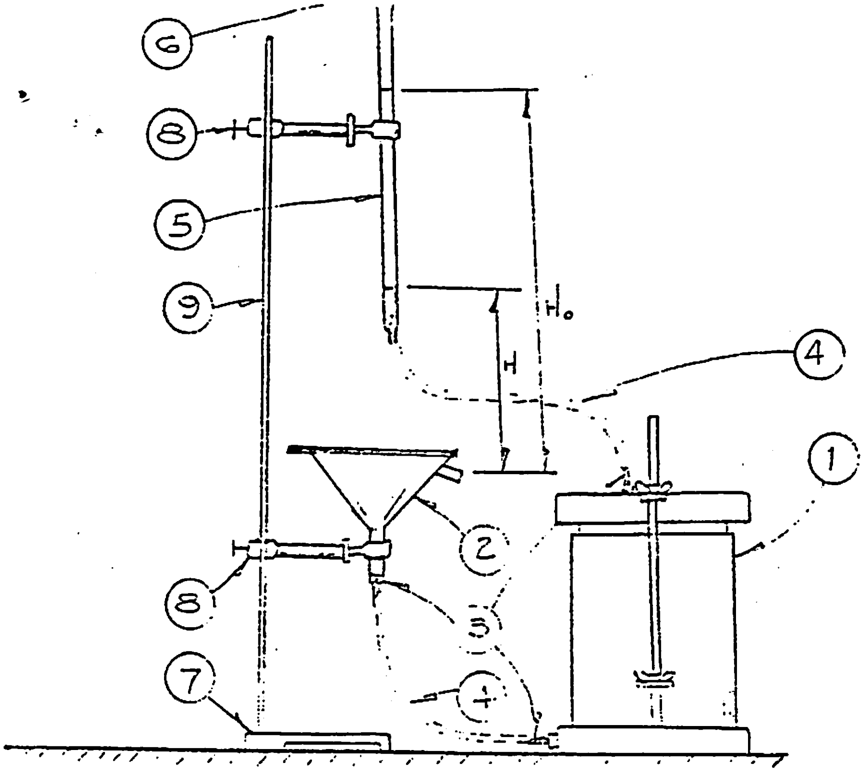


FIG. 1

K-670 Schematic

Legend: Dotted line from the base of the mold represents connection required for constant head test.

Solid line from the base of the mold represents connection required for falling head or constant head test using the tank.



Falling Head Permeability Test with K-610

Parts List

	No.	Name	Rqd.
1.	K-610	Compaction Permeameter	1
2.	K-609-A9	Funnel	1
3.	AP 405-10	Hose Coupling	1
4.	G- 132	Plastic Tubing	5 ft.
5.	A-1-10	Burette (100 ML)	1
6.	G-121	Funnel	1
7.	CT-12-9	Ring stand base	1
8.	K-605-13	Clamp	2
9.	CT-12-9-1	Aluminum rod, 32"	1

Fig. 2

GEOTECHNICAL REPORT
for
IDAHO MARYLAND MINE RETENTION BASIN
Nevada County, California

Prepared for: Emperor Gold Corporation
12503 Brunswick Road
Grass Valley, California 95945

Prepared by: Holdrege & Kull
308 Main Street, Suite 5
Nevada City, California 95959

MAY 24, 1996

HK HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

Project No. 469-01
May 24, 1996

Mr. Tom Knoch
c/o Emperor Gold Corporation
12503 Brunswick Road
Grass Valley, California 95945

Reference: Idaho Maryland Mine Retention Basin
Brunswick and Bennett Roads
Grass Valley, California

Subject: ***Geotechnical Report***

Dear Tom:

As requested, we performed a geotechnical investigation and stability analyses for the existing pond at the Old Bohemia Mill site at Brunswick and Bennett Roads in Grass Valley. We understand that the existing pond is to be used as a retention basin and water treatment facility during dewatering of the Idaho Maryland Mine. Our findings and geotechnical recommendations pertaining to the dam are included.

Our work revealed that the structural stability of the dam is adequate for use during dewatering of the mine. However, we did encounter some unsuitable soil under the dam that precludes its use for long term storage or expansion.

We appreciate the opportunity to work with you on this project and look forward to continuing our working relationship.

Please contact us if you have any questions.

Sincerely,

HOLDREGE & KULL

Tom Holdrege for
Jason Muir
Staff Engineer

Tom Holdrege for
Charles R. Kull
C.E. 46701/C.E.G. 1622

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INTRODUCTION

Purpose

The purpose of our study was to perform the tasks outlined in our February 2, 1996, proposal for the project. We were to perform a subsurface investigation, perform laboratory tests on soil samples obtained during the investigation, perform stability analyses of the dam and provide geotechnical recommendations for the existing earth dam at the Idaho Maryland Mine retention basin.

Scope of Services

To prepare this report, we drilled six borings along the top of the earth dam. The site plan, Figure 1 in the Appendix, shows the approximate boring locations. During drilling, we collected bulk samples and relatively undisturbed soil samples for laboratory testing. We used the test results to perform stability analyses and to provide geotechnical recommendations for the dam.

Project Location and Description

The existing dam is located on the west side of the retention basin at the Idaho Maryland Mine at Brunswick and Bennett Roads in Grass Valley, California. The retention basin is to be used during dewatering of the Idaho Maryland Mine.

The pond was previously used for water retention during operations of the Bohemia lumber mill. The water was reclaimed after irrigating logs during the milling process.

FIELD INVESTIGATION

Exploratory Drilling

A graduate engineer from Holdrege & Kull observed the drilling of six exploratory borings through the dam on April 25, 1996. Borings were advanced with a truck mounted CME-75 drill rig equipped with 6-inch diameter, hollow stem augers. Two of the borings were drilled into underlying native soil to determine the soil/rock conditions comprising the dam foundation. We had originally proposed to drill three to four borings; however, a fifth boring was drilled to compensate for another boring that had met refusal at shallow depth. We drilled a sixth boring at Tom Knoch's request to determine the lateral extent of a sawdust layer encountered in Boring 1. We observed the subsurface soil conditions to determine the type, depth, and uniformity of the underlying soil and rock. We obtained bulk soil samples and relatively undisturbed soil samples during our field exploration for specific laboratory testing purposes. At Emperor Gold's request, the borings were backfilled with grout, and the upper foot of the borings was backfilled with native material.

Site Conditions

At the time of our field investigation, vegetation around the dam consisted of alders, willows, and riparian grasses and shrubs. We did not observe any seepage on the downstream side of the earth dam. We observed riparian plants and trees in the meadow downstream of the earth dam. We observed erosion in a relatively dry stream bed near the downstream side of the southern section of the dam. More significant erosion was evident downstream of the corrugated metal pipe (CMP) culvert under the dam. However, this erosion was farther from the toe of the dam than the erosion noted near the southern section of the dam. As the site map did not provide topographic contours of the downstream side of the dam, we estimated the slope in the field.

Soil/Rock Conditions

The following paragraphs describe general soil conditions encountered during drilling. For more detailed soil descriptions, refer to the boring logs, Figures 2 through 7, in the Appendix.

The upper 12 to 15 feet of the earth dam generally consisted of medium orange-brown to red-brown, slightly moist, medium-dense to dense, sandy silt with minor clay and minor organic material. This layer was interbedded with grey-green, slightly moist, dense silt with minor clay, organic debris and gravel to 2 inches in diameter.

The southern section of the dam contained a layer of well-preserved to slightly decomposed sawdust. We encountered the sawdust at depths of 12.5 feet and 15 feet in borings 1 and 6, respectively. The sawdust layer was approximately 6 feet thick in those two borings. The sawdust was overlain in boring 1 by 2.5 feet of gravel. The sawdust was generally underlain by dark grey to dark red-brown, wet, loose silt with minor sand and minor organic material between depths of approximately 20 and 45 feet. The silt was underlain by several feet of dark brown, very moist, medium dense, silt with clay and increasing rock structure between 47 and 49 feet below ground surface (bgs).

In the northern section of the dam, the upper silt layer was underlain by light brown to medium orange-brown, slightly moist, dense, sandy silt with rock fragments to 3 inches in diameter. We observed minor sawdust at 6.5 feet in boring 4. The soil became increasingly rocky with depth, and the drill rig met refusal at 17.5 and 14.5 feet in borings 4 and 5, respectively.

A rock buttress had been constructed at the toe of the northern section of the dam, as shown on Figure 8 in the Appendix. The buttress was comprised of gravelly soil with cobbles and boulders to 12 inches in diameter.

We encountered water at depths ranging from 10 to 28 feet bgs in the southern section. We did not encounter water in the northern section of the dam.

SOIL ANALYSES

Laboratory Testing

Laboratory testing consisted of 11 moisture-density determinations, two undrained shear strength determinations, one consolidation test, and one determination of maximum dry density and optimum moisture content (compaction curve).

The consolidation test was not originally proposed. However, it was performed at Tom Knoch's request after we informed him of the loose, clayey silt underlying the sawdust layer. Results of the consolidation test were used to determine the percent consolidation of the underlying clayey silt and to determine the potential for additional settlement. The Appendix contains the laboratory results.

Slope Stability Analyses

Using soil strength data from the laboratory tests, we analyzed critical soil slopes with a computer-assisted slope stability program (Stabl 5). Slopes were analyzed under static, pseudostatic and saturated conditions. Results of the stability analyses are summarized below.

We developed two cross sections, A-A' and B-B', based on generalized soil conditions. Figure 1 in the Appendix shows the cross section locations; Figures 8 and 9 present the cross sections. Circular failure analyses result in a calculated "factor of safety" for the slope analyzed. The factor of safety is the ratio of forces resisting failure to the driving forces which cause slopes to fail. A factor of safety greater than one indicates the slope is theoretically stable; a factor of safety less than one indicates a theoretically unstable slope. Depending on the type of analysis and assumptions made, engineers typically require factors of safety to be a minimum of 1.3 to 1.5 for static design purposes and 1.2 to 1.3 for dynamic design.

Our slope stability analyses, which are summarized in Table 1, are based on the following assumptions and variables. These assumptions provided us with the most realistic values for slope stability given the existing slope configuration.

1. Strength data variables - The strength data used in our calculations were varied based the following assumptions:

    a. Generalized soil conditions were used to develop cross-sections at the northern and southern ends of the earth dam. The cross-section locations are shown on Figure 1; Figures 7 and 8 show the cross sections. The northern section of the dam was divided into four distinct layers with a large rock buttress on the outside toe of the dam. The southern section was divided into three distinct sections. We determined soil strengths from laboratory direct shear test results or by correlating blow counts obtained during drilling (Terzaghi and Peck, 1948).

    b. We estimated shear strengths for the sawdust layer.

2. We estimated the most probable failure plane through the earth dam, thus forcing theoretical failure circles through the top of the earth dam, sawdust layer and underlying clayey silt.

3. We used an assumed water table in our analyses to determine how the earth dam would behave under saturated conditions. The water table was estimated from the static water levels measured in the open borings.

4. We input earthquake loading into several analyses to determine how the slope would behave under pseudostatic conditions. Anticipated peak ground accelerations locally in Nevada County are estimated at 0.15g.

Table 1 summarizes the stability analyses that we performed on the existing slope.

Table 1 - Summary of Slope Stability Analyses

Case Number	Code	Saturation	Earthquake Load	Factor of Safety
1	1	Full	None	4.2
2	1	Full	0.15g	3.4
3	2	Full	None	1.6
4	2	Full	0.15g	1.1

Codes:

1	North slope cross section (A-A')
2	South slope cross section (B-B')

Consolidation Analysis

One consolidation test was performed on sample 1-8, which was obtained from Boring 1 at a depth of 31 feet below the top of the earth dam. The material consisted of dark reddish-brown, clayey silt with minor sand. The soil had a compression index (C_c) of 0.22 and a coefficient of consolidation (C_v), of 0.008 in^2/min at a consolidation pressure of 3200 psf. The total estimated settlement 7.5 inches, based on the C_c, a preconstruction effective stress of 1390 psf and a total overburden effective stress (weight of the earth dam) of 2976 psf. The time for total consolidation is estimated at 1.1 years, based on a double drained layer thickness of 12 feet and a C_v of 0.008 in^2/min.

DISCUSSION AND CONCLUSIONS

Our slope stability analyses indicate that the existing earth dam is theoretically stable under the assumed loading conditions. Dynamic earthquake loading of 0.15g yielded a factor of safety of 1.1 through section B-B' which was the lowest factor of safety recorded. This factor of safety indicates marginal stability under the assumed conditions. The northern cross section (A-A') had a higher factor of safety due to flatter slope gradients, more suitable fill material, and the rock buttress.

The consolidation data indicated that the majority of settlement has occurred. No consolidation analysis was performed on the sawdust layer. However, the sawdust was isolated to a small area around Borings 1 and 6.

Our opinion is that the earth dam is theoretically stable in its current condition. We base our opinion on the strength data obtained and consolidation and stability analyses performed. The inside slope of the dam had gradients flatter than 3:1 horizontal to vertical (H:V), resulting in a projected width of the base of the dam of approximately 85 feet. This width provides an extended flow path for seepage, thus decreasing the potential for seepage velocities to affect the stability of the downstream slopes.

RECOMMENDATIONS

We understand that the pond will not be required after the mine has been dewatered. The stability analysis results indicate that the dam is marginally stable in its present condition. Therefore we do not recommend long term use of the retention basin unless the water level is lowered. This can be accomplished by lowering the spillway culvert by 2 to 3 feet, breaching the earth dam, or draining the pond and backfilling with on-site soil.

If the dam is to be used for long term water retention, we strongly recommend reconstruction of the dam.

LIMITATIONS

Review, Observation and Testing

The recommendations and conclusions in this report are preliminary in nature based on the limited subsurface investigation conducted.

Uniformity of Conditions

The recommendations in this report are based on the assumption that the soil conditions do not deviate from those we observed in the exploratory boring.

Services Provided

We issue this report with the understanding that it is the responsibility of the owner, or their agent, to furnish the information contained herein to the project engineers so that it can be incorporated into the project plans. The conclusions and recommendations contained herein are professional opinions derived in accordance with the current standards of professional practice. No warranty, expressed or implied, including any implied warranty of merchantability or fitness for the purpose is made or intended in connection with our work. We did not conduct any studies to determine the presence of hazardous materials.

