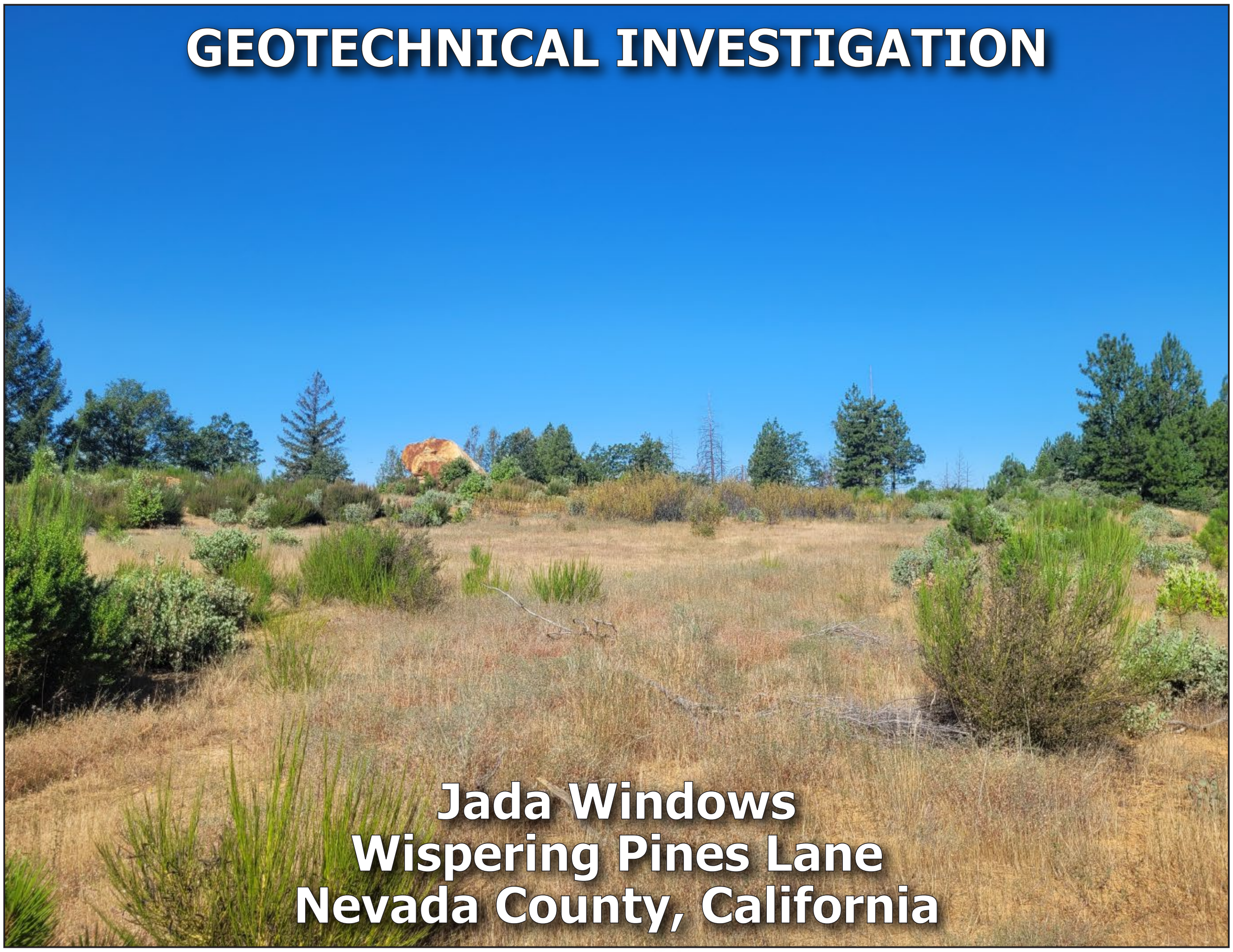
GEOTECHNICAL INVESTIGATION



Jada Windows
Wispering Pines Lane
Nevada County, California

PREPARED FOR:

JADA, INC.
124 CLYDESDALE COURT
GRASS VALLEY, CALIFORNIA 95945

PREPARED BY:

GEOCON CONSULTANTS, INC.
3160 GOLD VALLEY DRIVE, SUITE 800
RANCHO CORDOVA, CALIFORNIA 95742

GEOCON PROJECT NO. S2851-05-02

DECEMBER 2024

Project No. S2851-05-02
December 3, 2024

Jada, Inc.
124 Clydesdale Court
Grass Valley, California 95945

Attention: Aaron Beyer, General Manager

Subject: GEOTECHNICAL INVESTIGATION
JADA WINDOWS
GRASS VALLEY, NEVADA COUNTY, CALIFORNIA

Mr. Beyer:

Geocon Consultants, Inc. is pleased to provide Jada, Inc. with this Geotechnical Investigation report in support of the proposed Jada Windows development to be located on Nevada County assessor's parcel numbers 009-680-050 and -056 (the site) south of Whispering Pines Lane in Grass Valley, Nevada County, California.

This report presents the methodology and findings of our geotechnical investigation, which was performed in general accordance with our proposal dated August 27, 2024, and the professional services agreement dated July 17, 2024. The findings presented in this report are based on our literature review, subsurface trenching investigation, laboratory testing, engineering evaluation, and our experience with soil and rock conditions in the area. Our opinion is that the project is feasible from a geotechnical engineering standpoint provided that the recommendations presented in this report are incorporated into the project design and followed during construction.

Our primary concern, from a geotechnical engineering standpoint, is the presence of resistant bedrock outcrop and shallow resistant bedrock in the central portion of the site, which may adversely impact grading and excavation operations for foundations and utilities. In addition, we identified two locations of undocumented fill and one area of expansive clay soil that should be removed and replaced in accordance with the grading recommendations presented herein. We recommend that we be retained to observe and test existing fill exposed during site earthwork/grading because it appears that existing undocumented fill and expansive clay soil on the site prior to the 2013 earthwork/grading at the site was not sufficiently removed or otherwise remediated.

We appreciate the opportunity to assist Jada, Inc., with this important project. Please contact the undersigned with any comments or questions regarding the conclusions and recommendations presented in this report.

Sincerely,

GEOCON CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
JASON W. MUIR
No. 2697
Exp. 06/26
GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JEREMY J. ZORNE
No. 2636
GEOTECHNICAL
STATE OF CALIFORNIA

Jason Muir, PE, GE
Senior Engineer

Jeremy Zorne, PE, GE
Senior Engineer

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION PAGE

GEOTECHNICAL INVESTIGATION

1.0 INTRODUCTION

Geocon Consultants Inc. (Geocon) prepared this report on behalf of Jada, Inc. (the Client) to "present the methodology and findings of our design-level geotechnical investigation in support of the proposed Jada Windows development to be located on Nevada County assessor's parcel numbers (APN) 009-680-050 and -056 (the site) south of Whispering Pines Lane in Grass Valley, Nevada County, California. We performed the investigation in general accordance with our proposal dated August 27, 2024, and the professional services agreement dated July 17, 2024.

1.1 Purpose

The purpose of our design-level geotechnical investigation was to evaluate subsurface conditions at the site. This report presents the methodology and findings of our geotechnical investigation. The findings are based on our literature review, subsurface trenching investigation, laboratory testing, engineering evaluation, and our experience with soil and rock conditions in the area.

1.2 Scope of Investigation

The geotechnical investigation included:

1. Literature review:
    a. review of previous geotechnical investigation reports,
    b. review of field reports prepared during previous site grading,
    c. review of local topographic, geologic, and soil survey publications, and
    d. review of historical maps and aerial photographs,
2. Field exploration:
    a. excavation of 10 exploratory trenches (T24-01 through T24-10) to depths up to 8 feet by using a rubber-tire Caterpillar 420D backhoe;
    b. observation and logging of the subsurface conditions encountered in the exploratory trenches;
    c. collection of relatively undisturbed soil samples and bulk soil samples;
3. Geotechnical laboratory testing to evaluate soil engineering properties;
4. Engineering analysis to develop geotechnical design criteria; and
5. Preparation of this report.

1.3 Report Organization

This report is organized as follows.

- *Section 1.0* presents the investigation purpose, scope, and guiding principles.
- *Section 2.0* describes the site and its previous investigation.
- *Section 3.0* describes the field exploration.
- *Section 4.0* describes the laboratory testing program.
- *Section 5.0* describes the site seismicity and identified geologic hazards.
- *Section 6.0* presents our conclusions and recommendations.
- *Section 7.0* recommends plan review and construction observation.
- *Section 8.0* describes the general limitations of the geotechnical investigation.
- *Section 9.0* lists the references cited in this report.

1.4 Guiding Principles

The following principles are an integral part of the geotechnical engineering practice.

Verification of Exploratory Data

This report is based on information collected from specific exploratory locations. Actual subsurface conditions may differ, sometimes greatly, between and beyond the exploratory locations. The findings and recommendations presented in this report are professional opinions based on the data collected during the investigation and are not scientific certainties. We should be retained to perform observation and testing during project development to confirm that the conditions encountered during construction are similar to those encountered at the exploratory locations. We cannot assume responsibility or liability for the recommendations presented in this report if we do not perform construction observation.

Incorporation of Geotechnical Recommendations

We should be retained to review the project plans and specifications to verify that the geotechnical recommendations presented in this report were appropriately incorporated into the project design documents. Only we can determine whether the project design complies with the recommendations presented in this report.

Conditions Can Change

This report is based on conditions that were present at the time the investigation was performed. Conditions can change due to the passage of time, the acts of man (e.g., grading on or adjacent to the site), or natural forces (e.g., flooding, earthquakes, groundwater fluctuations, climate change). Only we can determine whether the recommendations presented in this report remain applicable to changes in site conditions.

Entire Report

Portions of this report should not be reproduced separately for the purpose of bid preparation. The report should be provided in its entirety and should be prefaced with a clearly written letter of transmittal advising contractors that the report was not prepared for the purposes of bid development and that the report's accuracy is limited. Contractors should be encouraged to confer with us or conduct additional study to obtain the specific types of information the contractor requires or prefers.

Geoenvironmental Concerns Are Addressed Separately

This report does not address the potential for hazardous materials. Geoenvironmental conditions are addressed separately in our Preliminary Endangerment Assessment (PEA) Report, which is required by the City of Grass Valley and is to be issued in December 2024.

2.0 BACKGROUND

This section describes the site and its physical setting, previous site grading, proposed improvements, and previous assessment. Photographs of site features described herein (Photos 1 through 28) are presented in Appendix A.

2.1 Site Description

The 7.74-acre light industrial (zoning designation M-1) site is in Grass Valley, Nevada County, California. Grass Valley is approximately 50 miles northeast of Sacramento, near the junction of State Routes 49 and 20 (Figure 1). The site is immediately south of Whispering Pines Lane and approximately 200 feet west of its intersection with Clydesdale Court (Figure 2). The site includes Nevada County APNs 009-680-050 and -056. According to a parcel report accessed via Nevada County Geographic Information Systems (GIS; https://www.nevadacountyca.gov/560/Geographic-Information-Systems) the site was previously referenced by APNs 09-680-49, -50, and -53 until a merger and boundary line adjustment in 2007.

2.2 Physical Setting

Referencing the United States Bureau of Land Management (BLM) National Public Land Survey System, the site is primarily in the southwest ¼ of the northwest ¼ of Section 25, Township 16 North, Range 8 East based on the Mount Diablo geodetic datum. The western end of the site, outside of the proposed development area, extends into Section 26.

2.2.1 Topography

The site is on a narrow tabular ridgetop that extends west from the vicinity of Whispering Pines Lane (Photo 1). According to the *Preliminary Site Plan for Jada Windows* (Appendix B; Nelson Engineering, 2024), site elevations range from approximately 2,580 feet above mean sea level (MSL) near its western end, outside of the proposed development area, to approximately 2,650 feet above MSL near its northeastern corner at Whispering Pines Lane. Adjacent land to the north, west, and south is lower in elevation.

2.2.2 Previous Site Grading

Earthwork cut and fill associated with previous site grading lowered the elevation of central portion of the site up to approximately 15 feet, and the current elevation is approximately 2,640 feet above MSL. The central portion of the development area is in cut native soil and weathered diabase rock, although resistant bedrock outcroppings extend up to approximately 10 feet above the current grade (Photos 2 through 7).

Soil cut from the central portion of the site was used to construct engineered fill (Figure 2):

- engineered fill on the southeastern end of the proposed development area has an estimated maximum thickness of 20 feet;
- engineered fill on the northern edge of the proposed development area has an estimated maximum fill thickness of approximately 15 feet; and
- engineered fill on the southwestern edge of the proposed development area has an estimated maximum fill thickness of approximately 5 feet.

Holdrege & Kull Consulting Engineers and Geologists (H&K) performed observation and testing during the previous site grading performed by C&D Contractors, Inc. in 2013. H&K prepared 10 geotechnical field reports from October 23 through November 2, 2013, summarizing the grading operations and presenting the results of field density testing by nuclear gauge using ASTM International (ASTM) Method D6938. H&K reported relative compaction results of 90% or greater for each of the 91 field density tests performed at the site based on ASTM Method D1557 (Modified Proctor). H&K obtained bulk soil samples for laboratory compaction curve testing, observed keyway preparation prior to fill placement, and observed the removal of clay soil from the base of a keyway to competent underlying native materials. H&K reported that soil and rock excavation was performed using Caterpillar 326 and 320 excavators; soil transport by ten-wheel dump trucks; moisture conditioning by Caterpillar D6R dozer and water truck; root removal by hand; engineered fill placement by Caterpillar D6R dozer; and compaction by Caterpillar 815F and CP563C compactors.

Boulder piles resulting from the previous site grading are near the southeast and northwest site boundaries (Figure 2; Photos 8 and 9). The boulders are typically up to approximately five feet in greatest dimension.