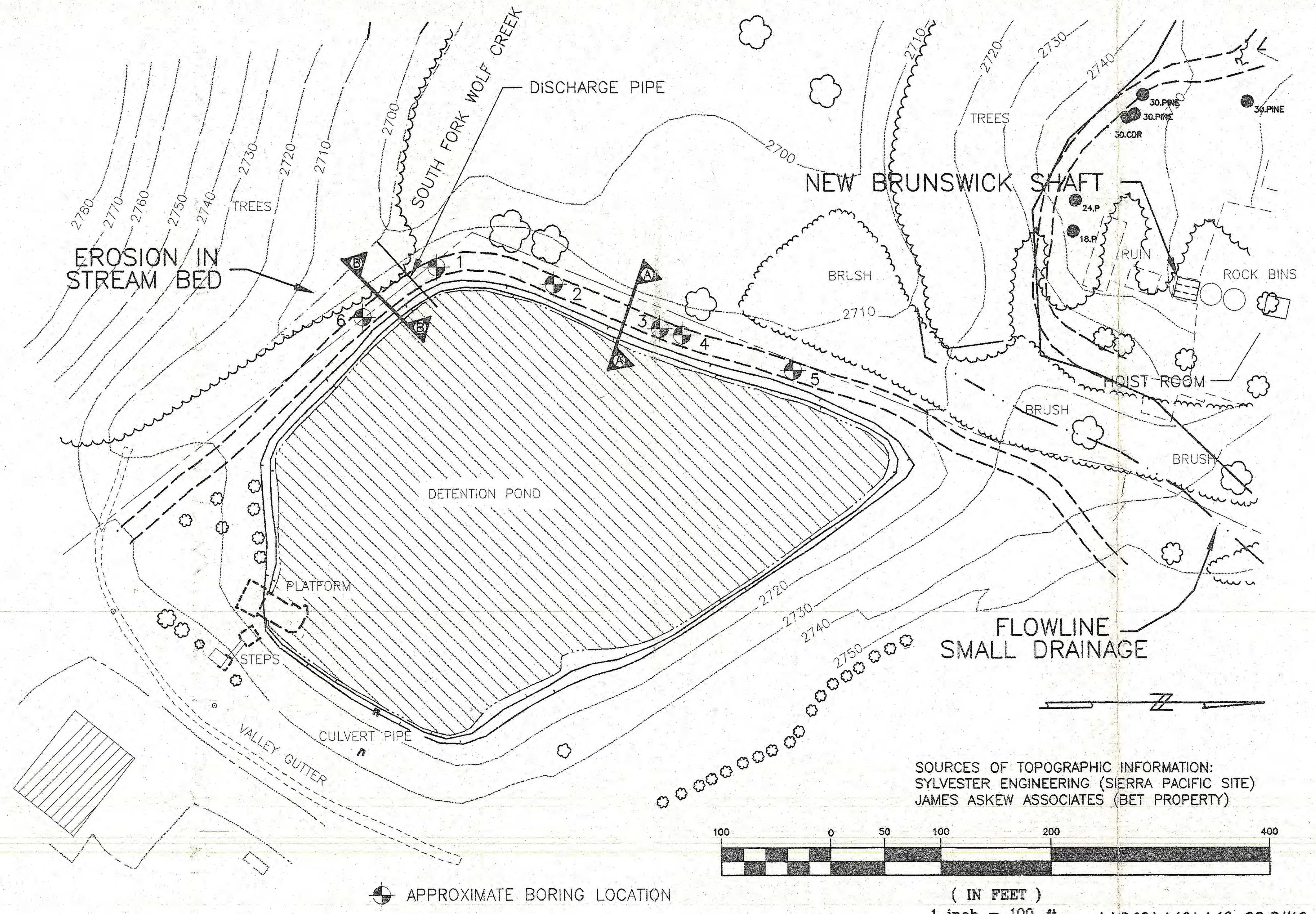
Time Limitations

The findings of this report are valid as of the present date. However, changes in the conditions of the property can occur with the passage of time. The changes may be due to natural processes or to the works of man, on the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or the broadening of knowledge.

APPENDIX

Figure 1	Site Plan
Figures 2 - 7	Boring Logs
Figures 8, 9	Cross Sections
Figure 10	Compaction Curve
Figures 11, 12	Direct Shear Results
Table 2	Moisture Density and Direct Shear Test Results

Results of Consolidation Testing



SOURCES OF TOPOGRAPHIC INFORMATION:
SYLVESTER ENGINEERING (SIERRA PACIFIC SITE)
JAMES ASKEW ASSOCIATES (BET PROPERTY)

⊕ APPROXIMATE BORING LOCATION

J: \R13 \469 \469-SP.DWG

HK **HOLDREGE & KULL**
CONSULTING ENGINEERS • GEOLOGISTS
308 MAIN STREET, STE. 5
NEVADA CITY, CA 95959
(916) 478-1305 FAX 478-1019

Site Plan for Idaho Maryland Mine

RETENTION BASIN DAM

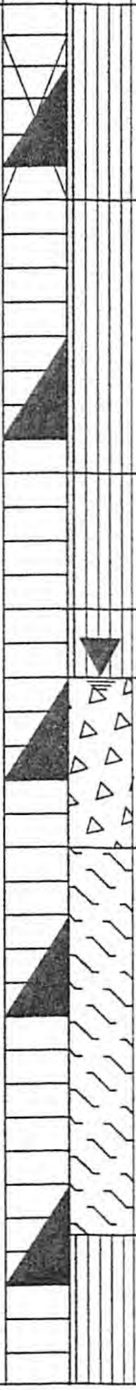
Nevada County, California

DRAWN BY: JWM	CHECKED BY: CRK
PROJECT NO.: 469-01	
DATE: MAY 1996	
FIGURE NO.: 1	

BORING 1

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	FIGURE NO.
469-01	IDAHO MARYLAND MINE	2715 feet	4/25/96	1 OF 2	2

DRILL METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75 – 8" HOLLOW-STEM AUGER	CALIFORNIA SPLIT-SPOON	10 feet	15 feet



SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					ML	Medium red-brown interbedded with orange-brown, slightly moist, medium-dense, sandy SILT with minor clay and organics
CB-1				1		
1-1	2/2/5			2		
				3	ML	Medium to dark brown, slightly moist, medium-dense, sandy SILT with minor clay and organics
				4		
				5		
1-2	2/4/4	56.8	41.7	6		
				7	ML	Red-brown, slightly moist, medium-dense, sandy SILT
				8		
				9	ML	Medium-brown, slightly moist, medium-dense, sandy SILT with sub-rounded rock to 2 inches in diameter
				10	ML	Angular fragments of serpentenized basalt to 3 inches in diameter
1-3	14/19/17			11		
				12		
				13		Well-preserved to slightly decomposed sawdust
1-4	1/2/2			14		
				15		
				16		
				17		
				18	ML	Grading to dark grey-brown, wet, loose SILT with organics Seepage at 19 feet
1-5	1/1/1	78.4	42.2	19		
				20		

BORING 1

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	FIGURE NO.
469-01	IDAHO MARYLAND MINE	2715 feet	4/25/96	2 OF 2	2

DRILL METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75 - 8" HOLLOW-STEM AUGER	CALIFORNIA SPLIT-SPOON	10 feet	15 feet

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
						Dark grey-brown, wet, loose SILT with organics (continued from page 1)
1-6	1/1/1	84.5	38.2	21		
				22	ML	
				23		
				24		
						Grading to dark red-brown, wet, loose, clayey SILT with sand
				25		
1-7	1/1/1	83.2	37.9	26		
				27		
				28		
				29		
				30	ML	
1-8	1			31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		
						BOTTOM OF BORING AT 40 FEET

HOLDREGE & KULL

BORING 2

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	FIGURE NO.
469-01	IDAHO MARYLAND MINE	2715 feet	4/25/96	1 OF 1	3

DRILL METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75 - 8" HOLLOW-STEM AUGER	CALIFORNIA SPLIT-SPOON	NONE	NONE

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)			USCS	DESCRIPTIONS/REMARKS
				1			ML	Orange-brown, slightly moist, dense, sandy SILT with minor clay and gravel to 1 inch in diameter
				2				
				3			ML	Grading to medium-brown, slightly moist, medium-dense, sandy SILT with minor clay and gravel
				4				
				5				
2-1	7/17/17	82.5	23.1	6	◢		ML	Grading to grey-green, slightly moist, dense SILT with minor organic debris and rock to 1 inch in diameter
				7			ML	Medium brown, slightly moist, dense, sandy SILT
				8				
				9				
2-2	50 for 3			10	◢		ML	Dark brown, moist, dense, gravely SILT with minor clay
				11				BOTTOM OF BORING AT 10.5 FEET (REFUSAL)
				12				
				13				
				14				
				15				
				16				
				17				
				18				
				19				
				20				

BORING 3

PROJECT NO. 469-01	PROJECT NAME IDAHO MARYLAND MINE	ELEVATION 2715 feet	DATE 4/25/96	PAGE 1 OF 3	FIGURE NO. 4
DRILL METHOD CME 75 - 8" HOLLOW-STEM AUGER	SAMPLING METHOD CALIFORNIA SPLIT-SPOON	GROUNDWATER ENCOUNTERED 27 feet	CAVED 30 feet		

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)			USCS	DESCRIPTIONS/REMARKS
								Orange-brown, slightly moist, dense, sandy SILT with minor clay and gravel to 1 inch in diameter
				1			ML	
				2				
				3				Grading to medium-brown, slightly moist, medium-dense, sandy SILT with minor clay and gravel
				4			ML	
				5				
				6			ML	Grading to grey-green, slightly moist, dense SILT with minor organic debris and rock to 1 inch in diameter
				7				Medium brown, slightly moist, dense, sandy SILT
				8				
				9				Increasing resistance at 9 feet due to the presence of angular to subrounded rock to 2 inches in diameter
				10				
				11				
				12				
				13			ML	
				14				
				15				
				16				
				17				
				18				
				19				
				20				

BORING 3

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	FIGURE NO.
469-01	IDAHO MARYLAND MINE	2715 feet	4/25/96	2 OF 3	4

DRILL METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75 – 8" HOLLOW-STEM AUGER	CALIFORNIA SPLIT-SPOON	27 feet	30 feet

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)			USCS	DESCRIPTIONS/REMARKS
				21			ML	Medium brown, slightly moist, dense, sandy SILT with rock to 2 inches in diameter (continued from page 1)
				22				Dark brown-grey, very moist, loose, clayey SILT with minor organics
				23				
3-1	1/1/1			24	◢		ML	
				25				Minor gravel at 25 feet
3-2	1/2/2	95.2	29.3	26	◢			Grading to red-brown, wet, loose SILT with minor clay and sand
				27			▼	
				28				
				29				
				30				
				31				
				32				
				33			ML	
				34				
				35				
				36				
				37				
				38				
				39				
				40				

BORING 3

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	FIGURE NO.
469-01	IDAHO MARYLAND MINE	2715 feet	4/25/96	3 OF 3	4

DRILL METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75 – 8" HOLLOW-STEM AUGER	CALIFORNIA SPLIT-SPOON	27 feet	30 feet

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)			USCS	DESCRIPTIONS/REMARKS
								Red–brown, wet, loose SILT with minor clay and sand (continued from page 2)
				41				
				42				
				43			ML	
				44				
				45				
								Increasing resistance at 46 feet
				46				Dark brown, very moist, medium dense SILT with clay
				47			ML	
				48				Increasing rock structure at 48 feet
3-3	30/50 for 4							
				49				BOTTOM OF BORING AT 49 FEET
				50				
				51				
				52				
				53				
				54				
				55				
				56				
				57				
				58				
				59				
				60				

BORING 4

PROJECT NO. 469-01	PROJECT NAME IDAHO MARYLAND MINE	ELEVATION 2715 feet	DATE 4/25/96	PAGE 1 OF 1	FIGURE NO. 5

DRILL METHOD CME 75 – 8" HOLLOW-STEM AUGER	SAMPLING METHOD CALIFORNIA SPLIT-SPOON	GROUNDWATER ENCOUNTERED NONE	CAVED 12.5 feet

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1 2 3	ML	Orange-brown, slightly moist, dense sandy SILT
4-1	8/13/17	87.7	28.9	4 5 6	ML	Grading to red-brown, slightly moist, medium dense, sandy SILT interbeded with blue-grey, slightly moist, medium-dense, gravely SILT Sawdust in shoe of sampler
				7 8	ML	Dark brown, slightly moist, medium dense, sandy SILT
4-2	10/20/15	111.0	4.9	9 10 11 12	ML	Grading to dark to light brown, slightly moist, medium-dense, sandy SILT with angular rock fragments to 1 inch in diameter
				13 14 15 16 17	ML	Grading to medium brown, slightly moist, dense, sandy SILT with rock fragments to 2 inches in diameter
				18 19 20		BOTTOM OF BORING AT 17.5 FEET (REFUSAL)

BORING 5

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	FIGURE NO.
469-01	IDAHO MARYLAND MINE	2716 feet	4/25/96	1 OF 1	6

DRILL METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75 - 8" HOLLOW-STEM AUGER	CALIFORNIA SPLIT-SPOON	NONE	NONE

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)			USCS	DESCRIPTIONS/REMARKS
								Orange-brown, moist, dense sandy SILT
				1				
CB-1				2			ML	
				3				
				4				Green-grey, slightly moist, medium-dense, gravely, sandy SILT
				5			ML	
5-1	7/10/8	97.9	8.5	6				
				7				Orange-brown to medium brown, slightly moist, medium dense, sandy SILT with rock to 3 inches in diameter
				8				
5-2	15/12/10			9				
				10			ML	
				11				
				12				
				13				
				14				
				15				BOTTOM OF BORING AT 14.5 FEET (REFUSAL ON ROCK)
				16				
				17				
				18				
				19				
				20				

BORING 6

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	FIGURE NO.
469-01	IDAHO MARYLAND MINE	2715 feet	4/25/96	2 OF 2	7