However, we observed untested, undocumented fill at the location of exploratory trench T24-09, as described in Section 3.1 and the exploratory logs (Appendix C). It appears that this undocumented fill was not removed or otherwise remediated during the 2013 earthwork/grading of the site. The undocumented fill is not considered suitable for support of the proposed improvements and should be removed and replaced in accordance with the recommendations presented in Section 6.0. We also observed relatively loose soil in the upper 18 inches of fill at the location of our exploratory trench T24-06 that should also be removed and replaced in accordance with the recommendations presented in Section 6.0.

A soil stockpile, likely resulting from previous site grading, is in the northeastern portion of the site near the site entrance (Figure 2, exploratory trench T24-08). The stockpile surface is approximately three feet above the surrounding ground surface. Dense vegetation is growing on the stockpile. This stockpile may be used for deep, engineered roadway fill near the southeastern corner of the site provided that unsuitable materials (e.g., organics, roots, etc.) are removed and its use is approved in the field by Geocon.

2.2.3 Regional Physiographic Conditions

The site is in the Sierra Nevada Foothills on the western slope of the Sierra Nevada geomorphic province. The Sierra Nevada province is an elongate, northwest-trending structural block that is tilted upward to form a steep scarp above the adjacent Basin and Range province to the east. The western slope of the Sierra Nevada dips gently westward and extends beneath sediment of the Great Valley province. Uplift and erosion of the Sierra Nevada contribute to sediment within the Great Valley.

The western foothills of the Sierra Nevada are a complex assemblage of igneous and metamorphic rocks. The regional structure of the foothills is characterized by the north-northwest trending Foothills Fault System, a feature formed during the Mesozoic era (dating from 65 to 230 million years ago) in a compressional tectonic environment. A change to an extensional tectonic environment during the Late Cenozoic (last 9 million years) resulted in normal faulting which has occurred coincident with some segments of the older faults.

2.2.4 Geology

The *Geologic Map of the Chico Quadrangle* (Saucedo and Wagner, 1981) and the *Geologic Map of the Grass Valley-Colfax Area* (Tuminas, 1983) depict the site as being underlain primarily by diabase (microgabbro), mafic intrusive rock associated with the Mesozoic (252 to 66 million years ago) Lake Combie complex. Tuminas (1983) maps Lake Combie serpentinite (ultramafic rock) extending approximately 170 feet into the site from its northeastern corner (Figure 2). An Asbestos Dust Mitigation Plan (ADMP) is presented under separate cover.

2.2.5 Local Mining History

Historical aerial photographs (USGS, 1939, 1947, and 1952) and historical topographic maps (USGS, 1901, 1939, and 1950) do not depict evidence of mining at the site. Other than a small water reservoir, a structure, and several small exploratory excavations mapped at the site prior to 1950, the site does not appear to have been developed.

A vertical mine shaft is mapped near the western end of the site, downslope and west of the proposed development area (Figure 2). Historical mining maps indicate that the feature is an air shaft (ventilation shaft) associated with deep underground workings of the Idaho-Maryland Mine. Historical operations of the Idaho-Maryland mine, including ore extraction from the underground workings, ore processing, and tailings disposal to land, were located downslope and to the west of the site.

At the historically mapped location of the vertical mine shaft, we observed a shallow depression (Photo 10). We did not observe evidence of a shaft portal, concrete structures, or mine waste at the historically recorded shaft location. The absence of waste rock is likely because the shaft was excavated upward from the underground tunnel, and waste rock from the shaft excavation was likely carried down to tunnel level and out through the Idaho Main Shaft northwest of the site.

Appendix D presents excerpts of aerial photographs, topographic maps, and mining maps of the site and vicinity. The maps and images are described below.

<u>Aerial Photographs</u>

A recent aerial photograph (Google Earth, August 12, 2024) depicts the vacant site and surrounding land uses, including Ferguson Plumbing Supply downslope to the west, Mountain Enterprises downslope to the south, and Peaceful Valley Farm & Garden Supply upslope to the east. The current Jada Windows manufacturing plant is southeast of and cross-gradient to the site.

The 1939 aerial photograph, procured by the Unted States Geological Survey (USGS), indicates that the site is generally vegetated and vacant aside from an unpaved road crossing through the northern portion of the site from present-day Whispering Pines Lane. A small clearing in the central-western portion of the site is likely associated with a small water reservoir shown on the 1901 topographic map (see below). The Idaho-Maryland mine is visible west of the site, including the Idaho Main Shaft headframe downslope to the northwest, an ore processing facility downslope to the west at the present-day Ferguson Plumbing Supply, and a tailings pond downslope to the west of the ore processing facility. The 1947 and 1952 aerial photographs (USGS) depict similar conditions. A log pond associated with the historical Lausman Lumber Mill (not associated with the gold mining operations) is downslope to the south-southwest of the site. The Lausman Lumber Mill is visible at the southern edge of the 1947 and 1952 images.

Topographic Maps

A 7.5-minute topographic map of *Grass Valley, California* (USGS, 1993) depicts the site as a narrow tabular ridgetop above the nearby historical gold mining and lumber milling operations. The Idaho-Maryland Ditch, an historical water conveyance canal, previously crossed the eastern portion of the site from north to south, providing water to the log pond southwest of the site.

The 1901 and 1939 topographic maps (USGS) depict a 0.15-acre water reservoir and a structure in the central-western portion of the site. The reservoir and structure are not depicted on the 1950 topographic map. Site grading in 2013 lowered the elevation of this portion of the site by approximately 15 feet.

The *Geologic Map of the Grass Valley Quadrangle and Adjacent Area* (Johnston, 1939) maps a historical vertical mine shaft near the western end of the site and maps an underground incline extending at depth from the vertical mine shaft to the southeast. Johnston's *The Gold Quartz Veins of Grass Valley, California* (Geological Survey Professional Paper 194; 1940) indicates that the vertical shaft is associated with the historical workings of the Idaho-Maryland Mine, and the underground incline is the Canyon (or Cañon) winze, which rakes to the east from the 1,000-foot level as far as the 1,900-foot level (commonly measured along the underground incline). As mentioned earlier in this section, we did not observe mine waste at the mapped location of the vertical shaft. This is likely because the shaft was excavated upward from the tunnel level, and waste rock from the shaft excavation was likely caried down to tunnel level and out through the Idaho Main Shaft northwest of the site.

The portal of the Idaho Main Shaft was located approximately 600 feet northwest of the site, near Idaho Maryland Road, and the shaft inclined to the 1,000-foot level at an approximate angle of 70 degrees. A tunnel at the 1,000-foot level extended from the Idaho Main Shaft to the Canyon winze, and the vertical shaft mapped near the western end of the site extended upward from this intersection.

Historical Mining Maps

Plate II of the *Plan of Underground Workings of Idaho-Maryland Development Co., Grass Valley, California* (Adams, W.J., undated) does not depict a vertical shaft extending to the ground surface, but depicts tunnels extending from the inclined winze at depth beneath the site at the 1,000-foot, 1,100-foot, 1,200-foot, 1,300-foot, and 1,400-foot levels (as measured in feet along the inclined winze rather than vertically). A tunnel also extends from the Idaho Main Shaft beneath the northern edge of the site at the 700-foot level.

The *Composite Map of Idaho-Maryland Mine to 2000 Level* (anonymous, 1950) depicts subsurface mining features similar to those depicted by Adams.

The map of near-surface workings labeled “Surface, L-50” (Idaho Maryland Mines Corporation, undated) depicts the water reservoir in the central-western portion of the site and several apparent exploratory prospects, one of which is labeled “Webster Shaft.” The remnants of the reservoir and two circular depressions were observed during a geotechnical investigation performed by H&K in 2010. The depressions were 8 to 10 feet in diameter and 6 to 7 feet deep. Site grading subsequently lowered the elevation of this area by approximately 15 feet. We excavated exploratory trenches T24-01 and T24-02 at the recorded locations of the depressions, and we encountered undisturbed native soil and variably weathered bedrock.

The *Supplemental Master Title Plat for Sections 25, 26, 35, and 36, Township 16 North, Range 8 East* (BLM, 1996) depicts a portion of the historical Schofield Gold Quartz Claim (Mineral Survey Plat No. 30) in the northeastern portion of the site. Mineral Survey Plat No. 30 (United States Surveyor General’s Office [USSGO], September 1867) depicts a 50-foot-deep shaft north of the site under the present-day Whispering Pines Road. The plat does not depict mining features at the site.

The *Plat of the Claim upon the South Idaho Consolidated Quartz Mine* (Mineral Survey Plat 2781, USSGO, 1888) references the site as “P.B. Mocks Agricultural Claim.” No mining features are depicted at the site.

The *Plat of the Claim of the Idaho Maryland Mines Company* (Mineral Survey Plat 5514-1, USSGO, 1921) references the site as “Phillip B. Mock Agricultural Patent.” No mining features are depicted at the site, although a deep underground incline is depicted as crossing beneath the southern portion of the site.

2.2.6 Regional Faulting

The online Fault Activity Map of California (CGS, 2024) depicts a segment of the Grass Valley Fault near the site, near the eastern edge of 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.

CGS (2024) indicates that the northwest-to-southeast-trending Grass Valley Fault is pre-Quaternary (older than 1.6 million years without recognized Quaternary displacement). The late Quaternary Wolf Creek Fault and Giant Gap fault (fault displacement during the past 700,000 years) are mapped approximately 6 miles south of the site and 12 miles east of the site, respectively. The nearest mapped faults with evidence of Holocene displacement (during the past 11,700 years) are near Oroville (30 miles northwest) and Truckee (45 miles east).

Special Publication 42 (CGS, 2018) is intended to promote uniform and effective statewide implementation of the evaluation and mitigation elements of the Alquist-Priolo Earthquake Fault Zoning Act. Pursuant to CGS (2018) guidance, we used the online California Earthquake Hazards Zone Application (EQZ App; https://maps.conservation.ca.gov/cgs/EQZApp/) to determine whether the site is located within a designated Earthquake Fault Zone (also known as Alquist-Priolo Zone, or A-P Zone). A-P Zones are regulatory zones that encompass traces of Holocene-active faults to address hazards associated with surface fault rupture. The site is not mapped within an A-P Zone.

2.2.7 Soil

The United States Department of Agriculture (USDA) *Web Soil Survey* application (https://websoilsurvey.nrcs.usda.gov/app/) characterizes site soil predominantly as “Sites very stony loam.” Up to approximately 15 feet of the soil profile was removed from the central portion of the development area during site grading/earthwork in 2013.

The soil survey describes the soil at the site as medium to high acid soil that may be highly corrosive to concrete and uncoated steel. A typical profile is described as heavy loam from 0 to 1 foot, underlain by clay loam, clay, and light clay loam from 1 to 6.5 feet. Variably weathered metasedimentary and basic rock is commonly encountered at depths greater than 6.5 feet. Up to one quarter of the soil profile is described as cobbles. Runoff is described as medium with a slight to moderate erosion hazard.

The Web Soil Survey maps alluvial land at the eastern end of the site. We observed alluvial conditions in a low, densely vegetated area near the southeastern site corner. Clayey soil associated with alluvium, as well as seasonal surface water and shallow groundwater seepage, may be present in the lower, southeast corner of the site, where a road is to be constructed that extends off-site to the south-southeast.

Prior to the earthwork grading performed in 2013, H&K (2010) excavated 10 exploratory trenches (T-1 through T-10) at the site to depths up to 9 feet. Exploratory trench locations are depicted on Figure 2, and soil conditions encountered in the exploratory trenches are described below.

- Trenches T-1 and T-2 were advanced in the northeastern portion of the site and revealed fill generally consisting of silty sand with gravel, angular cobble-sized rock fragments, and small boulders to depths up to 7 feet. The fill was generally medium-dense and dry. The fill in trench T-1 was underlain by severely weathered rock, and in T-2 by sandy fat clay.
- Trenches T-3 and T-4 were advanced in the southeastern portion of site and encountered damp, clayey, native soil underlain by variably weathered rock. Trench T-3 encountered refusal on resistant rock at approximately 4.6 feet. Groundwater seepage was encountered in T-3 at 3.9 feet.
- Trench T-5 and T-10 were advanced in the north-central portion of the site and encountered silty sand with gravel, underlain at 1 to 2.5 feet by weathered bedrock. The bedrock encountered in T10 was resistant to excavation at a depth of 3 feet.
- Trenches T-6 and T-7 were advanced at a hilltop that was subsequently removed, except for large bedrock outcroppings.
- Trenches T-8 and T-9 were advanced in the western portion of the proposed development area and exposed silt with fine sand underlain at 4 feet by weathered rock.

2.2.8 Surface Water

No surface water bodies are present at the site. USGS topographic maps from 1949 to 1993 (Appendix D) depict the historical Idaho-Maryland Ditch, a former water conveyance ditch, crossing the east side of the site from north to south. The ditch is no longer present as a result of previous earthwork grading at the site. The *Geologic Map of the Grass Valley Quadrangle and Adjacent Area* (Johnston, 1939) maps a historical, 0.15-acre, rectangular water reservoir near the site center. The reservoir is no longer present as a result of previous earthwork grading at the site. Wolf Creek is approximately 600 feet north of the site at an elevation approximately 100 feet lower than the site elevation and flows generally west.

2.2.9 Groundwater

The depth to groundwater at the site is not known. The California Department of Water Resources (DWR) *Well Completion Report Map Application* (DWR, 2024) depicts two monitoring wells approximately 500 feet north of the site at an approximate elevation of 2,530 feet above MSL (110 feet lower than the site elevation) where groundwater was encountered within 10 feet of the ground surface. These monitoring wells were near Wolf Creek. Information provided by Rise Gold Corporation indicates that the water level in the abandoned Idaho Maryland Mine workings is approximately 2,500 feet above MSL (approximately 140 feet below the current site elevation). Seasonal seepage and wet soil conditions have been reported near the lower, southeastern corner of the site.

2.3 Proposed Improvements

The 7.74-acre site is zoned light industrial (M-1). Appendix B is a *Preliminary Site Plan* prepared by Nelson Engineering (July 5, 2024). The *Preliminary Site Plan* includes 72,500 square feet (sf) of proposed building coverage and an additional 12,800 sf of future building coverage; 97,761 sf of pavement area; and 14,000 cubic yards of earthwork cut and fill. The single-story structures will be framed metal buildings with continuous perimeter foundations and interior concrete slabs-on-grade. Basements are not planned. Associated improvements will include the construction of landscaping, underground utilities, trash enclosure, depressed (below-grade) loading docks, bio-retention swale and detention pond, exterior flatwork, asphalt concrete paved parking areas and drive aisles. Based on the grades depicted on the *Preliminary Site Plan*, we anticipate that 4 to 6 feet of cut will be required for building subgrade and up to approximately 15 feet of fill is proposed at the perimeter of the development area. We understand that the fill will be generated from on-site excavation rather than being imported.

2.4 Previous Investigations

We performed a *Preliminary Geotechnical Evaluation* of the site and summarized our evaluation in a report dated August 5, 2024. Prior to site grading in 2013, geotechnical and geoenvironmental studies were performed by others. We obtained the following previous investigation reports from NV5, Inc. and from local government records.

2.4.1 Preliminary Geotechnical Study

H&K performed a preliminary geotechnical study in 2003 of a 66-acre property that included the site. Findings are presented in the *Preliminary Geotechnical Engineering Report, Milco and Platner Property, Between Whispering Pines Lane and East Bennett Road, Nevada County, California* (H&K, April 22, 2003).

2.4.2 Design-Level Geotechnical Study

H&K performed a design-level geotechnical study of the site in 2010, prior to site grading, for a previously proposed commercial development. Findings are presented in the *Geotechnical Engineering Report, Milco Business Park, Phase III, Whispering Pines Lane, APNs 09-680-49, -50, and -53, Nevada County, California* (H&K, November 8, 2010). Laboratory test results are summarized in Section 4.0.

3.0 FIELD EXPLORATION

We excavated ten exploratory trenches at the site on November 5, 2024. Approximate exploration locations are shown in Figure 2. Photographs and exploration logs are presented in Appendices A and C, respectively.

3.1 Exploratory Trenching

We advanced ten exploratory trenches (T24-01 through T24-10) to depths up to 8 feet by using a Caterpillar 420D backhoe equipped with a 24-inch-wide bucket. Upon completion, the trenches were backfilled with excavation spoils and were not compacted; therefore, the exploratory trench backfill may settle over time and is not suitable for support of structural improvements or flatwork unless the backfill is reworked during site development according to the grading recommendations presented herein. Subsurface conditions observed in the explorations are summarized below.

TABLE 3.1.1
SUMMARY OF EXPLORATORY TRENCHING

Exploratory Trench	**Approximate Elevation** (feet above MSL)	**Summary of Subsurface Observations**	**Ultramafic Rock** (Yes/No)
T24-01	2636	Completely to highly weathered diabase rock. Trench terminated at 6.5 feet in highly weathered rock.	No
T24-02	2638	Completely to moderately weathered diabase rock. Zones of more resistant rock may require splitting or grinding. Refusal of backhoe at 6.5 feet on large rock.	No
T24-03	2639	Completely to moderately weathered diabase rock. Zones of more resistant rock may require splitting or grinding. Trench terminated at 8.0 feet.	No
T24-04	2637	Engineered fill. Trench terminated at 7.5 feet in fill.	No
T24-05	2634	Engineered fill. Trench terminated at 5.0 feet in fill.	No
T24-06	2636	Undocumented fill in upper 1.5 feet. Engineered fill from 1.5 to 3.0 feet, underlain by undisturbed clayey silt. Trench terminated at 5.0 feet in completely weathered diabase rock.	No
T24-07	2636	Engineered fill. Trench terminated at 8.0 feet in fill.	No
T24-08	2641	Stockpile: sandy silt with gravel to cobble-size rock and variable wood debris. Trench terminated at 3.0 feet at base of stockpile.	No
T24-09	2639	Undocumented fill. Trench terminated at 4.0 feet in fill. Undocumented fill with ultramafic rock extends to a depth of 8 feet per H&K (2010) trench T1.	**Yes**
T24-10	2624	Clayey silt and clay near drainage swale in proposed road alignment. Trench terminated at 7.0 feet in undisturbed clayey silt.	No

Note: MSL = mean sea level

4.0 LABORATORY TESTING

We performed the following laboratory tests for soil classification and engineering material properties:

- Moisture-Density Relations (Compaction Curve, ASTM D1557)
- Moisture Determination and Unit Weight (ASTM D2937)
- Sulfate (California Test Method [CTM] 417)
- Chloride (CTM 422m)
- pH and Minimum Resistivity (CTM 643)

Test results are summarized in the following tables.

TABLE 4.1
SOIL MOISTURE-DENSITY RELATIONS

Sample Identification	**Trench ID**	**T24-04**	**T24-07**
	Sample ID	CB-04-01	CB-07-01
	Depth (feet)	1 – 7	1 – 7
Compaction Curve (ASTM D1557)	Maximum Dry Density (pcf)	89.7	100.2
	Optimum Moisture Content (%)	24.8	20.2
Soil Classification (ASTM D2487/D2488)	USCS Symbol	ML/MH	ML/MH
	Description	Sandy silt with clay, red (2.5 YR 48 to strong brown (7.5 YR 5/8)	Sandy silt with clay, strong brown (7.5 YR 5/6)

Notes: ASTM = ASTM International, USCS = Unified Soil Classification System, pcf = pounds per cubic foot

TABLE 4.2
SOIL MOISTURE AND DENSITY

Sample ID	Sample Depth (feet)	Moisture Content (%)	Dry Unit Weight (pcf)
ST-04-2.0	2	18.8	78.5
ST-04-4.0	4	18.8	73.3
ST-04-6.0	6	35.1	71.9
ST-05-2.0	2	17.8	70.5
ST-05-4.0	4	18.4	90.6
ST-06-2.0	2	14.6	84.7
ST-07-2.0	2	15.9	97.4
ST-07-4.0	4	21.2	102.9
ST-07-6.0	6	24.9	97.0

Notes: CTM = Caltrans Test Method, ohms-cm = ohms-centimeter, ppm = parts per million

The dry unit weight values reported in Table 4.2 do not necessarily represent the actual in-place density of the soil due to sample disturbance.

TABLE 4.3
SUMMARY OF SOIL CORROSIVITY TESTING

Sample ID	Sample Depth (feet)	pH (CTM 643)	Minimum Resistivity (ohms-cm) (CTM 643)	Chloride (ppm) (CTM 422M)	Sulfate (ppm) (CTM 417)
CB-03-01	1 - 4	4.82	10,180	2.9	7.0

Notes: CTM = Caltrans Test Method, ohms-cm = ohms-centimeter, ppm = parts per million

H&K (2010) performed the following laboratory tests:

- Direct Shear (ASTM D3080)
- Atterberg Limits (ASTM D 4318)
- Expansion Index (ASTM D4829)
- Resistance Value (R-Value, ASTM D2844)

A direct shear test (ASTM D3080) performed on a soil sample obtained from exploratory trench T3 at a depth of 2.5 feet below the 2010 ground surface resulted in a shear friction angle of 21.5 degrees and a cohesion of 2,083 pounds per square feet (psf).

An Atterberg Limits determination (ASTM D4318) performed on a soil sample obtained from 6 to 7 feet in exploratory trench T2 resulted in a liquid limit of 126, a plastic limit of 30, and corresponding plasticity index of 96 for the portion of the sample passing the No. 40 sieve. H&K classified the soil as sandy fat clay (Unified Soil Classification System [USCS] symbol CH).

An expansion index test (ASTM D4829) performed on a soil sample obtained from six feet in exploratory trench T2 by remolding a portion of the sample in a 1.0-inch-high ring and submerging it in water under an applied load of 144 psf and measuring the change in height with a dial micrometer. The test result of 75 indicated that the sample exhibited medium expansion potential pursuant to Uniform Building Code (UBC) guidelines.

An R-Value test (ASTM D 2844) performed on a bulk soil sample (described as light brown sandy clay, predominantly granular) obtained from 2 to 4 feet in exploratory trench T6 resulted in an R-Value of 21 as calculated by exudation pressure.

5.0 SEISMICITY AND GEOLOGIC HAZARDS

This section discusses the likelihood of geologic hazards identified by our literature review and site reconnaissance.

5.1 Abandoned Mine Features

The findings of our historical research, site reconnaissance, and subsurface investigation did not identify evidence of historical mining in the proposed development area. As discussed in Section 2.2.5, A vertical mine shaft is mapped near the western end of the site, downslope and west of the proposed development area (Figure 2, Photo 10). Historical mining maps indicate that the feature is an air shaft (ventilation shaft) associated with deep underground workings of the Idaho-Maryland Mine. We did not observe evidence of a shaft portal, concrete structures, or mine waste at the historically recorded shaft location. The absence of waste rock is likely because the shaft was excavated upward from the underground tunnel, and waste rock from the shaft excavation was likely caried down to tunnel level and out through the Idaho Main Shaft northwest of the site.

If improvements are planned within 100 feet of the recorded vertical shaft location, we recommend that the shaft location be determined by survey and the shaft portal be physically closed with a concrete slab or plug. Physical closure, if performed, should be performed under permit with Nevada County and according to an engineered design. Survey data and as-built closure documentation should be retained for future reference.

Deep underground mine workings associated with the Idaho-Maryland Mine extend beneath the site as discussed in Section 2.2.5. Based on the recorded depth of the underground mine workings beneath the site, we do not anticipate that the underground workings would impact the proposed site development from a geotechnical engineering standpoint.

5.2 Faulting and Seismicity

The site is not located on any known “active” earthquake fault trace. In addition, the site is not contained within an Alquist Priolo Zone. Therefore, fault rupture is not considered a hazard for the site. As discussed in Section 2.2.6, CGS (2024) maps no faults with identified Holocene displacement within approximately 30 miles of the site.

5.3 Liquefaction

Liquefaction is a phenomenon in which saturated, cohesionless soils are subject to a temporary loss of shear strength due to pore pressure build up under the cyclic shear stress associated with earthquakes. Liquefaction is more likely under strong ground shaking (from a large and nearby seismic source), when relatively clean, loose, granular soil (primarily poorly graded sands and silty sands) is present, and under saturated soil conditions.

As discussed in Section 2.2.6, the site is not in a designated Seismic Hazard Zone for liquefaction. We are not aware of any reported historical instances of liquefaction in the Grass Valley area. The site is not located near a large seismic source, subsurface conditions appear to be primarily granular, compacted fill and bedrock, and groundwater is relatively deep. Therefore, we expect that the potential for liquefaction and significant adverse impacts from liquefaction is low.

5.4 Landslides and Slope Stability

The proposed improvements include engineered, 2H:1V (horizontal to vertical) cut and fill slopes. Based on competent native materials at the site and the nature of the proposed improvements, we consider deep-seated slope instability to be unlikely. However, near-surface soil, undocumented fill, and highly weathered bedrock are subject to instability, particularly under saturated conditions and/or seismic forces. Therefore, we should assess the potential for slope instability during project design.

5.5 Expansive Soil

A relatively thin layer of clay soil was identified by H&K (2010). The soil was classified as fat clay (CH) and had a liquid limit of 126, a plastic limit of 30, and a plasticity index of 96, and exhibited medium expansion potential (Expansion Index = 75) per ASTM D4829 classification. The expansive clay soil likely remains in place in the northeastern corner of the site (exploratory trenches T-1, T-2, and T24-09) at depths below 6 to 7 feet (elevations less than 2,631 feet above MSL), and is also present in the proposed southeastern road alignment (exploratory trench T24-10).

The proposed finished subgrade elevation in the northeastern portion of the site is approximately 2,634 feet above MSL, and the base of footings may be at an elevation of approximately 2,632 feet above MSL. We recommend that the clay soil be removed from the building area where present within 3 vertical feet beneath the base of footings, as described in Section 6.1. We anticipate that the layer of clay soil can be blended with predominantly on-site granular soil and used as deep fill in the southeastern road alignment. We should observe soil conditions during earthwork improvements and foundation excavation to verify that the potentially expansive soil does not remain with a few feet of proposed building/surface improvements.

5.6 Naturally Occurring Asbestos

The referenced geologic maps indicate that the northeastern corner of the site is underlain by serpentinite, an ultramafic rock often associated with naturally occurring asbestos (NOA). Our surface and subsurface observations confirmed that serpentinite rock is present within approximately 170 feet of the eastern site boundary. We have prepared an Asbestos Dust Mitigation Plan (ADMP) for this project under separate cover.

When ultramafic rock, serpentinite, or NOA-containing minerals are encountered at a site, site grading is regulated under California Air Resources Board (CARB) Regulation 93105, *Asbestos Airborne Toxic Control Measure for Construction, Grading, Quarrying, and Surface Mining Operations* (Asbestos ATCM). The Asbestos ATCM specifies, 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 to prevent the generation of dust from the site.

No ultramafic rock, serpentinite, or NOA-containing minerals are to remain within six inches of finish grade at the completion of grading. The grading plans require the removal of approximately 4 vertical feet of soil and rock from the northeastern corner of the site to achieve the proposed finish subgrade elevation. As described in Section 6.1, we recommend that all ultramafic materials excavated from the northeastern corner of the site during site grading, as well as all undocumented fill and underlying expansive clay, be removed from the northeastern corner of the site and placed as deep engineered fill in the southeastern road alignment. These materials should not be placed within three vertical feet of the road subgrade elevation to reduce the possibility of disturbance during future utility trench excavation.