DRILL METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75 - 8" HOLLOW-STEM AUGER	CALIFORNIA SPLIT-SPOON	NONE	NONE

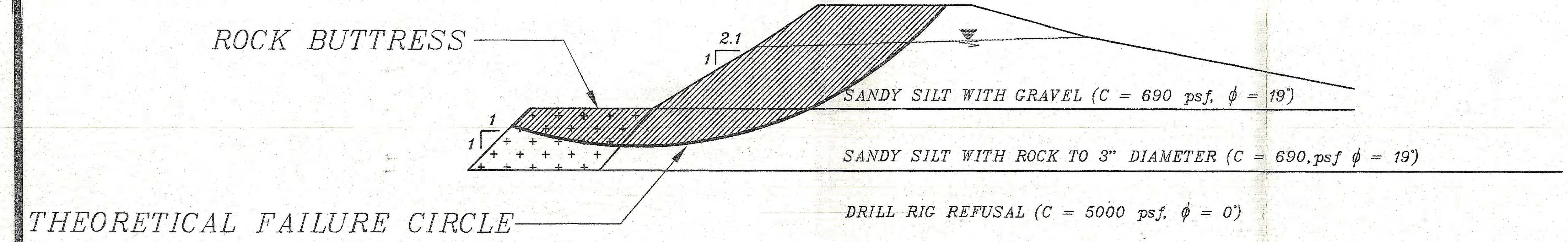
SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)			USCS	DESCRIPTIONS/REMARKS
								Sawdust (continued from page 1)
6-1	1/1/1			21			ML	Grades to dark brown-grey, wet, loose, clayey SILT
				22				BOTTOM OF BORING AT 21.5 FEET
				23				
				24				
				25				
				26				
				27				
				28				
				29				
				30				
				31				
				32				
				33				
				34				
				35				
				36				
				37				
				38				
				39				
				40				

BORING 6

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	FIGURE NO.
469-01	IDAHO MARYLAND MINE	2715 feet	4/25/96	1 OF 2	7

DRILL METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75 - 8" HOLLOW-STEM AUGER	CALIFORNIA SPLIT-SPOON	NONE	NONE

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)			USCS	DESCRIPTIONS/REMARKS
								Red-brown, slightly moist, dense, sandy SILT with minor clay
				1				
				2				
				3			ML	
				4				
				5				
				6				
				7				Grades to dark brown, slightly moist, medium-dense, sandy SILT
				8				
				9				
				10			ML	Increasing resistance at 10 feet
				11				
				12				
				13				Seepage at 13 feet No sawdust observed on auger bit at 13.5 feet
				14				
				15				Sawdust
				16				
				17				
				18				
				19				
				20				



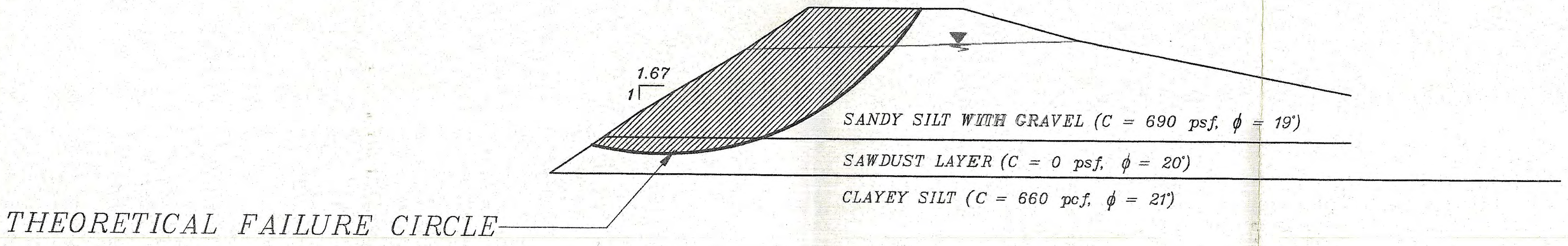
NORTH SECTION A–A'

J: \R13\469-07\PROFILE

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS
308 MAIN STREET, STE. 5
NEVADA CITY, CA 95959
(916) 478-1305 FAX 478-1019

North Section of Retention Basin Dam
IDAHO MARYLAND MINE
Nevada County, California

DRAWN BY: CRK
CHECKED BY: CRK
PROJECT NO.: 469-01
DATE: MAY 1996
FIGURE NO.: 8



SOUTH SECTION B–B’

J:\R13\469-01\PROFILE

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308 MAIN STREET, STE. 5
NEVADA CITY, CA 95959
(916) 478-1305 FAX 478-1019

South Section of Retention Basin Dam
IDAHO MARYLAND MINE
Nevada County, California

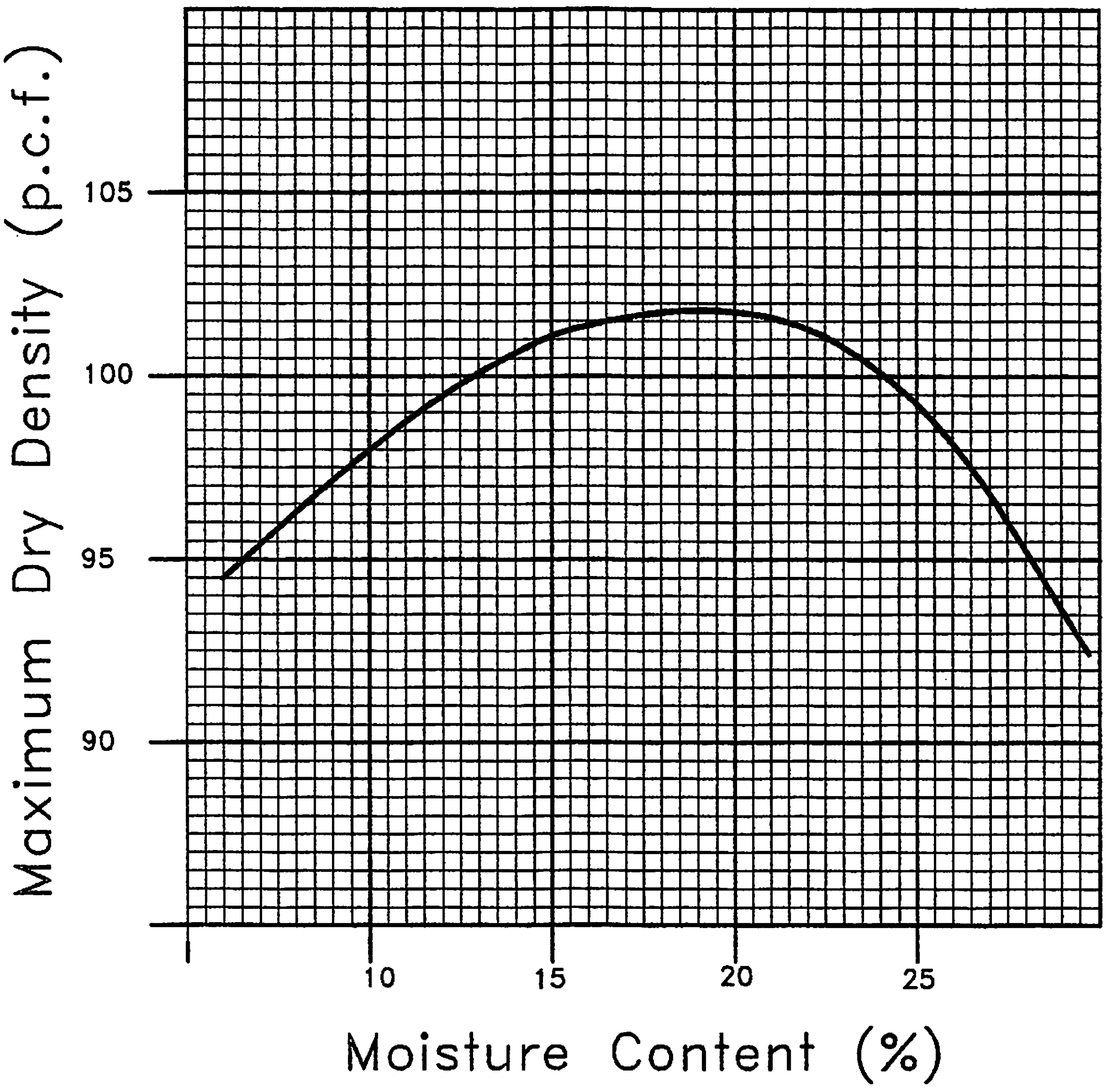
DRAWN BY: CRK
CHECKED BY: CRK
PROJECT NO.: 469-01
DATE: MAY 1996
FIGURE NO.: 9

SAMPLE NO. CB-1

SAMPLE DESCRIPTION: Red-brown to orange-brown sandy SILT with minor clay and organics

MAXIMUM DRY DENSITY: 101.8 P.C.F.

OPTIMUM MOISTURE: 19.0%



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COMPACTION CURVE
ASTM D-1557

PROJECT NO. 469-01

MAY 1996

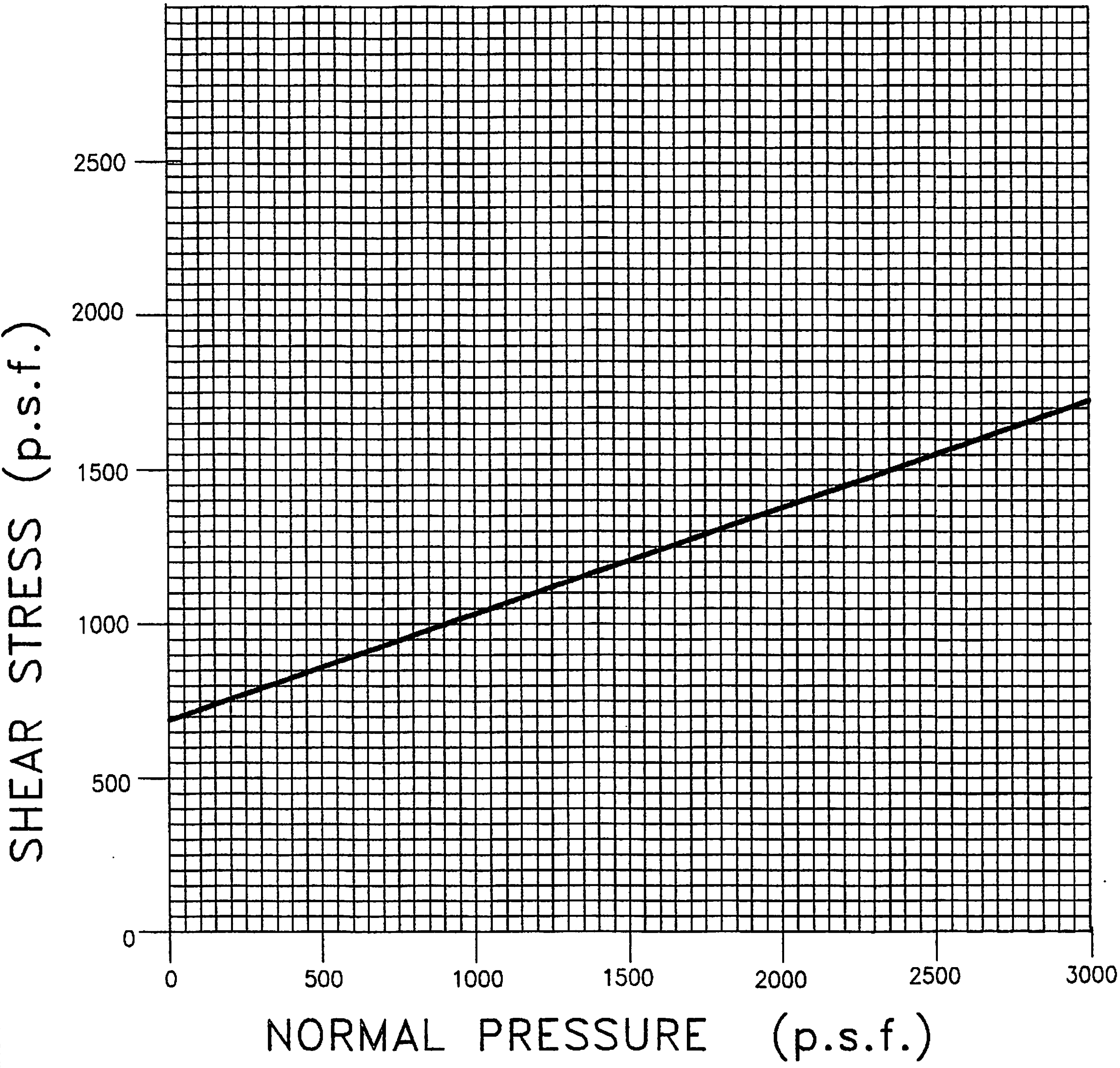
FIGURE 10

SAMPLE NO. 1-1

SAMPLE DESCRIPTION: Red-brown to orange-brown sandy SILT with minor clay and organics

PHI ANGLE: 19 degrees

COHESION: 690 p.s.f.