The ultramafic rock, undocumented fill, and expansive clay soil in the northeastern portion of the site should be replaced to finish subgrade elevation with engineered fill borrowed from the remainder of the site, outside of the mapped serpentinite area.

5.7 Soil Corrosion Potential

We retained Sunland Analytical Laboratory to perform pH, resistivity, chloride, and sulfate tests on a bulk soil sample from the site to estimate the corrosion potential of the soil with respect to proposed subsurface structures. The results presented in Section 4.0 should be considered for design of underground structures.

<u>Soil pH</u>

The soil sample was obtained from completely weathered diabase bedrock and had a pH value of 4.82. Per Caltrans *Corrosion Guidelines* (Caltrans, 2021), soil with a pH of 5.5 or lower may be corrosive to concrete or steel in contact with the ground. Based on the laboratory pH test results and Caltrans criteria, soil at the location tested has a higher propensity for corrosion.

Soil Resistivity

The soil sample had a soil resistivity value of 10,180 ohms-cm. Soil resistivity is the measure of the soil's ability to transmit electric current. Corrosion of buried ferrous metal is proportional to the resistivity of the soil. A lower resistivity indicates a higher propensity for transmitting electric currents that can cause corrosion of buried ferrous metal items. In general, the higher the resistivity, the lower the rate for corrosion. Per Caltrans *Corrosion Guidelines*, resistivity serves as an indicator parameter for the possible presence of soluble salts and it is not included as a parameter to define a corrosive area for structures. A minimum resistivity value for soil less than 1,500 ohm-cm may indicate the presence of high quantities of soluble salts and a higher propensity for corrosion. Based on the laboratory minimum resistivity test results and Caltrans criteria, soil at the location tested does not have a higher propensity for corrosion.

Chloride and Sulfate

The soil sample had a chloride concentration of 2.9 ppm and a sulfate concentration of 7.0 ppm. The reported chloride and sulfate concentrations are not considered to have a higher propensity for corrosion according to the guidelines summarized below.

TABLE 5.7A
REQUIREMENTS FOR CONCRETE EXPOSED TO CHLORIDE-CONTAINING SOLUTIONS
(AFTER ACI 318 TABLES 19.3.1.1 and 19.3.2.1)

Chloride Severity	Exposure Class	Condition	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	C0	Concrete dry or protected from moisture	N/A	2,500
Moderate	C1	Concrete exposed to moisture but not to external sources of chlorides	N/A	2,500
Severe	C2	Concrete exposed to moisture and an external source of chlorides	0.40	5,000

Table 5.7A above presents a summary of concrete requirements set forth by the CBC Section 1904 and American Concrete Institute (ACI) 318 for possible chloride exposure. Chlorides can break down the protective oxide layer on steel surfaces resulting in corrosion. Sources of chloride include, but are not limited to, deicing chemicals, salt, brackish water, seawater, or spray from these sources.

The appropriate Chloride Severity/Exposure Class should be determined by the project designer based on the specific conditions at the location of the proposed structure. Further guidance is provided in ACI 318. Per Caltrans *Corrosion Guidelines*, soil with a chloride concentration of 500 ppm or higher may be corrosive to steel structures or steel reinforcement in concrete. Based on Caltrans criteria, soil at the location tested is not considered to be corrosive with respect to chloride content.

Table 5.7B presents a summary of concrete requirements set forth by CBC Section 1904 and ACI 318 for sulfate exposure. Similar to chlorides, sulfates can break down the protective oxide layer on steel leading to corrosion. Sulfates can also react with lime in cement to soften and crack concrete.

TABLE 5.7B
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS
(AFTER ACI 318 TABLES 19.3.1.1 and 19.3.2.1)

<table>
<tr><th rowspan="2">Sulfate Severity</th><th rowspan="2">Exposure Class</th><th colspan="2">Water-Soluble Sulfate (SO_4) Content</th><th rowspan="2">Cement Type (ASTM C 150)</th><th rowspan="2">Maximum Water to Cement Ratio by Weight[1]</th><th rowspan="2">Minimum Compressive Strength (psi)</th></tr>
<tr><th>Percent By Mass</th><th>Parts Per Million (ppm)</th></tr>
<tr><td>Not Applicable</td><td>S0</td><td>$SO_4 < 0.10$</td><td>$SO_4 < 1,000$</td><td>No Type Restriction</td><td>N/A</td><td>2,500</td></tr>
<tr><td>Moderate</td><td>S1</td><td>$0.10 \leq SO_4 < 0.20$</td><td>$1,000 \leq SO_4 < 2,000$</td><td>II</td><td>0.50</td><td>4,000</td></tr>
<tr><td>Severe</td><td>S2</td><td>$0.20 \leq SO_4 \leq 2.00$</td><td>$2,000 \leq SO_4 \leq 20,000$</td><td>V</td><td>0.45</td><td>4,500</td></tr>
<tr><td rowspan="2">Very Severe</td><td>S3 – Option 1</td><td rowspan="2">$SO_4 > 2.00$</td><td rowspan="2">$SO_4 > 20,000$</td><td>V+Pozzolan or Slag</td><td>0.45</td><td>4,500</td></tr>
<tr><td>S3 – Option 2</td><td>V</td><td>0.40</td><td>5,000</td></tr>
</table>

Note: 1. Maximum water to cement ratio limits are different for lightweight concrete; see ACI 318 for details.

Based on the laboratory test results, the Sulfate Severity is classified as "Not Applicable", and the Exposure Class is S0. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration. The concrete mix design(s) should be developed accordingly.

Geocon does not practice in the field of corrosion engineering and the above information is provided as screening criteria only. If corrosion sensitive improvements are planned, we recommend that further evaluations by a corrosion engineer be performed to incorporate the necessary precautions to avoid premature corrosion on buried metal pipes and metal or concrete structures in direct contact with the soil.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on our literature review, subsurface trenching investigation, laboratory testing, engineering evaluation, and experience with soil and rock conditions in the area.

6.1 General

6.1.1 Resistant bedrock is present at the ground surface in the central portion of the site, and resistant bedrock outcroppings extend up to approximately 10 feet above the existing ground surface in the proposed improvement area. Based on the grades depicted on the *Preliminary Site Plan*, we anticipate up to approximately 6 feet of cut will be required to achieve the proposed building subgrade elevation and up to approximately 7 feet of cut will be required to achieve subgrade elevation in the paved area west of the building. Deeper excavation will be required for footings and underground utility trenches. We were able to excavate exploratory trenches T24-01, T24-02, and T24-03 to depths of 6 to 8 feet with significant effort using a Cat 420D backhoe with a 24-inch-wide bucket; however, we encountered refusal at an approximate depth of 6.5 feet in exploratory trench T24-02, which is located between two large, resistant rock outcroppings. We anticipate that resistant rock may be encountered during grading and/or excavation for foundations and utilities that requires grinding, splitting and/or blasting. Oversize rock encountered during excavation may be used in landscape areas, incorporated into slope protection, and/or incorporated in deep fill in accordance with specific recommendations from the geotechnical engineer of record.

6.1.2 Because the eastern portion of the site (Figure 2) contains ultramafic rock (serpentinite), site grading is regulated under CARB Regulation 93105, *Asbestos Airborne Toxic Control Measure for Construction, Grading, Quarrying, and Surface Mining Operations* (Asbestos ATCM). We have prepared an ADMP under separate cover to outline dust mitigation measures such as limiting site access, restricting onsite construction vehicle speeds, covering stockpiled soil, and liberal use of water during grading to prevent the generation of dust from the site. Soil excavated from the eastern end of the building area (e.g., to achieve finish subgrade and to remove undocumented fill) should be removed from the building area and placed as deep engineered fill in the southeastern road alignment. The excavated materials should be replaced to finish subgrade elevation with engineered fill borrowed from the remainder of the site, outside of the mapped serpentinite area.

6.1.3 We observed untested, undocumented fill at the locations of exploratory trenches T24-06 (approximately 18 inches deep) and T24-09 (approximately 7 feet deep) that is not suitable for support of the proposed improvements and should be removed and replaced in accordance with the recommendations presented in Section 6.5. According to the *Preliminary Site Plan*, grade is to be lowered approximately 4 feet at T24-09 to achieve building subgrade elevation. The undocumented fill remaining below finish subgrade elevation should not be relied upon to support footings and flatwork, and should be removed and replaced with engineered fill.

6.1.4 Expansive clay soil is present in the northeastern corner of the site underlying the undocumented fill at depths below six to seven feet (elevations less than 2,631 feet above MSL). The clay soil should be removed from the building area where present within 3 vertical feet beneath the base of footings. We anticipate that the layer of clay soil can be blended with predominantly granular soil and used as deep fill in the southeastern road alignment in accordance with recommendations provided by Geocon in the field. We should observe soil conditions during earthwork improvements and foundation excavation to verify that the potentially expansive soil does not remain with a few feet of sensitive improvements.

6.1.5 A soil stockpile, likely resulting from previous site grading, is in the northeastern portion of the site near the site entrance (Figure 2, exploratory trench T24-08). The stockpile surface is approximately three feet above the surrounding ground surface. We anticipate that the stockpiled soil can be used as deep fill in the southeastern road alignment after woody debris and other potentially unsuitable materials are removed. The soil conditions should be reviewed and approved by Geocon after debris removal and prior to placement as engineered fill.

6.1.6 If future improvements are planned within 100 feet of the recorded vertical shaft location at the lower, western end of the site (downslope and west of the currently proposed development area, see Figure 2), we recommend that the shaft location be determined by survey and the portal be physically closed with a concrete slab or plug. Physical closure, if performed, should be performed under permit with Nevada County and according to an engineered design. The location of the closed feature should be surveyed and recorded along with as-built closure documentation.

6.1.7 The lower, southeast corner of the site is to receive fill for road construction from the site to adjoining property to the south-southeast. Excavation of a base key for the roadway fill will likely encounter clay soil and seasonal surface water, groundwater seepage, and saturated soil conditions. Significant drying effort to attain moisture content suitable for compaction should be anticipated regardless of the time of year, and particularly during and following the rainy season.

6.1.8 In addition to the potentially wet soil conditions near the lower, southeastern corner of the site, moist to saturated soil conditions may be encountered in excavations advanced during and following the rainy season, particularly in excavations that reveal the soil/weathered rock transition. If grading occurs during or after the wet season (typically winter and spring), or in periods of precipitation, in-place and excavated soils will likely be wet. Earthwork contractors should be aware of moisture sensitivity of clayey and fine-grained soils and potential compaction/workability difficulties.

6.1.9 The conclusions and recommendations provided in this report are based on our understanding of the proposed development at this time. We should review the project plans as they develop further, provide engineering consultation as needed during final design, and perform geotechnical observation and testing services during construction.

6.2 Seismic Design Criteria

6.2.1 Seismic design of structures should be performed in accordance with the provisions of the 2022 CBC which is based on the American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI) publication *ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI, 2017). We used the Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) web application *Seismic Design Maps* (https://seismicmaps.org/) to evaluate site-specific seismic design parameters in accordance with ASCE 7-16 (Appendix F).

For seismic design purposes, sites are classified as Site Class “A” through “F” as follows:

- Site Class A – Hard Rock;
- Site Class B – Rock;
- Site Class C – Very Dense Soil and Soft Rock;
- Site Class D – Stiff Soil;
- Site Class E – Soft Clay Soil; and
- Site Class F – Soils Requiring Site Response Analysis.

Based on the subsurface conditions at the site and the presence of relatively deep compacted fill, the general Site Classification is Site Class "C – Very Dense Soil and Soft Rock" per Table 20.3-1 of ASCE/SEI 7-16. For the purpose of evaluating code-based seismic parameters for design, we assumed a seismic Risk Category II (per the CBC) for the project. Results are summarized in Table 6.2.1.

TABLE 6.2.1
ASCE 7-16 SEISMIC DESIGN PARAMETERS
SITE CLASS "C – VERY DENSE SOIL AND SOFT ROCK"

Parameter	Value	ASCE 7-16 Reference
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.560g	Figure 22-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.234g	Figure 22-2
Site Coefficient, F_A	1.3	Table 11.4-1
Site Coefficient, F_V	1.5	Table 11.4-2
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.715g	Eq. 11.4-1
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.350g	Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.477g	Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.234g	Eq. 11.4-4

6.2.2 Table 6.2.2 presents additional seismic design parameters for projects with Seismic Design Categories of D through F in accordance with ASCE 7-16 for the mapped geometric mean peak ground acceleration (PGA) for the maximum considered earthquake (MCE_G).

TABLE 6.2.2
ASCE 7-16 SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.243g	Figure 22-9
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.292g	Section 11.8.3 (Eq. 11.8-1)

6.2.3 Conformance to the criteria presented in Tables 6.2.1 and 6.2.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, since such design may be economically prohibitive.

6.3 Soil and Excavation Characteristics

6.3.1 Excavation characteristics will vary at the site depending on location and excavation depths. Table 6.3.1 summarizes anticipated excavation characteristics.