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NEVADA CITY, CA 95959
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DIRECT SHEAR RESULTS ASTM D-3080

PROJECT NO. 469-01

MAY 1996

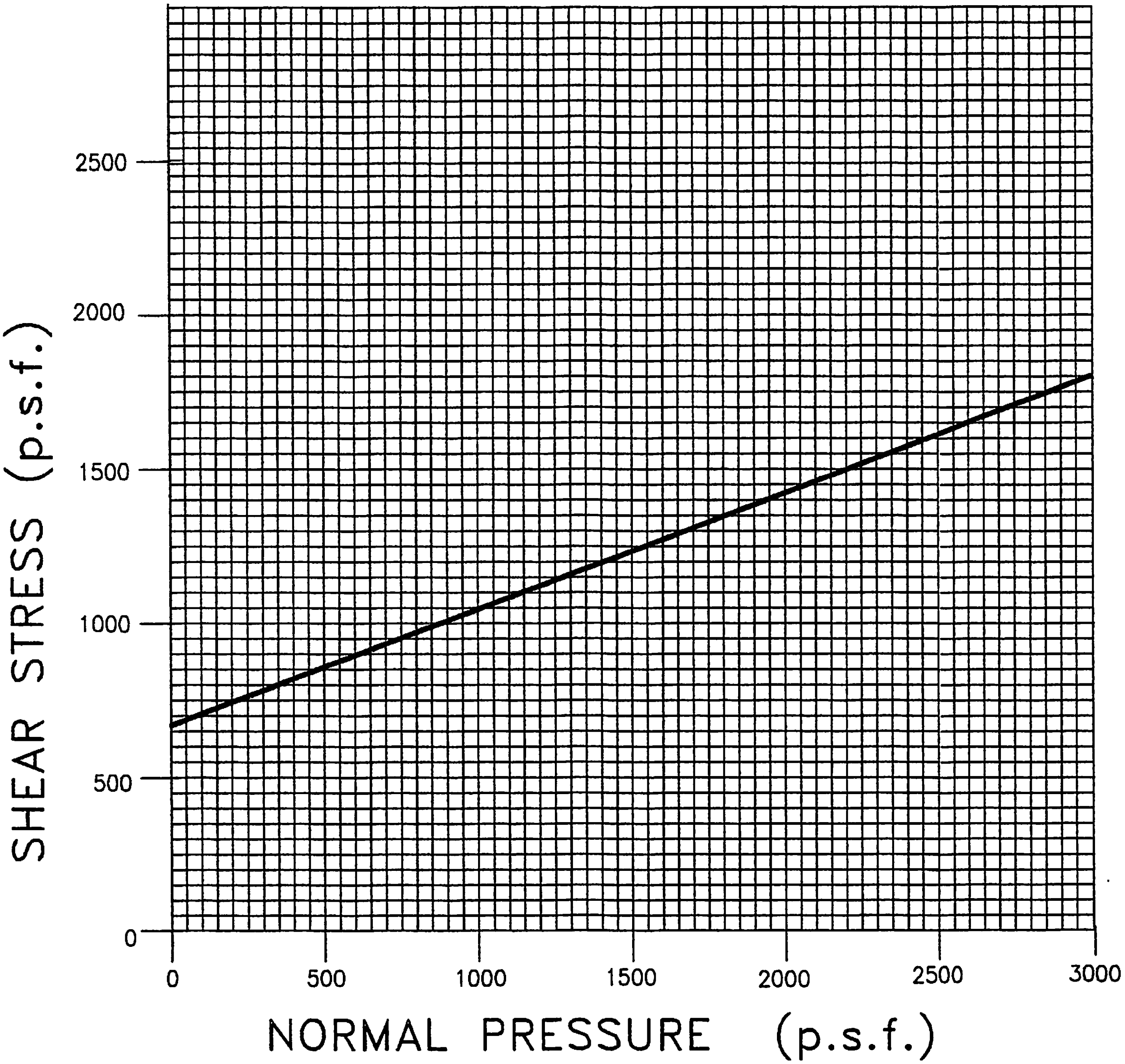
FIGURE 11

SAMPLE NO. 6-1

SAMPLE DESCRIPTION: Dark brown-grey, clayey SILT

PHI ANGLE: 21 degrees

COHESION: 660 p.s.f.



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308 MAIN STREET, STE. 5
NEVADA CITY, CA 95959
(916) 478-1305 FAX 478-1019

DIRECT SHEAR RESULTS ASTM D-3080

PROJECT NO. 469-01

MAY 1996

FIGURE 12

Table 2 - Moisture Density and Direct Shear Test Results

Sample Number	Depth (feet)	Moisture content (% dry wt.)	Dry Density (p.c.f.)	Cohesion (p.s.f.)	Friction Angle (degrees)
1-1	1.5	--	--	690	19.0
1-2	5.5	41.7	84.7	--	--
1-5	18.5	42.2	116.8	--	--
1-6	21.0	38.2	125.9	--	--
1-7	25.5	37.9	124.0	--	--
2-1	5.5	23.1	123.0	--	--
3-2	25.5	29.3	142.0	--	--
4-1	5.5	28.9	130.7	--	--
4-2	11.0	4.9	165.5	--	--
5-1	5.5	8.5	146.0	--	--
6-1	20.5	--	--	660	21.0

STL

SIERRA TESTING LABORATORIES, INC.

GEOTECHNICAL AND MATERIALS TESTING SERVICES

Project 96-125
14 May 1996

Holdrege & Kull
308 Main Street, Suite 5
Nevada City, California 95959

Attention: Mr. Chuck Kull

Subject: **Project # 469-01**
LABORATORY TEST RESULTS

Dear Chuck:

As requested, *Sierra Testing Laboratories, Inc*. has performed laboratory testing on an undisturbed sample, from the above referenced project. The sample was identified as 1-8. The sample was received by our laboratory on 2 May 1996.

The test performed on the submitted sample was as follows:

1) Consolidation, (ASTM D2435).

The results of the Consolidation test are displayed on Figure 1, attached.

We appreciate the opportunity to be of service to you on this project and look forward to providing additional service, as needed, in the future.

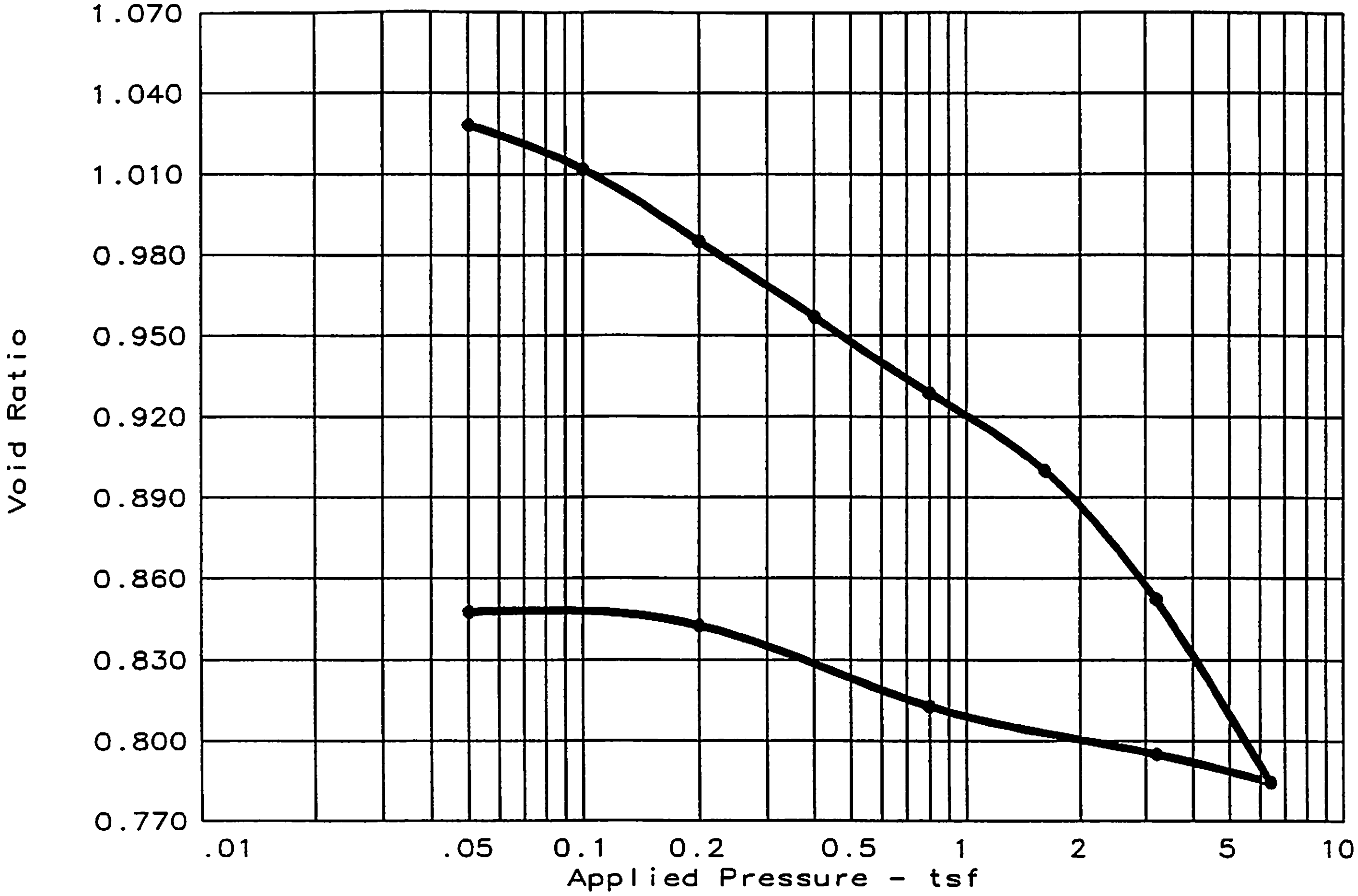
Should you have any questions or require additional information, please contact our office at your convenience.

Very truly yours,
Sierra Testing Laboratories, Inc.

Mark E. Leitschuh
Laboratory Manager

mel:mpw
copies: 2 to Holdrege & Kull, Mr. Chuck Kull
enclosure: Figure 1.

CONSOLIDATION TEST REPORT



Coefficients of Consolidation (sq. in./min.)								
No.	Load	Cv	No.	Load	Cv	No.	Load	Cv
5 6	0.80 1.60	0.025 0.008						

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (tsf)	C_c	e_o
99.4 %	41.5 %	78.6			2.650	2.16	0.22	1.1052

TEST RESULTS

Compression Index = 0.22

MATERIAL DESCRIPTION

Dark Brown lean CLAY w/sand

Remarks:

Project No.: 96-125
Project: # 469-01
Location: 1-8

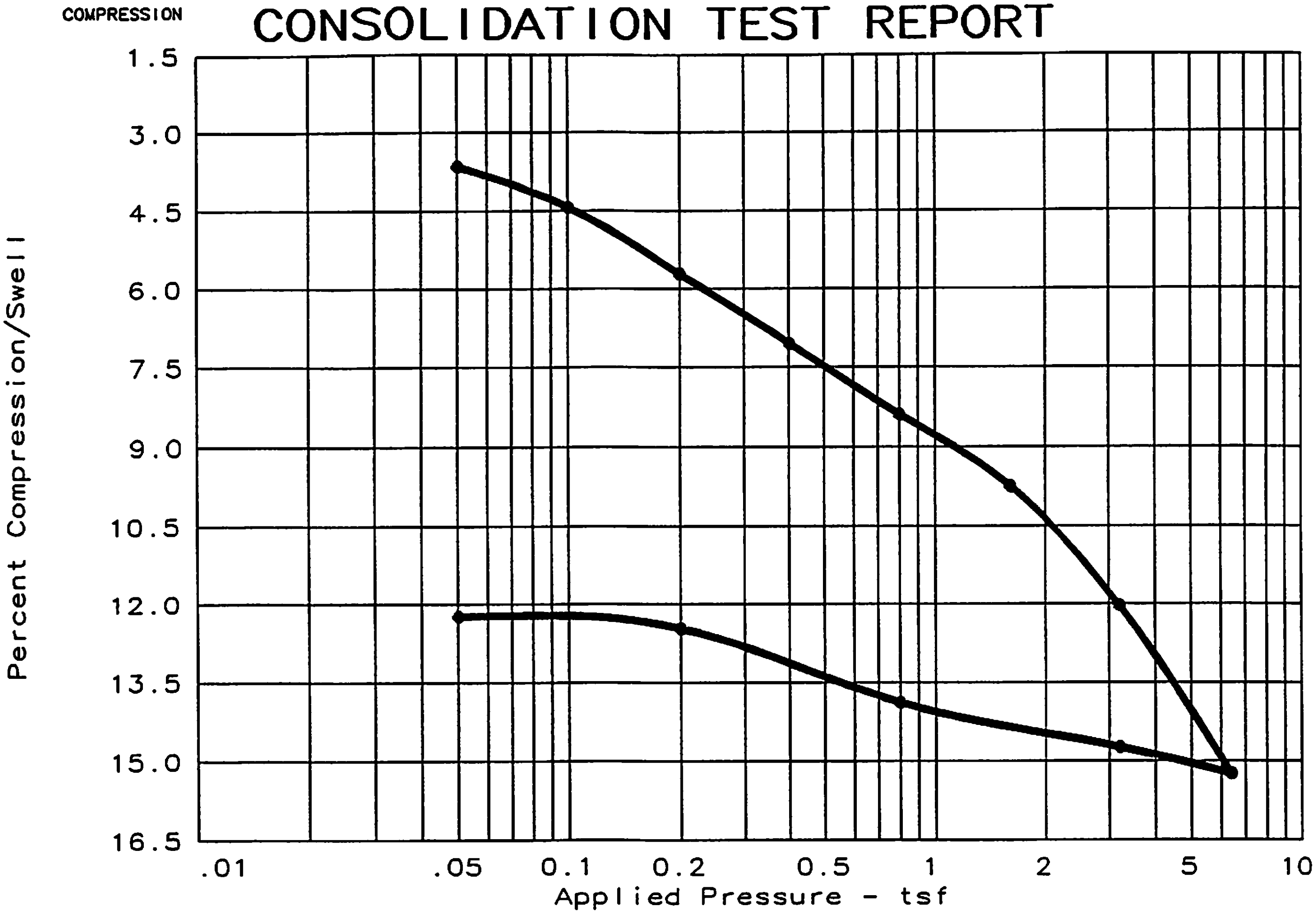
Date: 5-7-96

CONSOLIDATION TEST REPORT

SIERRA TESTING LABORATORIES

Fig. No. 1

CONSOLIDATION TEST REPORT



Coefficients of Consolidation (sq. in./min.)								
No.	Load	Cv	No.	Load	Cv	No.	Load	Cv
5	0.80	0.025						
6	1.60	0.008						

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (tsf)	C_c	e_o
99.4 %	41.5 %	78.6			2.650	2.16	0.22	1.1052

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.22	Dark Brown lean CLAY w/sand

Project No.: 96-125
Project: # 469-01
Location: 1-8

Date: 5-7-96

CONSOLIDATION TEST REPORT

SIERRA TESTING LABORATORIES

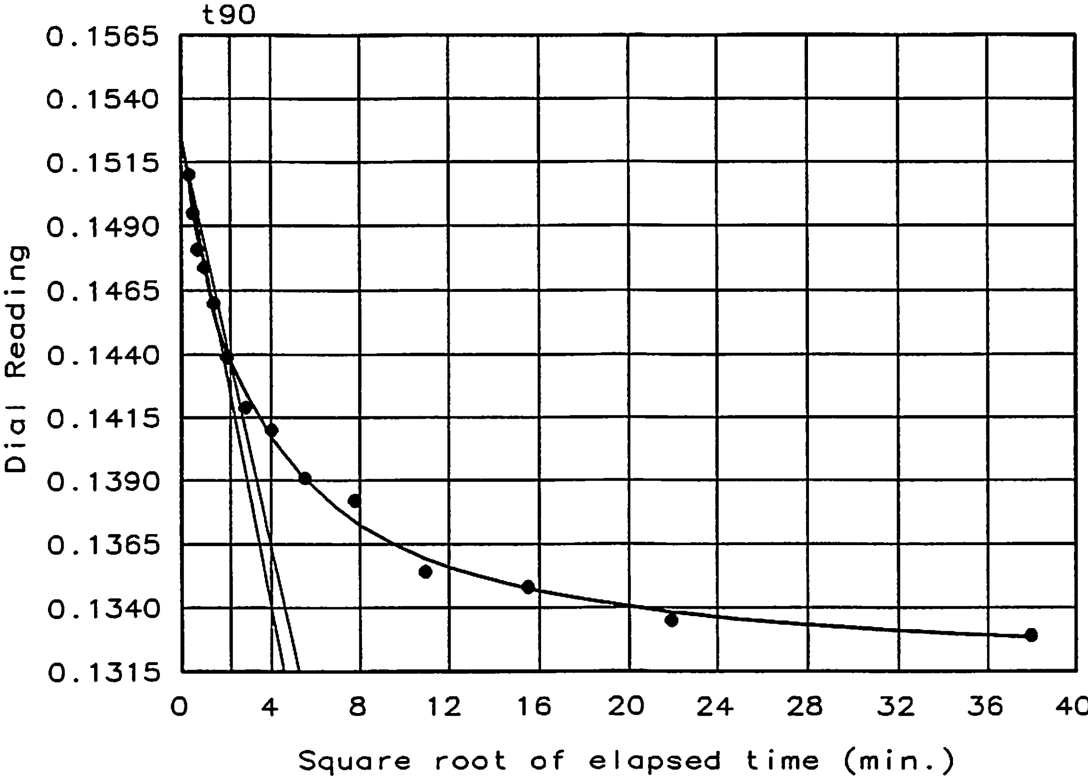
Remarks:

Fig. No. 1

Dial Reading vs. Time

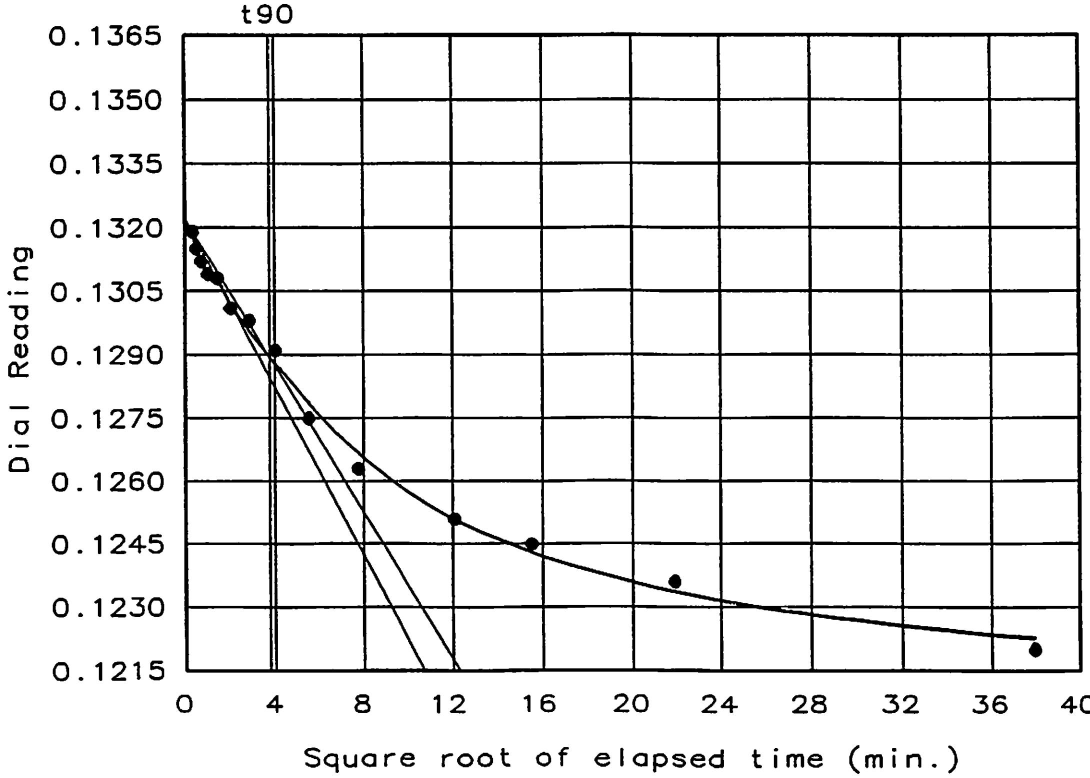
Project No.: 96-125
Project: # 469-01
Location: 1-8

Date: 5-7-96



Load No.= 5
Load= 0.80 tsf
D_0 = 0.1525
D_{90} = 0.1437
D_{100}= 0.1427
T_{90}= 4.74 min.

C_v @ T_{90} =
.025 in.^{2}/min.



Load No.= 6
Load= 1.60 tsf
D_0 = 0.1322
D_{90} = 0.1290
D_{100}= 0.1286
T_{90}= 14.08 min.

C_v @ T_{90} =
.008 in.^{2}/min.

```
================================================================================
03:10, 5-10-1996       CONSOLIDATION TEST PROJECT DATA                Test No. 32
================================================================================

   Project Number:       96-125
   Project:              # 469-01
   Date:                 5-7-96
   Location 1:           1-8
            2:
   Remarks 1:
           2:
           3:
           4:
           5:
   Material description Dark Brown lean CLAY
                         w/sand
   Classification:
   Liquid limit:
   Plasticity index:
   Figure Number:        1

================================================================================
                         CONSOLIDATION TEST SPECIMEN DATA
================================================================================

    TOTAL SAMPLE            BEFORE TEST                     AFTER TEST
Wet w+t  =  170.30 g.   Oedometer No. = 7            Wet w+t  = 116.40 g.
Dry w+t  =  134.90 g    Machine No.   = N/A          Dry w+t  = 99.90  g.
Tare wt. =   49.50 g.   Spec. Gravity = 2.650        Tare wt. = 51.40  g.
Height   =  0.8000 in.  Height    =  0.8000 in.
Diameter =  1.9400 in.  Diameter =  1.9400 in.
Weight   =   69.00 g.

Moisture =  41.5 %      Ht. Solids = 0.3800 in.      Moisture     =   34.0 %
Wet Den. = 111.2 pcf    Dry wt.    =  48.78 g. *     Dry wt.      = 48.50  g.
Dry Den. =  78.6 pcf    Void ratio = 1.1052          Void ratio = 0.8476
                        Saturation =   99.4 %
 *  Initial dry weight used in calculations
================================================================================
                       CONSOLIDATION TEST READINGS SUMMARY
================================================================================

    LOAD       DIAL      DEFLECTION     CORRECTED      VOID RATIO   % SWELL/COMPRS.
   (tsf)      (in.)        (in.)       DIAL (in.)
  Initial    0.20000                                    1.1052
     0.05    0.17080      0.0000        0.17080         1.0283         3.7 Comprs.
     0.10    0.16450      0.0000        0.16450         1.0118         4.4 Comprs.
     0.20    0.15430      0.0000        0.15430         0.9849         5.7 Comprs.
     0.40    0.14360      0.0000        0.14360         0.9568         7.1 Comprs.
     0.80    0.13290      0.0000        0.13290         0.9286         8.4 Comprs.
     1.60    0.12200      0.0000        0.12200         0.8999         9.8 Comprs.
     3.20    0.10380      0.0000        0.10380         0.8520        12.0 Comprs.
     6.40    0.07810      0.0000        0.07810         0.7844        15.2 Comprs.
     3.20    0.08210      0.0000        0.08210         0.7949        14.7 Comprs.
     0.80    0.08890      0.0000        0.08890         0.8128        13.9 Comprs.
     0.20    0.10020      0.0000        0.10020         0.8426        12.5 Comprs.
     0.05    0.10210      0.0000        0.10210         0.8476        12.2 Comprs.
```

```
==============================================================================
                          CONSOLIDATION TEST RESULTS
==============================================================================

Compression index = 0.22
Preconsolidation pressure = 2.16 tsf

==============================================================================
Load      0.80 tsf        CONSOLIDATION TEST READINGS                Load No.  5
==============================================================================
                No.    Elapsed    Dial      No.    Elapsed    Dial
                         Time    Reading             Time    Reading
                 1       0.00    0.15360    11       60.00    0.13820
                 2       0.10    0.15100    12      120.00    0.13540
                 3       0.25    0.14950    13      240.00    0.13480
                 4       0.50    0.14810    14      480.00    0.13350
                 5       1.00    0.14740    15     1440.00    0.13290
                 6       2.00    0.14600
                 7       4.00    0.14390
                 8       8.00    0.14190
                 9      16.00    0.14100
                10      30.00    0.13910

   Void Ratio: 0.9286  Compression: 8.4 %
   D0 = 0.1525    D90 = 0.1437    D100 = 0.1427    T90 = 4.74 min.
   Cv @ 4.7 min.= 0.025 sq. in./min.
==============================================================================
Load      1.60 tsf        CONSOLIDATION TEST READINGS                Load No.  6
==============================================================================
                No.    Elapsed    Dial      No.    Elapsed    Dial
                         Time    Reading             Time    Reading
                 1       0.00    0.13290    11       60.00    0.12630
                 2       0.10    0.13190    12      146.00    0.12510
                 3       0.25    0.13150    13      240.00    0.12450
                 4       0.50    0.13120    14      480.00    0.12360
                 5       1.00    0.13090    15     1440.00    0.12200
                 6       2.00    0.13080
                 7       4.00    0.13010
                 8       8.00    0.12980
                 9      16.00    0.12910
                10      30.00    0.12750

   Void Ratio: 0.8999  Compression: 9.8 %
   D0 = 0.1322    D90 = 0.1290    D100 = 0.1286    T90 = 14.08 min.
   Cv @ 14.1 min.= 0.008 sq. in./min.
```

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

Project No. 469-05
April 29, 1997

Mr. Tom Knoch
c/o Emperor Gold Corporation
12503 Brunswick Road
Grass Valley, California 95945

Reference: Idaho-Maryland Mine
Brunswick and Bennett Roads
Grass Valley, California

Subject: ***Foundation Recommendations and Design Criteria for Proposed Hoist Building***

Dear Tom:

This letter provides recommendations and design criteria for the proposed hoist building to be located west of the existing shaft of the Idaho-Maryland Mine. In a separate letter, we will provide a cost estimate for construction of a pier foundation for the proposed head frame to be located over the existing shaft of the Idaho-Maryland Mine.

Purpose

The purpose of our study was to perform the tasks outlined in our *Revised Proposal to Provide Foundation Recommendations and Design Criteria*, dated April 15, 1997. Our services included a subsurface investigation and laboratory testing program to provide foundation recommendations and design criteria for construction of the proposed hoist building.

(916) 478-1305 • FAX (916) 478-1019 • E-mail: holdkull@gv.net • 792 Searls Avenue • Nevada City, CA 95959 • A California Corporation

Scope of Services

In order to prepare this report, we performed the following:

- Hand-augered two exploratory borings at the locations shown on Figure 1.
- Obtained relatively undisturbed soil samples from the borings.
- Performed laboratory tests on samples obtained during our field investigation. Testing included direct shear, confined swell and moisture/density.
- Analyzed laboratory test results and performed calculations to provide criteria for foundation design.

Project Location and Description

The proposed hoist building is to be located approximately 100 feet west of the existing New Brunswick Shaft at Idaho-Maryland Mine in Grass Valley. The footprint of the proposed building is approximately 50 feet by 150 feet; the long axis of the building is oriented approximately north-south.

We understand that a conventional perimeter footing and slab-on-grade floor will be used for the proposed hoist building. Two hoist foundations are to be placed inside the building, one of which will include a massive hoist counterweight. The proposed foundation design and loadings are to be provided by G.L. Tiley and Associates.

Field Investigation

We performed our field investigation on April 21, 1997. Our field representative hand-augered two exploratory borings in the area of the proposed hoist building. Approximate locations of the borings are shown on Figure 1.

The soil/rock conditions described in the following paragraphs are generalized, based on the two exploratory borings. Figures 2 and 3 are logs of the borings.

The upper 3 inches of the borings was composed of compacted aggregate base rock. The aggregate base rock was underlain by red-brown, slightly moist, dense, native, clayey silt and silty clay with sand. In Boring 2, this soil layer extended to the bottom of the boring at a depth of 24 inches. In Boring 1, the red-brown silty sand/sandy silt graded into orange, slightly moist, dense, residually weathered volcanic rock at a depth of 10 inches. The residually weathered rock excavated as a clayey silt with sand.

Laboratory Testing

Laboratory testing included one direct shear, one confined swell and two moisture/density tests. The sample tested for direct shear had a friction angle of 29 degrees and a cohesion intercept of 820 p.s.f. The shear test was performed to determine soil bearing capacity and frictional resistance to sliding. The swell test had less than 1 percent swell under a confining pressure of 225 p.s.f. The confined swell test was performed to determine the presence of potentially expansive soils. The results can be found in Tables 1 and 2 at the end of this letter report.

DESIGN CRITERIA

Following are design criteria for the hoist and hoist building foundations. The compressive loads will be resisted by vertical bearing on native and fill soil. Uplift and sliding will be further analyzed when foundation loads are submitted by G.L. Tiley and Associates.

Bearing Capacity

An allowable bearing capacity of 3000 psf can be used for design provided the foundation is constructed in native soil or in fill placed in accordance with our recommendations. The bearing pressure is for dead loads and live loads. A 33% increase can be used for short term live loads.

Lateral Resistance

Lateral forces will be resisted by a combination of passive pressure and friction. A frictional resistance of 0.4 can be used for design in fill and native soil conditions, provided fill is compacted to a minimum of 95% relative compaction.