TABLE 6.3.1
ANTICIPATED EXCAVATION CHARACTERISTICS

Geologic Unit	Excavation Characteristics
Fill	Engineered fill material generally consists of clayey silt and sandy silt with angular, gravel to cobble size rock. The fill was borrowed onsite. We anticipate standard to moderate excavation effort with conventional, heavy-duty grading equipment. Undocumented fill was observed in the upper 18 inches of exploratory trench T24-06 and in the upper seven feet at the location of exploratory trench T24-09. This fill is not suitable for support of site improvements and must be replaced. The undocumented fill at location T24-09 contains oversize ultramafic rock and may be subject to sidewall caving when excavated.
Residual Soil	Up to approximately 15 feet of residual soil were was cut from the central portion of the site during site grading in 2013. Areas of resistant rock outcrop extend up to 10 feet above current grade. We were able to excavate exploratory trenches T24-01, T24-02, and T24-03 to depths of 6 to 8 feet with significant effort by Caterpillar 420D backhoe with a 24-inch-wide bucket; however, we encountered refusal at a depth of 6.5 feet in exploratory trench T24-02, which is located between two large, resistant rock outcroppings.
Igneous Bedrock	Completely to moderately weathered diabase rock was encountered at the ground surface in exploratory trenches T24-01, T24-02, and T24-03, and the weathering of formational material generally decreases with depth. The presence of oversize rock (greater than 6 to 12 inches in maximum dimension) should be anticipated and may increase excavation difficulty. We anticipate moderate to difficult excavation effort with conventional, heavy-duty grading equipment. Pre-ripping with a large dozer (such as Caterpillar D10 or larger) may be required, and large excavators (such as Caterpillar 323 or larger) with a ripping shank or rock trenchers will likely be required for trenching at some locations. We anticipate that resistant rock may be encountered during grading and/or excavation for foundations and utilities that requires grinding, splitting and/or blasting. Oversize rock encountered during excavation may be used in landscape areas, incorporated into slope protection, and/or incorporated in deep fill in accordance with specific recommendations from the geotechnical engineer of record.

6.3.2 Protruding rocks in excavation bottoms should be removed and resulting depressions filled in accordance with the recommendations in this report.

6.3.3 The project excavations will generate oversized rock material (greater than 6 inches in dimension) and boulders at the anticipated excavation depths. Excavation difficulty may be difficult in the upper 5 feet due to the presence of weathered bedrock and boulders, and may increase significantly in areas of less weathered igneous rock. The contractor should select appropriate excavation equipment.

6.3.4 Temporary excavations deeper than 4 feet and entered by workers must meet Cal-OSHA requirements as appropriate. Excavation sloping, benching, the use of trench shields, and the placement of trench spoils should conform to the latest applicable Cal-OSHA standards. The contractor should have a Cal-OSHA-approved “competent person” onsite during excavation to evaluate trench conditions and to make appropriate recommendations where necessary. It is the contractor’s responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements.

6.3.5 The excavation support recommendations provided by Cal-OSHA are generally geared toward protecting human life and not necessarily toward preventing damage to nearby structures or surface improvements. The contractor should be responsible for using the proper active shoring systems or sloping to prevent damage to any structure or improvements near underground excavations.

6.3.6 Permanent cut and fill slopes should be constructed no steeper than 2H:1V (horizontal to vertical). To mitigate potential erosion, slopes should be vegetated as soon as possible and surface drainage should be directed away from the tops of slopes.

6.3.7 Seasonal shallow perched groundwater (seepage) could be present if grading occurs during or after the wet season (typically winter or spring). Perched groundwater typically develops atop cemented horizons and at the contact between residual soil and formational material. Fill derived from shallow excavations during perched groundwater conditions will likely need to be aerated/dried to achieve suitable moisture content for compaction. We should evaluate conditions in the field at the time of construction and evaluate the type, level, and extent of mitigation alternatives.

6.3.8 If grading occurs during or after the wet season (typically winter and spring) or in periods of precipitation, in-place and excavated soils will likely be wet. Earthwork contractors should be aware of moisture sensitivity of the near-surface fine-grained soil and potential compaction/ workability difficulties. The presence of weathered formational materials tends to exacerbate wet soil conditions as water can become trapped (perched) on the less permeable materials.

6.3.9 Earthwork and pad preparation operations in wet conditions will likely be difficult with low productivity. Often, a period of at least one month of warm and dry weather is necessary to allow the site to dry sufficiently so that heavy grading equipment can operate effectively. Conversely, during dry summer and fall months, dry clay soils may require additional grading effort (discing, mixing, or other means) to attain proper moisture conditioning.

6.3.10 Given the variable range of in-situ moisture content and the fine-grained, silty nature of the soil, additional grading effort and moisture conditioning may be required to achieve suitable moisture content for compaction. Fine-grained soils at the site with above-optimum moisture content may experience instability under construction equipment loading in project excavations and exposed subgrades. Difficulty achieving compaction of soils with high moisture content should be anticipated when excavated onsite soil is placed as backfill. Mitigation alternatives may include aerating/drying the exposed soils (assuming favorable weather conditions), or chemical treatment (e.g., lime treatment). Unstable excavation bottoms may require over-excavating 12 to 18 inches and placing crushed rock wrapped in a geotextile fabric or geogrid covered with aggregate for stabilization. We can provide specific recommendations during construction, based on conditions encountered.

6.4 Materials for Fill

6.4.1 Excavated soil and rock generated from cut operations at the site are suitable for use as engineered fill in structural areas provided they are selectively placed during grading in accordance with the following recommendations:

- Deleterious material, material with greater than 3% organics by weight, and debris should be exported from the site and not incorporated into structural fill.
- Fill material in areas with underground utilities and foundations should consist of 6-inch-minus material with a sufficient amount of soil to provide adequate binder to reduce the potential for excavation caving.
- In other areas (general fill areas without utilities or foundations) rock or cementations up to 2 feet in maximum dimension may be used. However, this material should contain a sufficient amount of smaller rock and soil to fill void spaces between large rocks and avoid rock nesting (concentrations of rock with void space). Nesting of oversized rock should be avoided.
- If sufficient soil fill materials are not present at the site to mix with on-site rock material, import of soil fill material will be necessary.

6.4.2 If soil is to be imported to the site, it should be primarily granular with a "very low" expansion potential (Expansion Index less than 20), have a Liquid Limit less than 50, Plasticity Index less than 15, be free of organic material and construction debris, not contain rock/cementations larger than 3 inches in greatest dimension, and contain sufficient fines (approximately 12% to 15% or more) to act as a binder to reduce caving potential when excavated.

6.4.3 Environmental characteristics and corrosion potential of import soil materials should also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.5 Grading

6.5.1 All earthwork operations should be observed and all fills tested for recommended compaction and moisture content by a representative of Geocon.

6.5.2 References to relative compaction and optimum moisture content in this report are based on the latest ASTM D1557 Test Procedure. Structural building pad areas should extend a minimum of 5 feet horizontally beyond the outside dimensions of structures, including footings and overhangs carrying structural loads.

6.5.3 Prior to commencing grading, a pre-construction conference with representatives of the client, grading contractor, and Geocon should be held at the site. Site preparation, soil handling, and/or the grading plans should be discussed at the pre-construction conference.

6.5.4 Site preparation should begin with removal of existing vegetation, debris, surface/subsurface structures (if any), and any organic material. Within areas to be developed, any existing trees or shrubs and associated root systems should be removed. Roots larger than 1 inch in diameter should be completely removed. Smaller roots may be left in place as conditions warrant and at the discretion of our field representative. Surface vegetation consisting of grasses and other similar vegetation should be removed by stripping to a sufficient depth to remove organic-rich topsoil. We estimate required stripping depths will range from approximately 1 to 2 inches. The actual stripping depth should be determined based on site conditions prior to grading. Material generated during stripping is not suitable for use within 5 feet of building pads or within pavement/flatwork areas but may be placed in landscaped or non-structural areas or exported from the site.

6.5.5 Alternatively, surface vegetation may be mowed such that 1 to 2 inches of stubble remains. After removing mowed vegetation, the ground surface should be thoroughly disced in two perpendicular directions to a depth of 12 inches to blend the remaining grass and roots into the surface soil. The resulting soil should be thoroughly mixed such that vegetation segments longer than 1 inch are not visually discernable, and the overall organic content is 3% by dry weight or less.

6.5.6 Within the proposed building areas (as defined in Paragraph 6.5.2), existing undocumented fill should be completely removed to expose undisturbed alluvium, residual soil, or weathered bedrock. Undocumented fill containing ultramafic rock (serpentinite) is present to a depth of approximately 7 feet near the northeastern corner of the site and is underlain by a relatively thin layer of expansive clay. These materials must be removed, and onsite materials borrowed from the non-serpentine remainder of the site may be used to construct engineered fill to finish subgrade elevation, which anticipated to be approximately four feet below existing grade. Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.5.7 The most effective site preparation alternatives will depend on site conditions prior to grading. We should evaluate site conditions and provide supplemental recommendations immediately prior to grading, if necessary.

6.5.8 In general, where fill will be placed on slopes steeper than 5H:1V, we recommend that horizontal benches angled slightly into the slope be cut into competent formational material on the slopes prior to placing fill. Benches should roughly parallel slope contours. These benches should extend at least 2 feet into competent formational material. In addition, a keyway should be cut into the slope at the base of the fill. In general, keyways should be at least 15 feet wide and extend at least 2 feet into competent formational material. Subdrains may be required along the back edge of keyways and/or benches. Bench and keyway criteria may need revision during construction based on the actual materials encountered and grading performed in the field.

6.5.9 After site preparation, the bottom of cut areas, areas left at grade, and areas to receive fill, should be scarified at least 12 inches, uniformly moisture-conditioned at or above optimum moisture content and compacted to at least 90% relative compaction. Scarification and re-compaction operations should be performed in the presence of our representative to evaluate performance of the subgrade under compaction equipment loading and to identify any areas that may require additional removals.

6.5.10 Engineered fill should be compacted in horizontal lifts not exceeding 8 inches (loose thickness) and brought to final subgrade elevations. Each lift should be moisture-conditioned at or above optimum moisture content and compacted to at least 90% relative compaction. The top 12 inches of building pads, whether completed at-grade or by excavation or filling, should be uniformly moisture-conditioned at or above optimum moisture content and compacted to at least 90% relative compaction, or per the compaction method specification presented in Section 6.5.11 if the soil contains greater than 30% rock larger than ¾ inches by mass.

6.5.11 Soils exceeding 30% rock larger than ¾ inches by mass are considered non-testable by conventional methods. In this case, the following compaction method specification will apply. Compaction equipment shall consist of a self-propelled sheepsfoot compactor with a minimum operating weight of 12 tons (Caterpillar 563 or equivalent). Under the continuous observation of a representative of Geocon, each lift of rocky soil fill shall be moisture-conditioned and uniformly compacted in place by 6 to 8 passes with the approved compactor. Additional passes as deemed necessary during fill placement to achieve the desired condition based upon field conditions may be recommended. Each compaction pass shall overlap the adjacent pass by a minimum of 1 foot. Geocon will visually verify proper lift thickness, spreading, mixing, and compaction operations. Fills containing soils exceeding 30% rock larger than ¾ inches by mass should be placed and proof-rolled under our observation.

6.5.12 Site grading may result in cut-fill transitions below some building pads. To reduce potential for differential settlement of planned structures, the cut portion of cut-fill transition building pads, if encountered, should be undercut to at least the depth of the fill, not to exceed 3 feet, and replaced with properly compacted fill soils. The undercut should extend at least 5 feet beyond the structure perimeter. Our office should review the grading plans to provide supplemental recommendations, if necessary.

6.5.13 Final pavement subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned at or above optimum moisture content, be compacted to at least 95% relative compaction, and be stable. The 95% relative compaction requirement applies to the top 6 inches of pavement area subgrade; however, underlying materials must be sufficiently compacted and stable. We recommend proof-rolling the subgrade with a loaded water truck (or similar equipment with high contact pressure) to verify the stability of the subgrade prior to placing aggregate base (AB). We note that deeper scarification, moisture-conditioning, and compaction efforts may be required in order to achieve overall stability and compaction.

6.5.14 Underground utility trenches should be backfilled with properly compacted material. Pipe bedding, shading, and backfill should conform to the requirements of the appropriate utility authority. Material excavated from trenches should be adequate for use as general backfill above shading provided it does not contain deleterious matter, vegetation, or cementations larger than 6 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 8 inches. Lifts should be compacted to a minimum of 90% relative compaction at or above optimum moisture content. Compaction should be performed by mechanical means only; jetting of trench backfill should not be allowed.

6.6 Shallow Foundations and Conventional Interior Slabs-on-Grade

6.6.1 Provided the building pads are graded in accordance with the recommendations of this report, the proposed buildings may be supported on conventional shallow foundations bearing on undisturbed native soil/rock or engineered fill. Deep foundations would reduce the chance of future settlement in areas of deep fill, but are likely not cost-effective for the project.

6.6.2 Foundations should consist of continuous perimeter footings with interior spread footings. Perimeter footings should be continuous around the entire perimeter of the structure without breaks or discontinuities. Continuous footings should be at least 12 inches wide and interior spread footings should be at least 24 inches square. All footings should be embedded at least 18 inches below pad grade.

6.6.3 Footing bottoms should be level, and projections of rock greater than 2 inches above the footing bottom should be removed or a leveling course of structural fill, crushed rock, or lean-mix concrete should be placed to at least 2 inches higher than the highest projection of rock. The intent of removing rock projections or placing fill is to avoid point loading of the foundation.

6.6.4 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area within 18 inches laterally of the footing, beneath the footing, and within a 1H:1V plane extending out and down from the bottom of the footing.

6.6.5 Continuous footings should be reinforced with at least four No. 4 reinforcement bars, two each placed near the top and bottom of the footing to allow footings to span isolated soil irregularities. The reinforcement recommended above is for soil characteristics only and is not intended to replace reinforcement required for structural considerations. The project structural engineer should evaluate the need for additional reinforcement.

6.6.6 Shallow foundations may be designed for an allowable bearing capacity of 3,000 pounds per square foot (psf) for dead plus live loads. A one-third increase in allowable bearing capacity is permitted for use with the alternative load combinations given in Section 1605.2 of the 2022 CBC.

6.6.7 Allowable passive pressure used to resist lateral movement of the footings may be assumed to be equal to a fluid weighing 350 pounds per cubic foot (pcf). The coefficient of friction to resist sliding is 0.35 for concrete against soil/rock. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

6.6.8 Foundations designed in accordance with the recommendations above should experience total post-construction settlement due to building loads of less than one inch and differential settlement of ½ inch or less over a distance of 30 feet due to the building loads. The majority of settlement will be immediate and occur as the buildings are constructed. Foundations constructed in areas of deep fill may experience greater settlement over time.