A passive pressure of 400 pcf can be used in conjunction with the frictional coefficient providing that concrete is placed neat against undisturbed soil and the values of both frictional resistance and passive pressure are considered ultimate values. The passive pressure should not be used for any footings located within 18 inches of the ground surface.

LIMITATIONS

Review, Observation and Testing

The recommendations and conclusions in this letter report are preliminary in nature based on the limited subsurface investigation conducted. The recommendations provided herein are contingent upon our review of final plans and specifications, and upon observation and testing of fill during grading. A representative from H&K should observe pier holes or footing excavations prior to the placement of concrete to confirm that the work is in accordance with our recommendations.

Uniformity of Conditions

The recommendations in this report are based on the assumption that the soil conditions do not deviate from those we observed on site. If, during construction, different subsurface conditions from those revealed during our investigation are observed, we must be advised promptly. We can then review those conditions and reconsider our recommendations where necessary.

Services Provided

We issue this report with the understanding that it is the responsibility of the owner, or their agent to furnish the information contained herein to the project architect and engineers so that it can be incorporated into the project plans. The conclusions and recommendations contained herein are professional opinions derived in accordance with the current standards of professional practice. No warranty, expressed or implied, including any implied warranty of merchantability or fitness for the purpose is made or intended in connection with our work. We did not conduct any studies to determine the presence of hazardous materials.

Time Limitations

The findings of this report are valid as of the present date. However, changes in the conditions of the property can occur with the passage of time. The changes may be due to natural processes or to the works of man, on the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or the broadening of knowledge. Therefore this report should not be relied upon for design after two years without our review.

Please contact us if you have any questions.

Most sincerely,

HOLDREGE & KULL

FOR
Jason Muir
Staff Engineer

Charles R. Kull
G.E. 2359/C.E.G. 1622

REGISTERED PROFESSIONAL ENGINEER
CHARLES R. KULL
No. 2359
Expires 6/99
GEOTECHNICAL
STATE OF CALIFORNIA

copies: G.L. Tiley & Associates

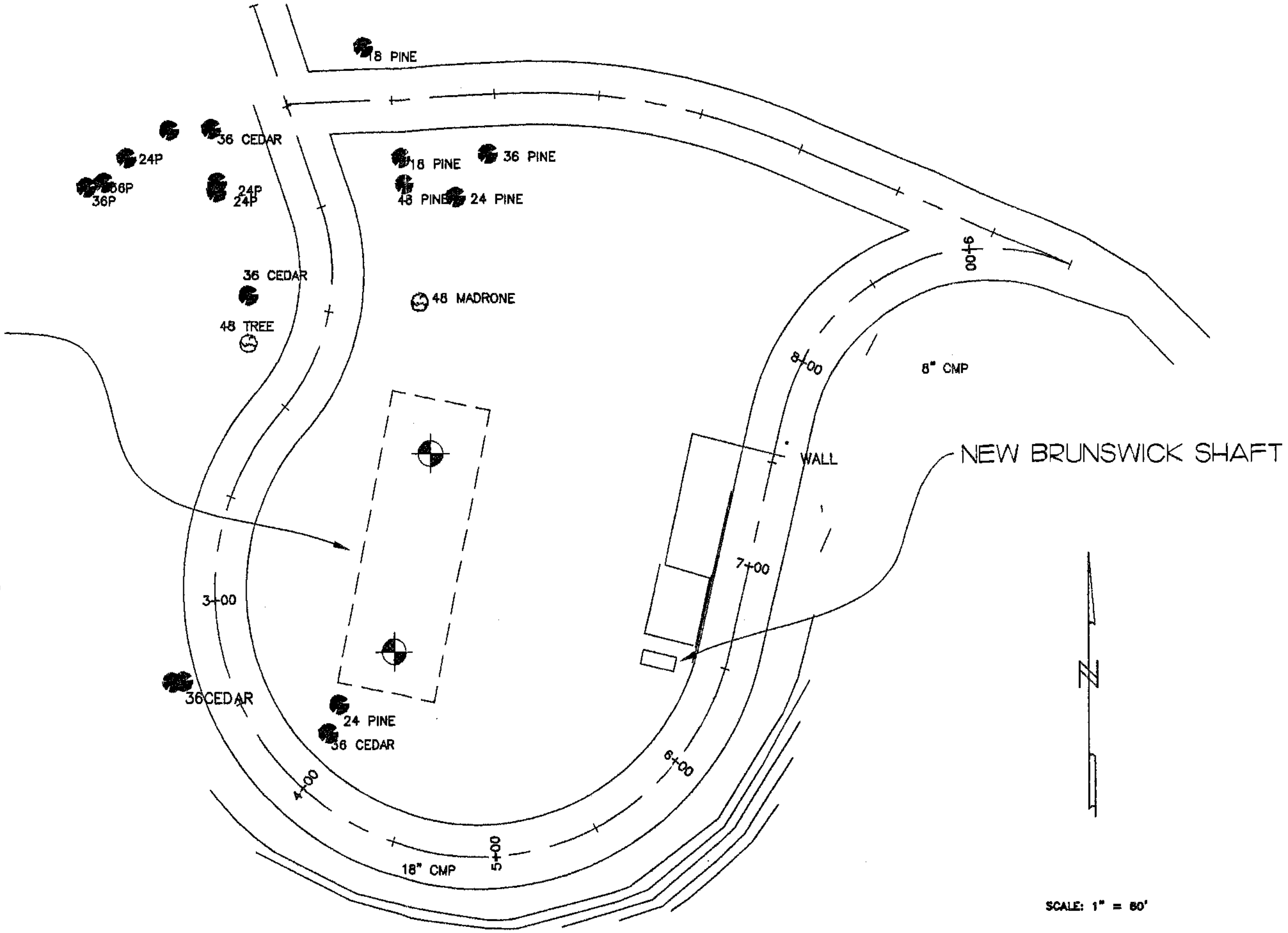
J:\WPDOCS\LET\469-05.FND

Table 1 - Moisture Density and Direct Shear Test Results

Sample Number	Depth (inches)	Moisture Content (% dry wt.)	Dry Density (pcf)	Cohesion (psf)	Friction Angle (degrees)
1-1	5 to 11	27.9	82.5	--	--
1-2	12 to 18	41.7	68	--	--
2-1	4 to 10	27	93	820	29
2-2	18 to 24	--	--	--	--

Table 2 - Confined Swell Test Results

Sample Number	Depth (inches)	Normal Pressure (pcf)	Percent Expansion
1-1	5 to 11	225	1.1



APPROXIMATE LOCATION OF
PROPOSED HOIST BUILDING

NEW BRUNSWICK SHAFT

BORINGS 1 AND 2 WERE HAND-AUGERED
ON APRIL 21, 1997.

LEGEND

⊕ APPROXIMATE BORING LOCATION

J:\R13\EMPMINE\569-05SP.DWG

HK HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS
308 MAIN STREET, STE. 5
NEVADA CITY, CA 95959
(916) 478-1305 FAX 478-1019

STAGING AREA AND ROADWAY
IDAHO-MARYLAND MINE
GRASS VALLEY, CALIFORNIA

DRAWN BY: JWM
CHECKED BY: CRK
PROJECT NO.: 469-05
DATE: APRIL 1997
FIGURE NO.: 1

BORING 1

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	FIGURE NO.
469-05	IDAHO-MARYLAND MINE	UNKNOWN	4/21/97	1 OF 1	2

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
HAND AUGER	HAND DRIVE SAMPLER	NONE	NONE

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)			USCS	DESCRIPTIONS/REMARKS
							GM	COMPACTED AGGREGATE BASE ROCK
1-1		XXX.X	XX.X		◣		CL	RED-BROWN, SLIGHTLY MOIST, DENSE, SILTY CLAY WITH SAND
				1				
1-2		XXX.X	XX.X		◣		ML	GRADES TO ORANGE, SLIGHTLY MOIST, DENSE, RESIDUALLY WEATHERED VOLCANIC ROCK WITH IRON STAINING IN FRACTURES (EXCAVATES AS CLAYEY, SILTY SAND)
								BOTTOM OF BORING AT 1.5 FEET
				2				
				3				
				4				
				5				
				6				
				7				
				8				
				9				
				10				

HOLDREGE & KULL

BORING 2

PROJECT NO. 469-05	PROJECT NAME IDAHO-MARYLAND MINE	ELEVATION UNKNOWN	DATE 4/21/97	PAGE 1 OF 1	FIGURE NO. 3

DRILLING METHOD HAND AUGER	SAMPLING METHOD HAND DRIVE SAMPLER	GROUNDWATER ENCOUNTERED NONE	CAVED NONE

SAMPLE NO.	BLOW COUNTS	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					GM	COMPACTED AGGREGATE BASE ROCK
						RED-BROWN, SLIGHTLY MOIST, DENSE, CLAYEY, SANDY SILT
2-1		XXX.X	XX.X			
				1	ML	
2-2		XXX.X	XX.X			
				2		
						BOTTOM OF BORING AT 2 FEET
				3		
				4		
				5		
				6		
				7		
				8		
				9		
				10		

PRELIMINARY GEOTECHNICAL ENGINEERING REPORT
for
IDAHO-MARYLAND MINING CORPORATION PROPERTY
Nevada County, California

Prepared for:
Idaho-Maryland Mining Corp.
179 Clydesdale Court
Grass Valley, California 95945

Prepared by:
Holdrege & Kull
792 Searls Avenue
Nevada City, California 95959

Project No. 2416-03
October 25, 2004

HK HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

Project No. 2416-03
October 25, 2004

Idaho-Maryland Mining Corp.
179 Clydesdale Court
Grass Valley, California 95945

Attention: Ross Guenther

Reference: ***Idaho-Maryland Mining Corporation Property***
Idaho-Maryland Mine, New Brunswick, and Roundhole Easement Sites
Nevada County, California

Subject: ***Preliminary Geotechnical Engineering Report***

Dear Mr. Guenther:

This report presents the results of our preliminary geotechnical engineering investigation for three sites on Idaho-Maryland Mining Corporation property. The Idaho-Maryland site encompasses 101 acres and is located south of Whispering Pines Lane and north of East Bennett Road near Grass Valley, California. The 37-acre New Brunswick site is located southwest of the intersection of Brunswick Road and East Bennett Road. The one-acre Roundhole Easement site is located north of Whispering Pines Lane near its intersection with Brunswick Road. We understand that, as currently proposed, the project will include the construction of industrial facilities associated with proposed mining on the Idaho-Maryland Site.

The preliminary findings presented in this report are based on a cursory surface reconnaissance at the site; review of selected geologic, soil survey and historical references; review of previous reports for the property; and our experience with subsurface conditions in the area. Based on our preliminary findings, the project as currently proposed appears to be feasible from a geotechnical engineering standpoint. We should be allowed to perform a subsurface investigation to confirm our preliminary recommendations as part of a design-level geotechnical engineering report. Furthermore, we should be allowed to perform testing and observation services during grading to confirm our design-level recommendations.

Please contact us if you have any questions regarding our observations or the preliminary recommendations presented in this report.

Sincerely,

HOLDREGE & KULL

Prepared by:	Reviewed by:
Zack Washburn Staff Geologist	Jason W. Muir C.E. 60167

copies: 6 to Idaho-Maryland Mining Corp. (one unbound)

F:\1 Job Data\2416 Idaho Maryland Mine Property\2416-03 prelim geotech report.wpd

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APPENDIX
Important Information About Your Geotechnical Engineering Report (included with permission of ASFE, Copyright 2004)

1 INTRODUCTION

At the request of Idaho-Maryland Mining Corporation (IMMC), Holdrege & Kull (H&K) performed a preliminary geotechnical engineering investigation of Idaho-Maryland Mining Corporation property in Nevada County, California. For your review, the Appendix contains a document prepared by ASFE entitled *Important Information About Your Geotechnical Engineering Report*, which summarizes the general limitations, responsibilities, and use of geotechnical reports.

1.1 SITE DESCRIPTION

The project includes three sites on IMMC property. The Idaho-Maryland site encompasses 101 acres and is located south of Whispering Pines Lane and north of East Bennett Road near Grass Valley, California. The 37-acre New Brunswick site is located southwest of the intersection of Brunswick Road and East Bennett Road. The one-acre Roundhole Easement site is located north of Whispering Pines Lane near its intersection with Brunswick Road. The sites are currently in an unincorporated portion of Nevada County adjacent to the city limits of Grass Valley, California. Site boundaries are shown on the attached Sheet 1.

1.2 PROPOSED IMPROVEMENTS

Our understanding of the project as currently proposed is based on our conversation with Mr. Ross Guenther and our review of a conceptual site plans prepared by IMMC dated May 2004. We understand that, as currently proposed, the project will include the construction of industrial facilities associated with proposed mining on the Idaho-Maryland site. A grading plan for the project was not available for our review.

1.3 PURPOSE

The purpose of our preliminary geotechnical investigation was to review pertinent geologic, soil survey and historical information; to review a report previously prepared by H&K; and to observe the site to assess the feasibility of development from a geotechnical engineering standpoint.

1.4 SCOPE OF SERVICES

To prepare this report, we performed the following scope of services:

- We performed a cursory reconnaissance of the site.
- We reviewed selected geologic and soil survey literature.
- We reviewed selected historical maps and literature pertinent to historic mining activity in the vicinity of the site.
- We reviewed our *Preliminary Geotechnical Engineering Report for Milco and Platner Property*, dated April 22, 2003, that pertains to a portion of the subject property.
- Based on observations made during our site reconnaissance, the results of our literature review and our experience with soil conditions in the area, we prepared this report, which provides preliminary geotechnical engineering recommendations for the proposed improvements.

2 SITE INVESTIGATION

The following sections summarize our literature review and field reconnaissance.

2.1 GEOLOGIC SETTING

We reviewed the Geologic Map of the Grass Valley - Colfax Area (A. Tuminas, 1983). According to this map, the Idaho-Maryland site is underlain by early Mesozoic rock associated with the Lake Combie complex. The geology of the western portion of the site is characterized by serpentinized rock. The central portion of the site is underlain by gabbro and diorite and the eastern portion is characterized by massive diabase. The Mesozoic era occurred between approximately 245 and 65 million years before present (MYBP).

The central portion of the New Brunswick site lies on Quaternary alluvium (i.e., water lain sediments deposited in the past 2 million years); the flanks of the site are underlain by massive diabase of the Lake Combie complex.