6.6.9 A Geocon representative should observe foundation excavations prior to placing reinforcing steel or concrete to observe that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

6.6.10 Conventional interior concrete slabs-on-grade are suitable for the building pads prepared as recommended in this report. Slab thickness and reinforcement should be determined by the structural engineer based on anticipated loading. However, slabs should be at least 4 inches thick and reinforced with at least No. 4 reinforcing bars placed 24 inches on center, each way. Control joints should be provided at periodic intervals in accordance with ACI or Portland Cement Association (PCA) recommendations, as appropriate.

6.6.11 If building pad soils become dry, they should be re-moistened prior to concrete slab-on-grade construction. Building pads should be moistened to at least optimum moisture content, at least 48 hours before placing the vapor barrier. Moisture content should be verified by Geocon prior to placing the vapor barrier.

6.6.12 Migration of moisture through concrete slabs-on-grade or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner and design team, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field.

6.6.13 For slabs that receive floor coverings, a minimum 10-mil-thick vapor retarder meeting ASTM E1745-97 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) may be used. The vapor barrier, if used, should extend to the edges of the slab and should be sealed at all seams and penetrations. At least 4 to 6 inches of ½- or ¾-inch crushed rock, with no more than 5% passing the No. 200 sieve, may be placed below the vapor barrier to serve as a capillary break. For slabs subject to heavy floor loads, the crushed rock layer may be replaced with Class 2 aggregate base uniformly compacted to at least 95% relative compaction. In this case, the vapor retarder/barrier membrane should be placed above the aggregate base layer. Please note that the AB layer is not an effective capillary moisture break and there may be an increased potential for moisture vapor transmission through the slab.

6.6.14 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.6.15 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the ACI, PCA, and ASTM.

6.7 Retaining Walls and Lateral Loads

6.7.1 Design of retaining walls and buried structures retaining cut native soil or engineered fill with appropriate drainage may be based on the lateral earth pressures (equivalent fluid pressure) summarized in Table 6.7.1.

TABLE 6.7.1
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density	
	Horizontal Retained Backfill Surface Without Surcharge	Sloping Retained Backfill up to 2H:1V
Active	40 pcf	60 pcf
At-Rest	60 pcf	80 pcf
Seismic[1]	Not Applicable	Not Applicable

Note: 1. Based on research by Lew, et al. 2010, the seismic increment of earth pressure may be neglected if the maximum peak ground acceleration (PGA) at the site is 0.4 g or less. The Site Class Modified MCE_G Peak Ground Acceleration (PGA_M) for this site is 0.292g; therefore, the seismic increment of earth pressure may be neglected.

6.7.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.001H (where H is the height of the wall). Walls restrained from movement should be designed using the at-rest case. The soil pressures above assume that the backfill material within an area bounded by the wall and a 1H:1V plane extending upward from the base of the wall will be composed of the existing onsite soils.

6.7.3 Retaining wall foundations with a minimum depth of 18 inches may be designed using the allowable bearing capacity provided in Section 6.6.1.5 of this report. To resist lateral movement of retaining wall foundations, an allowable passive earth pressure equivalent to a fluid density of 350 pcf may be used for footings or shear keys poured neat against properly compacted engineered fill soils or undisturbed natural soils. This allowable passive pressure is based on the assumption that a horizontal surface extends at least 5 feet or three times the depth of the footing or shear key, whichever is greater, beyond the face of the retaining wall foundation. If this surface is not protected by floor slabs or pavement, the upper 12 inches of material should not be included in the design for lateral resistance. An allowable friction coefficient of 0.35 may be used for resistance to sliding between soil and concrete. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

6.7.4 Retaining walls greater than 2 feet tall (retained height) should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material, which leads to suitable drainage facilities.

6.7.5 The recommendations presented above are generally applicable to the design of rigid concrete or masonry retaining walls with a level backfill and having a maximum retained height of 10 feet. In the event that walls higher than 10 feet or other types of walls are planned, Geocon should be consulted for additional recommendations.

6.8 Concrete Sidewalks and Flatwork

6.8.1 It should be noted that even with implementation of the following measures, minor slab movement or cracking could still occur.

6.8.2 Concrete flatwork should be at least 4 inches thick and underlain by at least 6 inches of non-expansive soil or aggregate base (AB) compacted to at least 90% relative compaction. The upper 12 inches of subgrade soil in exterior flatwork areas should be uniformly moisture-conditioned at least 2% above optimum and compacted to at least 90% relative compaction.

6.8.3 We recommend using construction and control joints in accordance with ACI and/or PCA guidelines. Construction joints that abut building foundations should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. Exterior slabs should be structurally independent of building foundations except at doorways, where vertical offset could impact doorway operation. Dowels should be used at these locations.

6.8.4 To reduce the potential for water from landscaped areas migrating under concrete flatwork and into the AB, consideration should be given to using plastic moisture cutoffs or full-depth curbs in areas where flatwork abuts irrigated landscaping. The cutoffs or full-depth curbs should extend at least 4 inches or more into the soil subgrade beneath the AB.

6.9 Pavement – Hot Mix Asphalt

6.9.1 H&K (2010) performed Resistance-Value (R-Value) testing on a bulk soil sample described as light brown sandy clay obtained from 2 to 4 feet in their exploratory trench T-6 (Figure 2). The soil had an R-value of 21 based on the exudation pressure; however, the expansion pressure was extrapolated to be 87 psf. We are providing the following recommended pavement sections based on a design R-Value of 15 and assumed traffic index (TI) values of 4 through 7. We can provide recommendations for additional TI values if required for the anticipated traffic patterns. We assume that areas of concentrated/heavy traffic loading (e.g., forklift traffic) will be designed with concrete pavement rather than flexible pavement.

TABLE 6.8.1
RECOMMENDED FLEXIBLE PAVEMENT SECTIONS

Traffic Index: 4 **Design R-Value:** 15 **Automobile Parking Only**	**Alternate A Pavement Section** (inches)	**Alternate B Pavement Section** (inches)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	2.5	3.0
Caltrans Section 26, Class II Baserock, 95% Compaction	8.0	6.0
Subgrade Soil, 95% Compaction	6.0	6.0
Traffic Index: 6 **Design R-Value:** 15 **Automobile Traffic and Driveways**	**Alternate A Pavement Section** (inches)	**Alternate B Pavement Section** (inches)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	2.5	3.5
Caltrans Section 26, Class II Baserock, 95% Compaction	13.0	11.0
Subgrade Soil, 95% Compaction	6.0	6.0
Traffic Index: 7 **Design R-Value:** 15 **Moderate to Heavy Truck Traffic**	**Alternate A Pavement Section** (inches)	**Alternate B Pavement Section** (inches)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	3.0	4.0
Caltrans Section 26, Class II Baserock, 95% Compaction	15.0	13.0
Subgrade Soil, 95% Compaction	6.0	6.0

Note: Traffic index and loading conditions to be determined by others. We can provide recommendations for additional TIs if required for the anticipated traffic patterns. We assume that areas of heavy loading (e.g., forklift traffic) will be designed with concrete pavement rather than flexible pavement.

6.9.2 Subgrade in areas to be paved must be stable, moisture-conditioned, and compacted in accordance with the recommendations of this report. The upper six inches of subgrade should be compacted to at least 95% of the ASTM D1557 maximum dry density. Prior to placing AB, subgrade soil should be proof rolled with a loaded water truck to verify stability. Proof rolling should be observed and accepted by Geocon prior to AB placement.

6.9.3 AB should be Class 2 with a minimum R-Value of 78 in accordance with the requirements of Section 26 of the latest Caltrans *Standard Specifications* and should be compacted to at least 95% or higher relative compaction at or near optimum moisture content. Prior to placing HMA, the AB should be proof rolled with a loaded water truck to verify stability. Proof rolling should be observed and accepted by Geocon prior to HMA placement.

6.9.4 HMA should conform to Section 39 of Caltrans' latest Standard Specifications. Periodic maintenance of HMA pavement is required to achieve the service life of the pavement.

6.9.5 HMA pavement section recommendations for driveways and parking areas are typically based on the design procedures of Caltrans' *Highway Design Manual* (Design Manual), Chapter 600, latest edition. It should be noted that most rational pavement design procedures are based on projected street or highway traffic conditions and, hence, may not be representative of vehicular loading that occurs in parking lots and driveways. Pavement proximity to landscape irrigation, reduced traffic speed, and short turning radii increase the potential for pavement distress to occur in parking lots even though the volume of traffic is significantly less than that of an adjacent street. The resulting pavement sections for parking lots based on traditional pavement design methods are reasonable because additional asphalt surfacing can be added later, if needed, and generally without incurring traffic hazards or traffic handling problems. It is generally not economically feasible to design and construct the entire parking lot and driveways for the unique loading conditions previously described. Periodic maintenance of the pavement in these areas, therefore, should be anticipated.

6.9.5 To reduce the potential for water from landscaped areas migrating under pavement into the AB, consideration should be given to using full-depth curbs in areas where pavement abuts irrigated landscaping. The full-depth curbs should extend at least 4 inches or more into the soil subgrade beneath the AB. Alternatively, modified drop-inlets that contain weepholes may be used to encourage accumulated water to drain from beneath the pavement.

6.10 Pavement – Rigid Concrete

6.10.1 If rigid Portland cement concrete (PCC) pavement is used in automobile and light-truck traffic areas and in front of trash bins, we recommend that the PCC pavement be at least 6 inches thick. PCC pavement should be underlain by at least 6 inches of Class 2 AB meeting the requirements of Section 26 of Caltrans' *Standard Specifications* and compacted to at least 95% relative compaction.

6.10.2 Subgrade soils should be prepared and compacted in accordance with the recommendations of this report. Subgrade should be finished to a smooth, unyielding surface and proof-rolled with a loaded water truck to verify stability.

6.10.3 PCC should have a minimum 28-day compressive strength of 3,500 pounds per square inch (psi). Adequate construction and crack control joints should be used to control cracking inherent in concrete construction. We note that the American Concrete Pavement Association (ACPA) recommends a maximum joint spacing no greater than 24 times the slab thickness for PCC pavements directly underlain by granular bases.

6.10.4 Steel reinforcement, if used, should be detailed in accordance with PCA, ACI, or similar guidelines. Alternatively, macro synthetic fibers (Euclid Chemical Tuf-Strand SF or equivalent) mixed into the concrete mix may be considered in lieu of conventional steel reinforcement provided they meet the requirements of ASTM C1116 and ASTM D7508 for Type III Synthetic Fibers.

6.10.5 Adequate dowels should also be used at joints to facilitate load transfer and reduce vertical offset. In general, we recommend that concrete pavements be detailed, designed, constructed, and maintained in accordance with industry standards such as those provided by the ACI and ACPA.

6.11 Site Drainage and Moisture Protection

6.11.1 Adequate site drainage is critical to reduce the potential for differential soil movement, soil expansion, erosion, and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to building foundations. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with the 2022 CBC or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices.

6.11.2 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.11.3 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend use of area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes. In addition, where landscaping is planned adjacent to the pavement or flatwork, we recommend construction of a cutoff wall (deepened curb) along the edge of the pavement/flatwork that extends at least 4 inches into the soil subgrade below the bottom of the base material.

6.11.4 We recommend that roof drains be connected to water-tight subdrains that direct the water to the storm drain system. However, we understand that Low-Impact Development (LID) and Leadership in Engineering and Environmental Design (LEED) requests disconnecting the roof drains to help obtain certification. The water from the roof drains should be directed away from buildings. Consideration should be given to draining roofs to lined planter boxes or placing liners below the proposed landscape areas to prevent infiltration of the water. Geocon can be contacted for additional recommendations.

6.11.5 We recommend implementing measures to reduce infiltrating irrigation water near buildings, flatwork, or pavements. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within 3 feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers.
- Using automatic timers for irrigation systems.
- Using appropriately spaced area drains.

The project landscape architect should consider incorporating these measures into the landscaping plans.

6.11.6 Experience has shown that even with these provisions, subsurface seepage may develop in areas where no such water conditions existed prior to site development. This is particularly true where a substantial increase in surface water infiltration has resulted from an increase in landscape irrigation.

7.0 FURTHER GEOTECHNICAL SERVICES

To reduce uncertainties regarding the subsurface conditions and to verify that our recommendations are incorporated into the project design and construction, we should be retained to review plans and specifications and perform observation and testing during project development.

7.1 Plan and Specification Review

Geocon should review the foundation and grading plans prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record (GER) throughout the construction phase and provide construction observation and testing services. Providing these services during construction is important in order to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others' interpretation of our recommendations or the future performance of the project.

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

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.

These services were performed consistent with our agreement with our client. We are not responsible for the impacts of any changes in 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.

This report is issued with the understanding that it is the responsibility of the owner or their representative to ensure that the information and recommendations contained herein are brought to the attention of the design team for the project and incorporated into the plans and specifications and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The recommendations contained in this report are preliminary until verified during construction by representatives of our firm. 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 we 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 regard to their impact 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 to 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.

The analyses, conclusions and recommendations presented in this report are based on site conditions as they existed at the time we performed the surface and subsurface field investigations. We have assumed that the subsurface soil and groundwater conditions encountered at the exploratory locations are generally representative of the subsurface conditions throughout the entire project 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, we should be notified immediately so that we can review these differences and, if necessary, modify our recommendations.

The project site map shows approximate exploratory locations as determined by a hand-held global positioning system (GPS) unit. Therefore, the exploratory locations and other mapped site features should not be relied upon as being exact nor located with surveying methods. The elevation or depth to groundwater underlying the project site may differ with time and location.

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, the recommendations presented in this report should not be relied upon after a period of three years from the issue date without our review.