The northern portion of the Roundhole Easement site is underlain by massive diabase, and the southern portion is characterized by serpentinized rock.

2.2 SITE SOIL CONDITIONS

2.2.1 Idaho-Maryland Site

We reviewed the *Soil Survey of the Nevada County Area, California* (USDA Soil Conservation Service, reissued August 1993). The soil survey indicates that the undisturbed portions of the southwestern part of the Idaho-Maryland site are located in an area typified by Secca-Rock outcrop complex. The soil survey describes the Secca soil type as moderately well drained soil underlain by metabasic or basic rock. Permeability is slow, and partly weathered basic rock is typically encountered at a depth of approximately 45 inches below the ground surface (bgs). Rock outcrop typically comprises 10 to 40 percent of the surface area typified by this complex. The undisturbed portions of the eastern side of the site are located in an area typified by Sites loam. The soil survey describes the Sites soil type as well drained soil underlain by tilted metasedimentary and metabasic rock. Weathered metasedimentary and basic rock is typically encountered at a depth of approximately 78 inches bgs. Permeability is moderately slow. The southern central portion of the site is classified by the soil survey as cut and fill land, which has been altered by methods other than mining. The survey states that deep accumulations of bark may be present at locations previously used as logging deck yards or lumber stack yards. The northwestern portion of the site is underlain by Placer diggings, according to the survey. This soil type occurs along drainage ways that have been placer mined and is typically comprised of gravel with little fines.

2.2.2 New Brunswick Site

The southwestern part of the New Brunswick is underlain by Aiken Loam according to the soil survey. The soil survey describes the Aiken Loam as a well-drained soil that forms on the sides of andesitic flows. According to the survey, permeability of the Aiken Loam soil type is moderately slow and weathered andesite is commonly encountered at about 64 inches bgs. The central portion of the site is characterized by Placer diggings while the northeastern portion is classified as clayey Alluvial Land. The soil survey describes clayey Alluvial Land as a miscellaneous land type consisting of narrow areas of alluvial deposits. These soils are moderately well drained to poorly drained and permeability is moderately slow to very slow.

2.2.3 Roundhole Easement Site

The Roundhole Easement site lies entirely on Secca-Rock outcrop complex according to the soil survey. The properties of this soil type are described above.

2.3 HISTORICAL RESEARCH

We reviewed portions of the following documents pertaining to historic mining activities in the immediate vicinity of the subject property.

2.3.1 Geologic Map of the Grass Valley Quadrangle and Adjacent Area, Nevada County, California

2.3.1.1 Idaho-Maryland Site

The Geologic Map of the Grass Valley Quadrangle and Adjacent Area, Nevada County, California (1939) contained in *The Gold Quartz Veins of Grass Valley, California* (W.D. Johnston, Jr., Geological Survey Professional Paper 194, U.S. Department of the Interior, 1940) depicted the Maryland Mine north of the subject property on the Eureka-Idaho-Maryland vein. The vein strikes west-northwest and dips 50 to 70 degrees to the south-southwest. An underground inclined shaft extended south and southeast from the Maryland Mine. This shaft lies north of the northern boundary of the site, according to the map.

The Geologic Map of the Grass Valley Quadrangle and Adjacent Area, Nevada County, California (1939) shows the South Idaho vein striking west-northwest across the central portion of the site. The vein dips 60 degrees to the south. The South Idaho shaft is shown in the alignment of the vein. A horizontal or inclined tunnel is shown striking east along the vein within the southeast portion of the site.

According to *The Gold Quartz Veins of Grass Valley, California* (W.D. Johnston, Jr., Geological Survey Professional Paper 194, U.S. Department of the Interior, 1940), the Idaho-Maryland shaft inclined to the 1000-foot level at an angle of 70 degrees. The Canyon (or Cañon) shaft, an inclined winze raking to the east from the 1000-foot level, was advanced as far as the 1900-foot level to a depth greater than 2,500 feet bgs. The Eureka-Idaho-Maryland vein strikes north 77 degrees west and has an average dip of 70 degrees southwest, ranging between 50 and 80 degrees. The hanging wall is composed of diabase and gabbro and the footwall is composed of serpentine.

2.3.1.2 New Brunswick Site

The Geologic Map of the Grass Valley Quadrangle and Adjacent Area, Nevada County, California (1939) shows Union Hill Mine in the western part of the site on the Union Hill vein. The vein strikes northwest and dips between 50 and 90 degrees to the southwest. An underground inclined shaft extended south-southwest from the Union Hill Mine beneath the site. The Union Hill shaft was advanced to about 1050 feet bgs, according to the researched documents.

The Lucky and Cambridge shafts lie a few hundred feet west of the site. These shafts are vertical and intersect the Lucky Cambridge vein, which parallels the Union Hill vein. The Lucky shaft was apparently advanced to the 300-foot level, but the depth of the Cambridge shaft is not stated in the *The Gold Quartz Veins of Grass Valley, California* report. The New Brunswick (shown as Brunswick on the map) vertical shaft was included on the 1939 map, but no description of the shaft dimensions was included in the report.

2.3.1.3 Roundhole Easement Site

No information on the Roundhole shaft is provided in the *The Gold Quartz Veins of Grass Valley, California* or shown on the Geologic Map of the Grass Valley Quadrangle and Adjacent Area, Nevada County, California (1939). The approximate location of the Roundhole shaft, as shown on Sheet 1, was provided by IMMC.

2.3.2 Mines and Mineral Resources of Nevada County

We reviewed the *Mines and Mineral Resources of Nevada County* (Errol MacBoyle, California State Mining Bureau, December 1918). This publication contained information regarding the Eureka-Idaho-Maryland vein that was discussed above, and contained additional information regarding the South Idaho Mine. The South Idaho vein is located approximately 1500 feet south of and parallel to the Eureka-Idaho-Maryland vein. The vein is located in diabase and gabbro, strikes north 85 degrees west, and dips 70 degrees to the south. The lode was developed by inclined shaft only to a depth of 155 feet at the time of the 1918 publication. A crosscut was driven south for a distance of 12 feet at a depth of 60 feet, and drifting was performed at the 100-foot level for a distance of 25 feet to the south. A tunnel was driven a distance of 800 feet on the vein east of the shaft location.

This publication also contained information regarding the Union Hill vein and shaft that was discussed above, but did not provide any information on the New Brunswick, Lucky, or Cambridge shafts.

2.3.3 Map of the Grass Valley Quadrangle included in the Nevada City Special Folio, California

The Map of the Grass Valley Quadrangle included *Nevada City Special Folio, California* (United States Geologic Survey, 1896) depicted the features described above that were shown on the 1939 Geologic Map of the Grass Valley Quadrangle and Adjacent Area, Nevada County, California.

2.3.4 Map Showing Mining Properties of the Grass Valley Mining District, Nevada County, California

2.3.4.1 Idaho-Maryland Site

The Map Showing Mining Properties of the Grass Valley Mining District, Nevada County, California (Division of Mines, 1930) showed the Idaho-Maryland site as being located within the Idaho-Maryland Mining Company claim. The map also depicted the Idaho-Maryland vein and shaft, as well as the South Idaho vein and shaft, as discussed above for the 1939 Geologic Map of the Grass Valley Quadrangle and Adjacent Area, Nevada County, California.

2.3.4.2 New Brunswick Site

The Map Showing Mining Properties of the Grass Valley Mining District, Nevada County, California (Division of Mines, 1930) showed the New Brunswick site as being located within the Idaho-Maryland Mining Company claim. The map also depicted the Union Hill vein and shaft, as well as the New Brunswick shaft, as discussed above for the 1939 Geologic Map of the Grass Valley Quadrangle and Adjacent Area, Nevada County, California.

2.3.4.3 Roundhole Easement site

The Map Showing Mining Properties of the Grass Valley Mining District, Nevada County, California (Division of Mines, 1930) did not show the Roundhole shaft, but did depict the area of the Roundhole shaft as being within the Idaho-Maryland Mining Company claim.

2.4 REVIEW OF OTHER REPORTS

In our 2003 *Preliminary Geotechnical Engineering Report for Milco and Platner Property*, we reviewed a report prepared by Neil O. Anderson and Associates, Inc. entitled *Preliminary Geotechnical Investigation, Grass Valley Business and Professional Park Between Idaho Maryland and East Bennett Rd, Grass Valley, California* (August 15, 1991). The 1991 investigation included the excavation of 22 exploratory trenches on portions of the Idaho-Maryland site. The trenches ranged from 2.5 to 13.5 feet deep.

The Anderson and Associates report described two previously graded areas on the Idaho-Maryland site that have loose, organic fill. One previously graded area lies in southeastern part of the site at the base of a cut slope. Fill up to 3.5 feet deep was encountered in the exploratory trenches excavated on the northern portion of this previously graded area. The fill was generally described as sand and gravel with organic material and occasional larger rock. Fill deeper than 13.5 feet was encountered in the exploratory trenches excavated on the southern end of this area. The fill was generally described as clay with abundant wood debris, as well as sand and gravel. There are no proposed structures shown on the site plan in the vicinity of the eastern previously graded area.

The other previously graded area lies in the southern part of the Idaho-Maryland site beneath the proposed employee parking area. Fill up to 4 feet deep was encountered in the exploratory trenches excavated on the perimeter of this area. The fill was generally described as sawdust, wood chips, clay, sand and gravel.

Atterberg Limits testing was performed using five bulk samples of clay soil obtained from depths ranging from 2.5 to 5.5 feet bgs in the exploratory trenches. Plasticity Indices ranged from 13 to 32, and Liquid Limits ranged from 37 to 57.

The Anderson and Associates report concluded that the relatively loose, organic fill onsite would not be suitable to support structural improvements.

We did not review any reports that pertain to the New Brunswick or Roundhole Easement sites.

2.5 FIELD INVESTIGATION

We performed a surface reconnaissance of the project sites on October 15 and 18, 2004 to observe near surface soil conditions and visible evidence of potential geologic hazards that may be present.

2.5.1 Surface Conditions

2.5.1.1 Idaho-Maryland Site

We have subdivided the Idaho-Maryland site into three sections to clarify discussion of the surface conditions at this site. We define the main area, southern area, and the southeastern area based on proposed use and topography.

The main area occupies the largest portion of the site and will include the ceramics plant and the majority of the other proposed improvements. The main area is located in the northwest part of the Idaho-Maryland site and had topography that slopes gently to the northwest. The slopes ranged from less than 5 percent along the perimeter of the main area to about 25 percent beneath the ceramics plant. The gently sloping areas (<5%) appeared to have been previously graded and much of their surfaces were covered with waste rock, presumably associated with past hard rock mining. The steeper slope in the vicinity of the proposed ceramics plant was covered with piles of waste rock up to 10-feet in height. A ditch also crossed this slope in the vicinity of the proposed ceramics plant. The western part of the central area was relatively flat lying and had patchy areas of sandy material on the surface. Two approximately 40-foot high reinforced concrete towers were seen in the northwestern portion of the main area. Site elevations ranged from approximately 2490 feet above mean sea level (MSL) near the concrete towers to 2560 feet above MSL in the northeastern part of the main area.

Topography of the southern area was dominated by the western end of a small, west-trending ridge and the land that sloped away from the ridge to the north, south and west. The native soil had been cut from the ridge top and deposited along the edges of the resulting flat-lying area. A short, timber crib wall retained less than 5 feet of fill on the southern edge of the previously graded area, immediately north of a dirt access road. The remainder of the property appeared to consist primarily of undisturbed native soil. Elevations ranged from approximately 2620 feet above MSL on the ridge near the eastern property boundary to approximately 2530 feet above MSL near the southwest property corner. Slope gradients ranged from approximately 2 to 8 percent on the previously graded ridge top and from approximately 2:1, horizontal to vertical (H:V) on the land sloping away from the ridge.

The southeastern area contains the previously graded area, cut slopes, and a steep natural slope along the eastern boundary. The southeastern area was relatively flat-lying and characterized by extensive cut and fill associated with past lumber milling activities. Several relic foundations, apparently associated with the past lumber mill, as well as a concrete slab-on-grade and a pile of large concrete fragments, were observed within the previously graded area. Cut slopes on the east side of the graded area were up to 30 feet in height, and slope gradients ranged from approximately 1:1, horizontal to vertical (H:V), to near vertical. Significant residual rock structure was observed in the soil exposed in the cut slope faces. Elevations in this area ranged from approximately 2590 feet above MSL on the graded area at the toe of the cut slope to approximately 2730 feet above MSL near the eastern site boundary. Slope gradients were generally less than 10 percent, excluding the natural slope, the cut slope, and a relatively steep fill slope located on the southern end of the historic mill area.

2.5.1.2 New Brunswick Site

The New Brunswick site sits in a valley created by the South Fork of Wolf Creek. The site is bounded by Bennett Road to the north, a pond and associated dam to the east, and a steep slope (60%) to the south. Elevations across the site ranged from 2540 feet above MSL at the western site boundary to roughly 2750 feet above MSL around the New Brunswick Mine area. The site consisted of the generally flat lying surfaces around the New Brunswick Mine, gently sloping open fields and tree covered areas extending downstream of the dam, and steep slopes along the southern part of the site.

Deep fill was apparent in the vicinity of the New Brunswick Mine workings. We also observed the mine silo, concrete slabs-on-grade, and the covered New Brunswick shaft in this area.

The gently sloping surfaces along the valley floor were covered with thick vegetation and we could not evaluate the nature of the material in this area.

We observed concrete walls and waste rock piles associated with the Union Hill shaft in the northwestern part of the site. We also observed numerous waste rock piles on the northeast facing slopes across from the Union Hill shaft. These piles were up to 10 feet in height and were likely associated with mining from the Cambridge shaft and nearby exploration.

2.5.1.3 Roundhole Easement Site

The Roundhole Easement site lies on the slope immediately north of Whispering Pines Lane. The proposed site consists of a 300-foot long access road and 300-foot diameter circular area according the site plan. We observed a north facing 25 percent slope along the access road and a shallower northeast facing 15 percent slope in the circular area. Elevations across the site ranged from 2705 feet above MSL at the top of the access road to 2640 feet above MSL at the lowest part of the site. We observed the remains of a concrete structure and waste rocks piles in the northern part of the site. These features were likely associated with the Roundhole shaft.

2.5.2 Surface and Groundwater Conditions

We did not observe standing water at either the Idaho-Maryland site or the Roundhole Easement site. However, the ground surface of the flat lying portions of both sites were saturated from recent rain.

The South Fork of Wolf Creek trends northwest through the center of the New Brunswick site. The low-lying areas downstream of the dam were covered with marsh vegetation, but we did not observe standing water in these areas at the time of our site visit, which was performed at the end of the dry season. However, we observed flowing water in the South Fork of Wolf Creek.

3 LABORATORY TESTING

Laboratory testing was not included in the scope of our preliminary geotechnical engineering investigation. Laboratory testing would be required as part of a design-level geotechnical engineering investigation for the project.

4 CONCLUSIONS

The following conclusions are based on our field observations and our experience in the area.

- Based on the results of our preliminary geotechnical investigation, our opinion is that the project is feasible from a geotechnical standpoint. The recommendations contained in this report are preliminary in nature and should not be used for construction.

- Based on our review of geologic maps pertaining to the subject sites, we do not anticipate that naturally occurring asbestiform minerals will be encountered in the native soil/rock encountered in the majority of the three sites. However, the western edge of the Idaho-Maryland site was mapped as serpentine and may contain natural asbestiform minerals. In addition, material that may have been imported to the sites may contain asbestiform minerals, although we did not encounter evidence of asbestiform minerals during our site reconnaissance. The State of California Environmental Protection Agency and Air Resources Board have recognized asbestos as a carcinogen. Grading in areas of fibrous serpentinite rock typically requires an asbestos dust mitigation plan. The plan would address engineering controls, air monitoring, laboratory testing, special handling and input from local regulatory agencies.

- Our primary concern, from a geotechnical standpoint, is the presence of relic mine features at the three sites and the presence of fill in portions of the previously graded areas of the Idaho-Maryland site and New Brunswick site. We observed and performed field density testing during fill placement in the area of the New Brunswick shaft, as summarized in our letter dated February 5, 1997. Much of the fill encountered during a previous subsurface investigation performed by others at the Idaho-Maryland site reportedly contained organic material that would not be suitable to support structural improvements. We anticipate that the relatively shallow fill across much of the southern area would be able to be removed or, if deemed suitable for the purpose, used for compacted fill. However, the deeper fill encountered by others in the southeastern area would likely require extensive excavation and would not likely be able to be reused due the reported abundance of organic materials.

- The disturbed material and waste rock identified in the northwestern part of the Idaho-Maryland site and at the New Brunswick site may not be suitable to support structural improvements.

- Waste rock piles cover portions of the Idaho-Maryland, New Brunswick, and Roundhole Easement sites. In general, these piles are not suitable to support structural improvements. The waste rock piles in the area of the proposed ceramic plant would likely have to be removed prior to construction.

- The most notable historic mining features documented on the site were the New Brunswick shaft; the Roundhole shaft; the South Idaho shaft; and a horizontal tunnel that extends east along the South Idaho vein in the southeastern part of the site. If improvements are planned in the immediate recorded mining features, the features should be identified, if possible, and closed per the recommendations of H&K or another qualified engineer. We would be able to provide closure recommendations as part of a design-level geotechnical engineering report.

- Based on our experience in the area, relatively shallow, resistant rock may be encountered in portions of the site during grading or excavation for utilities. Preliminary recommendations for resistant rock are presented in the following section. Subsurface soil and existing fill may also contain significant oversized rock and other large material that would require specific recommendations for use as fill. General recommendations for placement of oversized rock are also presented in the following section.

- Based on our experience in the area and our review of laboratory test results prepared by others, we anticipate that potentially expansive clay soil may be encountered in some portions of the site above relatively shallow, weathered rock. Expansive clay soil is typically encountered in this area in thin layers that require relatively modest design modification. General recommendations pertaining to expansive soil are presented in the following section.

- If the proposed improvements are to be located immediately above or below the relatively high cut slopes on the southeastern area of the Idaho-Maryland site, we anticipate that the slopes would require further evaluation.

- Other mine features may be present on or extending beneath the subject properties which were not identified during this preliminary investigation.

5 PRELIMINARY RECOMMENDATIONS

The following preliminary geotechnical engineering recommendations are based on our understanding of the project as currently proposed, our field observations, and our experience in the area. The recommendations are preliminary and should be verified by a design-level geotechnical engineering investigation.

5.1 GRADING

5.1.1 Clearing and Grubbing

Subgrade for fill placement, paved areas, and building pads should be cleared and grubbed of vegetation and other deleterious materials as described below.

1. Strip and remove organic surface soil (typically 0 to 2 inches in undisturbed areas) containing shallow vegetation and any other deleterious materials. Topsoil can be stockpiled onsite and used in landscape areas, but is not suitable for use as fill. The actual depth of stripping may vary across the site. We anticipate that deeper fill with organics will be encountered in portions of the previously graded areas.

2. Overexcavate loose fill, debris and/or other onsite excavations to underlying, competent material. Possible excavations include exploratory trenches excavated by others, mantles or soil test pits, mining features, and tree stump holes.

3. Remove all rocks greater than 8 inches in greatest dimension (oversized rock) from the top 12 inches of soil. Oversized rock should be placed in deep fill per the recommendations of the project geotechnical engineer, stockpiled for later use in landscape areas or stacked rock walls, or removed from the site.

4. Vegetation, tree stumps and exposed root systems, any other deleterious materials and oversized rocks not used in landscape areas should be removed from the site.

5.1.2 Preparation for Fill Placement

Upon completion of site clearing, grubbing and overexcavation, the exposed native soil should be observed by a representative of our firm prior to placement of fill at the project site. Fill placed on slopes steeper than 5:1, horizontal:vertical (H:V), should be benched into the existing slope to allow placement of fill in horizontal lifts.

5.1.3 Fill Placement

Fill should be placed according to the following guidelines:

1. Material used for fill construction should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. Rock used in fill should be no larger than 8 inches in diameter. Rocks larger than 8 inches are considered oversized material and should be placed in deep fill per the recommendations of the project geotechnical engineer, stockpiled for use in landscape areas or rock walls, or removed from the site.

2. Oversized material may be windrowed in deeper fill under the observation of a representative of the project geotechnical engineer. The windrows should be separated by at least one equipment width. Compacted fill should be worked into the sides of each windrow, and remaining voids should be filled with smaller rock. If the oversized material is to be incorporated into a rock fill that does not permit density testing by nuclear methods, the contractor should prepare a test fill during initial fill placement for observation and testing. The means and methods of subsequent fill placement will be evaluated for conformance with the approved test fill. Subsurface seepage should be addressed in areas of oversized rock placement and rock fill to reduce the chance of soil migration in the fill associated with groundwater seepage through the oversized material or rock fill.

3. Imported fill material should be predominantly granular, non-expansive and free of deleterious or organic material. If imported material is required to grade the site, it should be submitted to H&K for approval and laboratory analysis at least 72 hours prior to use as fill.

4. Clay soil, if encountered, may be used as fill if mixed with granular soil at a ratio determined by the project geotechnical engineer. A typical mixing ratio for granular soil to clay soil is four to one.

5. Fill should be uniformly moisture conditioned and placed in maximum 8-inch thick loose lifts (layers) prior to compacting.

6. All fill should be compacted to at least 90 percent of the maximum dry density per ASTM D1557. The upper 8 inches of fill in building footprints and paved areas should be compacted to a minimum of 95 percent of the maximum dry density per ASTM D1557.

7. The moisture content, density and relative compaction of all fill should be evaluated by our firm during construction.

5.1.4 Differential Fill Depth

To reduce the magnitude of differential settlement associated with variable fill depth beneath structures, we recommend that differential fill depths beneath structures should not exceed 5 feet. For example, if the maximum fill depth is 8 feet across a building pad, the minimum fill depth beneath that pad should not be less than 3 feet. If a cut-fill building pad is used in this example, the cut portion would need to be overexcavated 3 feet and replaced with compacted fill. As part of a design-level geotechnical investigation, we would be able to provide additional recommendations to reduce differential settlement for structures, such as the proposed ceramics plant, which are to be located in moderately sloping portions of the site.

5.1.5 Cut/Fill Slope Grading

1. Cut and fill slopes should generally be no steeper than 2:1, H:V. Based on our experience in the area, 1½:1, H:V, or steeper cut slope gradients may be possible in some areas that have significant rock structure. Allowable slope gradients must be verified based on the results of laboratory testing performed as part of a design-level geotechnical investigation.

2. Fill slopes should be constructed by overbuilding the slope face and then cutting it back to the design slope gradient. Fill slopes should not be constructed or extended horizontally by placing soil on an existing slope face and/or compacted by track walking.

3. Benching during placement of fill on an existing slope must extend through loose surface soil into firm material, and be performed at intervals such that

no loose soil is left beneath the fill. An equipment width bench should be made at least every 5 vertical feet.

4. Our observation of rock outcrop and our experience in the area has shown that isolated areas of moderately or slightly weathered rock that is difficult to trench with conventional trenching equipment may be encountered in some portions of the site during grading or trenching. Pre-ripping, blasting, or splitting may be required in these isolated areas.

5.1.6 Erosion Control

Graded portions of the site should be seeded as soon as possible following grading to allow vegetation to become established prior to the rainy season. The following erosion control measures should be implemented for cut and fill slopes to reduce erosion.

1. Slopes should be hydroseeded or hand seeded/strawed with an appropriate seed mixture compatible with the soil and climate conditions of the site as recommended by the local Resource Conservation District office.

2. Following seeding, jute netting should be placed and secured over the slopes to keep seeds and straw from being washed or blown away. Tackifiers or binding agents may be used in lieu of jute netting. Surface water drainage ditches should be established at the top of all graded slopes to intercept and redirect surface water away from the slope face.

3. Under no circumstances should surface water be allowed to run over slope faces. The intercepted water should be discharged into natural drainage courses or into the on site storm water drainage system.

5.1.7 Subsurface Drainage

If grading is performed during or immediately following the rainy season, seepage may be encountered, particularly in the low-lying portions of the New Brunswick and Idaho-Maryland sites. If groundwater or saturated soil conditions are encountered during grading, we anticipate that dewatering may be possible by gravity or by installation of sump pumps in the excavation. Control of subsurface seepage at the base of fill areas can typically be accomplished by placement of an area drain or a strip drain. Underlying, saturated soil is typically removed and replaced with free draining, granular drain rock enveloped in geotextile fabric. Fill soil can be placed after placing the granular rock to an elevation that is higher than the encountered groundwater. Rock drains typically consist of open graded rock enveloped in a non-woven geotextile filter fabric such as Amoco 4546™ or equivalent. Drains should have a minimum 4-inch diameter, perforated, schedule 40, PVC pipe placed at the low point of the drain, inside the drainrock, with the perforations placed down. The PVC pipe should be sloped so that water is directed away from the fill placement area by gravity. Site specific subsurface drainage recommendations can be provided as part of a design-level geotechnical report.

5.1.8 Surface Water Drainage

Proper surface water drainage is important to the successful development of the project. We recommend the following measures to help mitigate surface water drainage problems:

1. Slope final grade in structural areas so that surface water drains away from buildings at a minimum 2 percent slope for a minimum distance of 10 feet.

2. Compact and slope all soil placed adjacent to building foundations such that water is not allowed to pond or infiltrate. Backfill should be free of deleterious material.

3. Direct downspouts to a closed collector pipe which discharges flow to positive drainage.

5.1.9 Construction Monitoring

Construction monitoring includes review of plans and specifications and observation of onsite activities during construction as described below.

1. We should be allowed to review the final grading plans prior to construction to determine whether recommendations presented in the design-level

geotechnical report have been implemented, and if necessary, to provide additional and/or modified recommendations.

2. We should be allowed to perform construction monitoring of earthwork grading performed by the contractor to determine whether our recommendations have been implemented, and if necessary, provide additional and/or modified recommendations.

5.2 FOUNDATION SYSTEMS

Our preliminary opinion is that shallow spread footings are suitable for support of structures across much of the subject site. Footings should be founded on native, undisturbed soil/rock or compacted, tested fill. Foundation design criteria and construction recommendations are typically provided as part of a design-level geotechnical engineering report.

If adverse subsurface conditions such as loose fill or expansive soil are encountered, such as the deeper fill documented in the eastern side of the Idaho-Maryland site, a deep foundation or removal and replacement of the fill may be required. Based on the larger material encountered by others in the deep fill, we do not anticipate that drilled piers would be appropriate at that particular location. We understand that improvement of this area is not currently proposed.

Footings should be deepened through expansive clay soil, if encountered at the base of the footing excavations. Expansive clay soil is occasionally encountered in relatively thin layers above the weathered rock in this area.

6 LIMITATIONS

The following limitations apply to the findings, conclusions and recommendations presented in this report:

1. Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in northern California. This warranty is in lieu of all other warranties, either expressed or implied.

2. These services were performed consistent with our agreement with our client. We are not responsible for the impacts of any changes in environmental standards, practices or regulations subsequent to performance of our services. We do not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of our client. Any reliance on this report by a third party is at the risk of that party.

3. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report should be considered invalid by all parties. Only our firm can determine the validity of the conclusions and recommendations presented in this report. Therefore, we should be allowed to review all project changes and prepare written responses with regards to their impacts on our conclusions and recommendations. Subsurface investigation and laboratory testing will be required to develop design-level recommendations.

4. The analyses, conclusions and recommendations presented in this report are preliminary, based on site conditions as they existed at the time we performed our surface observations. The subsurface conditions should be confirmed by a design-level geotechnical investigation prior to construction.

5. Our scope of services did not include evaluating the project site for the presence of hazardous materials. Waste rock associated with historic mining has the potential to contain elevated metals concentrations which may pose a hazard to human health and water quality. Although we did not identify hazardous materials at the time of our field investigation, we understand that petroleum products have been released at the subject site. Project personnel should be careful and take the necessary precautions should hazardous materials be encountered during construction.

6. The findings of this report are valid as of the present date. Changes in the conditions of the property can occur with the passage of time. The changes may be due to natural processes or to the works of man, on the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or the broadening of knowledge. Therefore, the recommendations presented in this report should not be relied upon after a period of two years from the issue date without our review.

SHEETS

Sheet 1 Approximate Site Map

APPENDIX ***IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT***

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