9.0 REFERENCES

Adams, W.J., undated, *Plan of Underground Workings of Idaho-Maryland Development Co., Grass Valley, California, Plate II*, scale 1 inch = 200 feet

American Concrete Institute, ACI 318-19, *Building Code Requirements for Structural Concrete and Commentary*, 2019

American Society of Civil Engineers / Structural Engineering Institute, ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, 2017

Anonymous, 1950, *Composite map of Idaho-Maryland Mine to 2000 Level*

California Building Standards Commission, *2022 California Building Code*, based on 2021 International Building Code, International Code Council, 2022

California Department of Transportation (Caltrans), Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Branch, *Corrosion Guidelines*, version 3.2, May 2021.

Caltrans, *Standard Specifications*, Section 26, 2018

California Department of Water Resources (DWR), accessed July 2024, *Well Completion Report Map Application*, https://water.ca.gov/Programs/Groundwater-Management/Wells/Well-Completion-Reports

California Geological Survey (CGS), 2022, Fault Activity Map of California, California Department of Conservation, available online at https://maps.conservation.ca.gov/cgs/fam/

CGS, 2018, Special Publication 42, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners / Developers and Geoscience Practitioners for Assessing Fault Rupture Hazards In California*, Department of Conservation

Geocon Consultants, Inc., August 5, 2024, *Preliminary Geotechnical Evaluation, Jada Windows, Grass Valley, Nevada County, California*

Google Earth, 2004, aerial photograph of Grass Valley area, imagery date August 12, 2024

Holdrege & Kull Consulting Engineers and Geologists (H&K), April 22, 2003, *Preliminary Geotechnical Engineering Report, Milco and Platner Property, Between Whispering Pines Lane and East Bennett Road, Nevada County, California*

H&K, November 8, 2010, *Geotechnical Engineering Report, Milco Business Park, Phase III, Whispering Pines Lane, APNs 09-680-49, -50, and -53, Nevada County, California*

H&K, November 4, 2010, *Phase I Environmental Site Assessment, Milco Business Park, Phase III, APNs 09-680-49, -50 and -53, Nevada County, California*

Johnston, W.D., Jr., 1939, *Geologic Map of the Grass Valley Quadrangle and Adjacent Area, Nevada County, California*, United States Department of the Interior Geological Survey, Professional Paper 194, Plate 1

Johnston, W.D., Jr., 1940, *The Gold Quartz Veins of Grass Valley, California*, Geological Survey Professional Paper 194, U.S. Department of the Interior

Idaho-Maryland Mines Corp., undated, map of near-surface mine workings labeled “Surface, L-50”, scale 1 inch = 200 feet

Mac Boyle, Errol, December 1918, *Mines and Mineral Resources of Nevada County, Chapters of the State Mineralogists Report, Biennial Period 1917-1918*, California State Mining Bureau

Nelson Engineering, July 5, 2024, *Preliminary Site Plan for Jada Windows, APN 009-680-050 and -056, County of Nevada, California*

Portland Cement Association, *Concrete Floors on Ground*, 4th Edition, 2008

Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils*, 2012

Saucedo, G.J., and Wagner, D.L., 1992, *Geologic Map of the Chico Quadrangle*, California, 1:250,000. California Division of Mines and Geology, Regional Geologic Map 7A

Tuminas, A., 1983, *Geologic Map of the Grass Valley-Colfax Area, Nevada and Placer Counties, California*

United States Bureau of Land Management (BLM), Folsom Field Office, June 14, 1996. *Supplemental Master Title Plat, Sections 25, 26, 35, 36, Township 16 North, Range 8 East, Mount Diablo Meridian, California*

United States Department of Agriculture, April 2024, *Web Soil Survey*, Natural Resources Conservation Service, http:// websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx, accessed April 2024

United States Geological Survey (USGS), 2015, *Grass Valley Quadrangle, California*, 7.5-minute series

USGS, 1952, aerial photograph of Grass Valley area

USGS, 1950, 7.5-Minute topographic map of *Grass Valley, California*

USGS, 1947, aerial photograph of Grass Valley area

USGS, 1939, aerial photograph of Grass Valley area

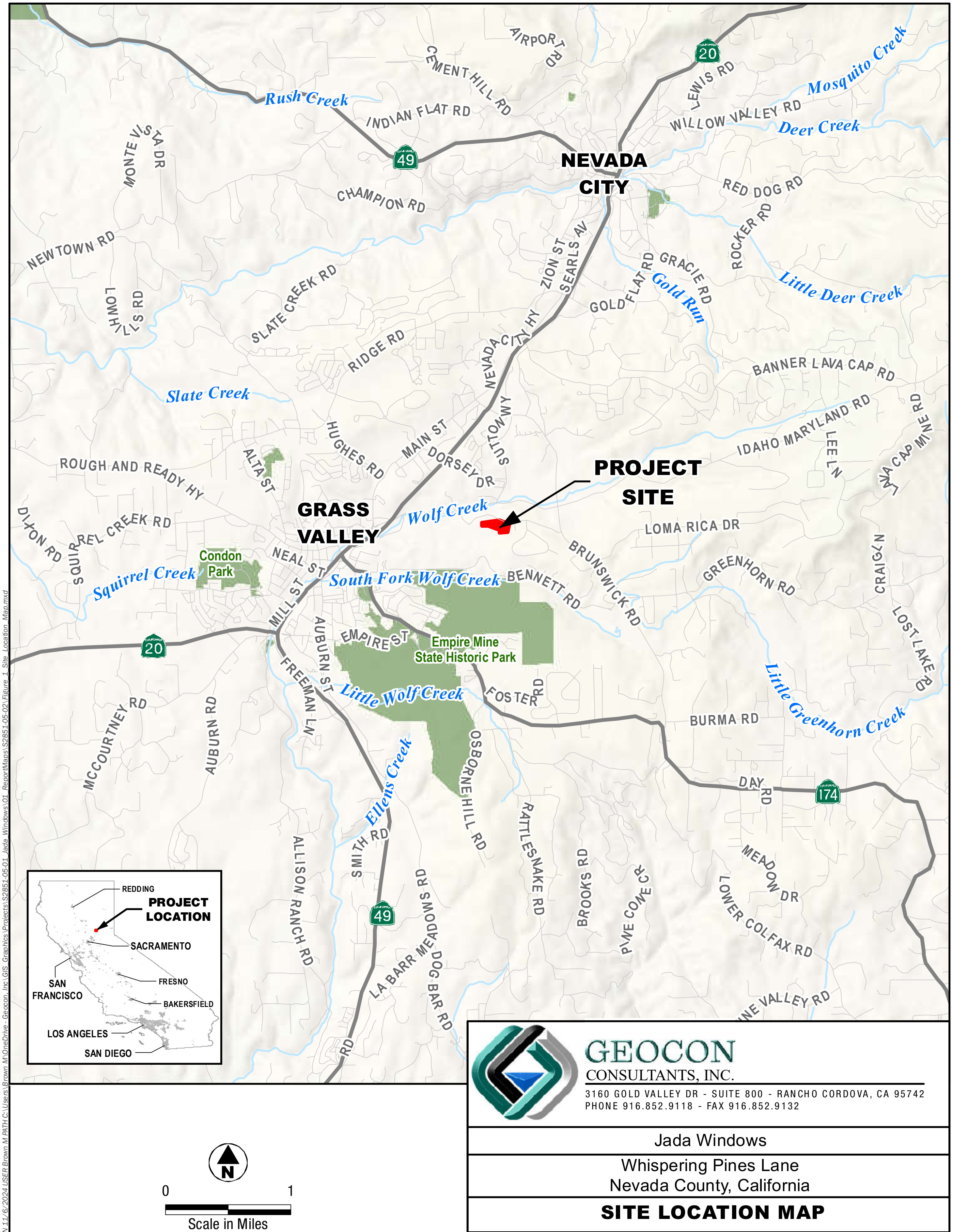
USGS, 1901, 7.5-Minute topographic map of *Grass Valley, California*

United States Surveyor General’s Office (USSGO), *Plat of the Claim of Idaho Maryland Mines Company*, Mineral Survey (MS) 5514-1

USSGO, October 1888, *Plat of the Claim of Joseph O’Keefe and James McCanna upon the South Idaho Consolidated Quartz Mine, Grass Valley Mining District, Nevada County, California*, MS 2781

USSGO, September 30, 1867, *Plat of the Schofield Gold Quartz Claim*, MS 30

Wire Reinforcement Institute / Concrete Reinforcing Steel Institute (WRI/CRSI), *Design of Slab-on-Ground Foundations*, March 1996



GEOCON CONSULTANTS, INC.
3160 GOLD VALLEY DR - SUITE 800 - RANCHO CORDOVA, CA 95742
PHONE 916.852.9118 - FAX 916.852.9132

Jada Windows

Whispering Pines Lane
Nevada County, California

SITE LOCATION MAP

S2851-05-02	December 2024	Figure 1

GEOCON 11/6/2024 USER Brown M PATH C:\Users\Brown M\OneDrive - Geocon, Inc\GIS_Graphics\Projects\S2851-05-01_Jada_Windows\01_ReportMaps\S2851-05-02\Figure_1_Site_Location_Map.mxd

Legend

T24-10	Approximate Exploratory Trench Location (Geocon, 2024)	
T-10	Approximate Exploratory Trench Location (Holdrege & Kull, 2010)	
	Historically Recorded Vertical Mine Shaft	
	Bedrock Outcropping	
	Proposed structure (Nelson Engineering, 2024)	
	Concrete Basin	
	Approximate Fill Area	
	Esitmated Boundary of Deep Undocumented Fill	
	Boulder Stockpile	
	Possible Fill Stockpile	
	Approximate Site Boundary	
	Inferred Geologic Contact	

LCmd Lake Combie massive diabase
LCsp Lake Combie serpentine

N

0 100
Scale in Feet

GEOCON
CONSULTANTS, INC.
3160 GOLD VALLEY DR - SUITE 800 - RANCHO CORDOVA, CA 95742
PHONE 916.852.9118 - FAX 916.852.9132

Jada Windows

Whispering Pines Lane
Nevada County, California

SITE PLAN

S2851-05-02	December 2024	Figure 2

APPENDIX A

Photo 1 – View of the site to the west from near the northwestern corner of the site. Whispering Pines Lane is visible to the right.

Photo 2 – Rock outcrop, approximately 10 feet above grade, in the central portion of the site.

Photo 3 – Resistant diabase rock outcrop, approximately 4 feet above grade, in the central portion of the site.

Photo 4 – Area of more resistant diabase rock outcrop in the central portion of the site.

Photo 5 – Typical diabase rock outcrop in the central portion of the site.

Photo 6 – Typical diabase rock outcrop in the central portion of the site.

Photo 7 – Typical diabase rock outcrop in the central portion of the site.

Photo 8 – View to the southwest of boulders, typically up to three feet in maximum dimension, stockpiled in the southeastern portion of the site during previous earthwork grading.

Photo 9 – View to the west of boulders, typically up to five feet in maximum dimension, stockpiled near the northwestern site boundary during previous earthwork grading.

Photo 10 – View to the west of the historically mapped Canyon shaft location in the western end of the site. The area is a shallow, densely vegetated depression. No mine rock or concrete was identified.

Photo 11 – View to the east of exploratory trench T24-01.

Photo 12 – Variably weathered diabase rock in exploratory trench T24-01.

Photo 13 – View to the southeast of exploratory trench T24-02.

Photo 14 – View to the west of exploratory trench T24-02.

Photo 15 – Variably weathered rock in exploratory trench T24-02.

Photo 16 – View to the west of T24-03.

Photo 17 – Variably weathered rock in exploratory trench T24-03.

Photo 18 – View to the north of exploratory trench T24-04.

Photo 19 – Engineered fill in exploratory trench T24-04.

Photo 20 – View to the west of exploratory trench T24-05.

Photo 21 – Engineered fill in exploratory trench T24-05.

Photo 22 – View to the south of exploratory trench T24-07.


Photo 23 – Engineered fill in exploratory trench T24-07.


Photo 24 – View to the south of exploratory trench T24-08.

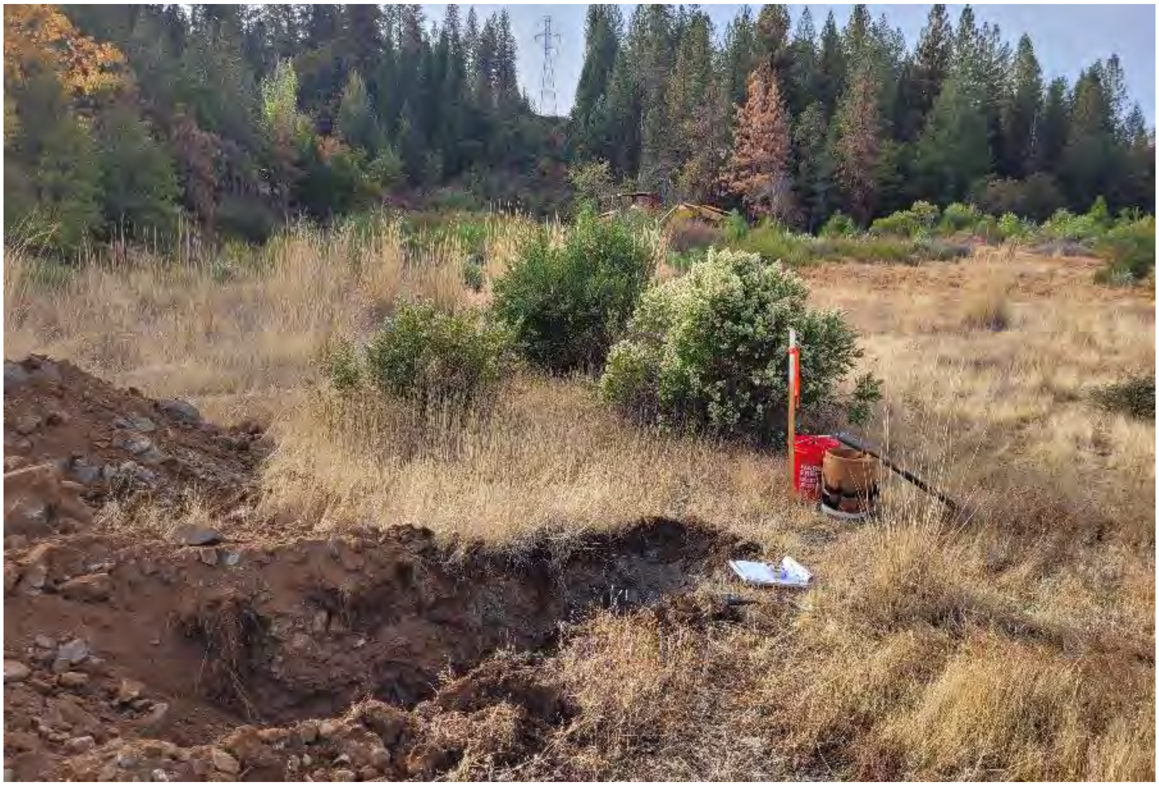


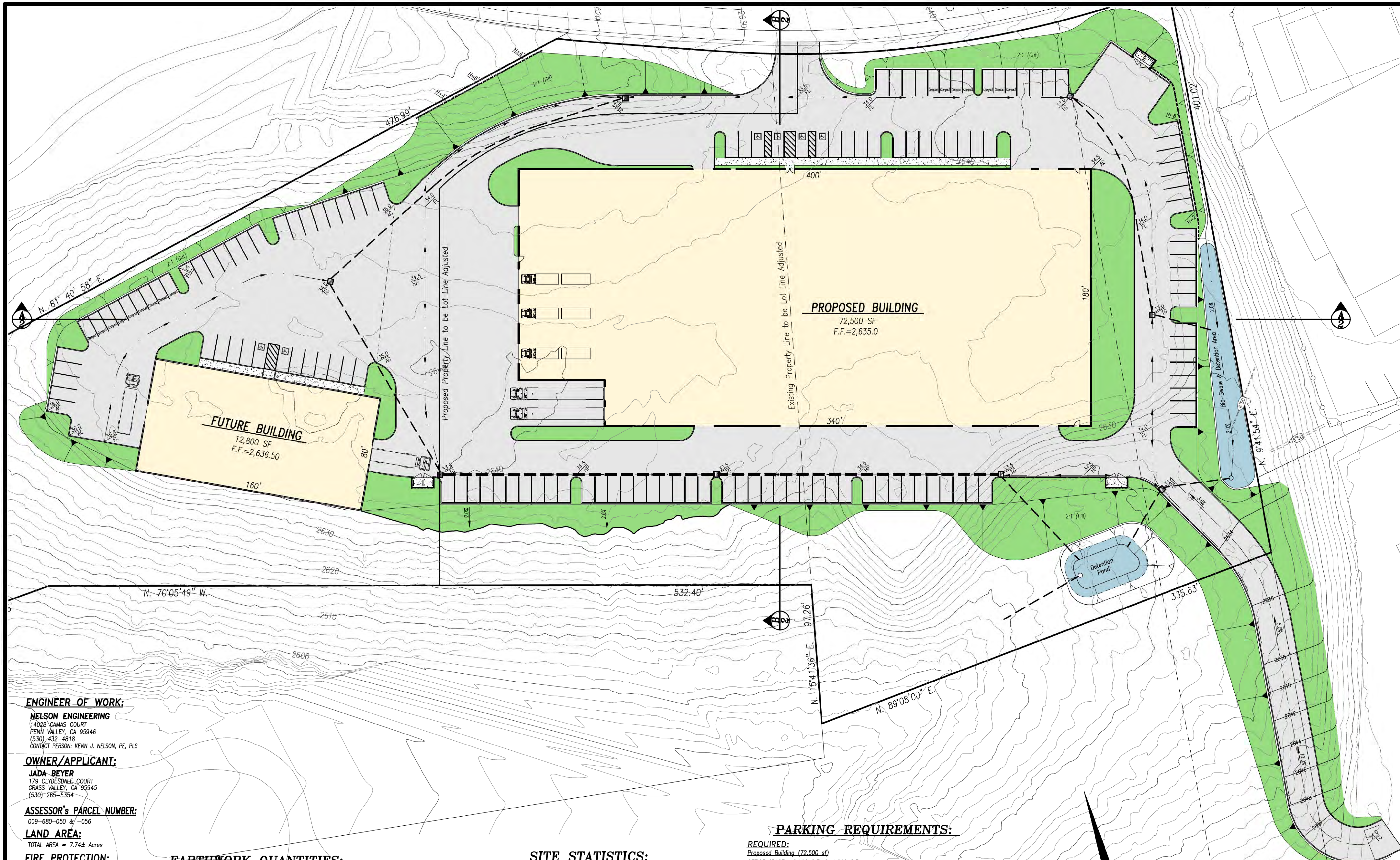
Photo 25 – View to the south of exploratory trench T24-09.

Photo 26 – Fill with serpentinite rock in exploratory trench T24-09.

Photo 27 – Ultramafic rock at the ground surface near exploratory trench T24-09.

Photo 28 – View to the south of exploratory trench T24-10.

APPENDIX B



ENGINEER OF WORK:

NELSON ENGINEERING
14028 CAMAS COURT
PENN VALLEY, CA 95946
(530) 432-4818
CONTACT PERSON: KEVIN J. NELSON, PE, PLS

OWNER/APPLICANT:

JADA BEYER
179 CLYDESDALE COURT
GRASS VALLEY, CA 95945
(530) 265-5354

ASSESSOR's PARCEL NUMBER:

009-680-050 & -056

LAND AREA:

TOTAL AREA = 7.74± Acres

FIRE PROTECTION:

CITY OF GRASS VALLEY FIRE DISTRICT

WATER:

NEVADA IRRIGATION DISTRICT

SEWER DISPOSAL:

CITY OF GRASS VALLEY

ELECTRICAL UTILITIES:

PACIFIC GAS & ELECTRIC

ZONING:

LIGHT INDUSTRIAL, M-1

EARTHWORK QUANTITIES:

NOTE TO CONTRACTOR: THE CALCULATION OF EARTHWORK QUANTITIES, AND THE DETERMINATION OF ANY REQUIRED IMPORT OR EXPORT SHALL BE THE SOLE RESPONSIBILITY OF THE CONTRACTOR. THE FOLLOWING QUANTITIES CALCULATED BY NELSON ENGINEERING ARE FOR FEE CALCULATION.

EXCAVATION	=	14,000	CY
FILL (10% SHRINK)	=	14,000	CY
EXPORT	=	0	CY

SITE STATISTICS:

DESCRIPTION	SQUARE FOOTAGE	PERCENTAGE
BUILDING COVERAGE (Proposed)	69,884 s.f.	20.7%
BUILDING COVERAGE (Future)	12,800 s.f.	3.8%
PAVEMENT AREA	97,761 s.f.	29.0%
LANDSCAPE AREA per Sec. 17.34		
INTERIOR PARKING	9,052 s.f.	2.7%
STREET BUFFER	45,535 s.f.	13.5%
NATURAL AREA/OPEN SPACE per Sec. 17.34	102,051 s.f.	30.3%
TOTALS:	337,083 s.f.	100.0%

PARKING REQUIREMENTS:

REQUIRED:

Proposed Building (72,500 sf)

OFFICE SPACE: 2,000 S.F. @ 1:200 S.F.	= 10 STALLS
PRODUCT ASSEMBLY, RECEIVING, LOADING & PACKAGING SPACE: 40,000 S.F. @ 1:500 S.F.	= 80 STALLS
WAREHOUSE & STORAGE SPACE: 27,884 S.F. @ 1:5,000 S.F.	= 6 STALLS

Future Building (12,800 sf)

OFFICE SPACE: 1,600 S.F. @ 1:200 S.F.	= 8 STALLS
PRODUCT ASSEMBLY, RECEIVING, LOADING & PACKAGING SPACE: 8,000 S.F. @ 1:500 S.F.	= 16 STALLS
WAREHOUSE & STORAGE SPACE: 3,200 S.F. @ 1:5,000 S.F.	= 1 STALLS
TOTAL PARKING SPACES REQUIRED	= 121 STALLS

	REQUIRED:	PROPOSED:
FULL SIZE PARKING STALLS	116	114
COMPACT PARKING STALLS	0	16
HANDICAP STALLS	5	6
TOTALS:	121	136

SCALE 1 INCH = 30 FEET
30 0 30 60

N

NO.	REVISIONS	DATE

DESIGNED: KJN
DRAWN: KJN
CHECKED BY: KJN
DATE: July 5, 2024
PROJECT No.: 21-242
DWG. NAME: 21-242 Site Plan-4.dwg

PRELIMINARY SITE PLAN FOR:

JADA WINDOWS

A.P.N. 009-680-050 & -056

COUNTY OF NEVADA, CALIFORNIA

NELSON ENGINEERING
Civil Engineering, Surveying & Land Planning
14028 Camas Court
Penn Valley, CA 95946
(530) 432-4818
www.nelsonengineer.com

REGISTERED PROFESSIONAL ENGINEER
KEVIN J. NELSON
No. 55101
Exp. 6-30-2022
CIVIL
STATE OF CALIFORNIA
Draft Only
Not for
Construction

1 OF 3

APPENDIX C

GEOCON

TRENCH NUMBER: T24-01

PROJECT NAME Jada Windows
PROJECT NUMBER S2851-05-02
DATE STARTED 11/05/2024 **COMPLETED** 11/05/2024
LATITUDE / LONGITUDE 39.2221, -121.0363
CONTRACTOR C&D Contractors, Inc.
EQUIPMENT CAT 420D
METHOD Backhoe W/2-Foot Bucket
LOCATION -
LOGGED BY J. Muir
DEPTH 6.5' **SURFACE ELEVATION** ~2636'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description
	2636				
	2635			Rx	Completely to highly weathered, **Diabase Rock**; excavates as Sandy SILT (ML) to Silty SAND (SM), dense, brownish yellow (7.5YR 6/6) with abundant rock, gravel to boulder size, 36" and greater
2					
4					
6	2630				

TRENCH TERMINATED AT 6.5 FEET

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

TRENCH NUMBER: T24-02

PROJECT NAME Jada Windows
PROJECT NUMBER S2851-05-02
DATE STARTED 11/05/2024 **COMPLETED** 11/05/2024
LATITUDE / LONGITUDE 39.2219, -121.035656
CONTRACTOR C&D Contractors, Inc.
EQUIPMENT CAT 420D
METHOD Backhoe W/2-Foot Bucket
LOCATION -
LOGGED BY J. Muir
DEPTH 6.5' **SURFACE ELEVATION** ~2638'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	2638			Rx	Completely to moderately weathered, **Diabase Rock**; excavates as Sandy SILT (ML) to Silty SAND (SM), dense, brownish yellow (7.5YR 6/6) with abundant gravel to boulder size angular rock to 36" diameter and greater	X		SS-02 -0.0
2								
	2635				-Zones of more resistant rock may require splitting or grinding			
4								
6								

TRENCH TERMINATED AT 6.5 FEET
Refusal of backhoe on large weathered rock

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

TRENCH NUMBER: T24-03

GEOCON

PROJECT NAME	Jada Windows	**PROJECT NUMBER**	S2851-05-02
DATE STARTED	11/05/2024 **COMPLETED** 11/05/2024	**LATITUDE / LONGITUDE**	39.2218, -121.0354
CONTRACTOR	C&D Contractors, Inc.	**EQUIPMENT**	CAT 420D
METHOD	Backhoe W/2-Foot Bucket	**LOCATION**	-
LOGGED BY	J. Muir	**DEPTH**	8' **SURFACE ELEVATION** ~2639'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	2639							
0				Rx	Completely to moderately weathered, **Diabase Rock**; excavates as Sandy SILT (ML) to Silty SAND (SM), dense, brownish yellow (7.5YR 6/6) with abundant gravel to boulder size angular rock to 36" diameter and greater	X		SS-03 -0.0
1						X		CB-03 -01
2								
4	2635							
6								
8					TRENCH TERMINATED AT 8 FEET			

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

DRAFT

GEOCON

TRENCH NUMBER: T24-04

PROJECT NAME	Jada Windows	**PROJECT NUMBER**	S2851-05-02
DATE STARTED	11/05/2024 **COMPLETED** 11/05/2024	**LATITUDE / LONGITUDE**	39.2217, -121.0348
CONTRACTOR	C&D Contractors, Inc.	**EQUIPMENT**	CAT 420D
METHOD	Backhoe W/2-Foot Bucket	**LOCATION**	-
LOGGED BY	J. Muir	**DEPTH**	7.5' **SURFACE ELEVATION** ~2637'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	2637							
0				ML/MH	**FILL** Medium dense, damp, red (2.5YR 4/8) to strong brown (7.5YR 5/8), **Sandy SILT**, with clay	X		SS-04 -0.0
2	2635					X	■	ST-04 -02
						X		CB-04 -01
4						X	■	ST-04 -04
6						X	■	ST-04 -06
	2630							

TRENCH TERMINATED AT 7.5 FEET

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

GEOCON

TRENCH NUMBER: T24-05

PROJECT NAME	Jada Windows	**PROJECT NUMBER**	S2851-05-02
DATE STARTED	11/05/2024 **COMPLETED** 11/05/2024	**LATITUDE / LONGITUDE**	39.2222, -121.035676
CONTRACTOR	C&D Contractors, Inc.	**EQUIPMENT**	CAT 420D
METHOD	Backhoe W/2-Foot Bucket	**LOCATION**	-
LOGGED BY	J. Muir	**DEPTH**	5' **SURFACE ELEVATION** ~2634'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	2634							
0				ML/MH	**FILL** Medium dense, damp, strong brown (7.5YR 5/6), **Sandy SILT**, with clay	X		SS-05 -0.0
2							■	ST-05 -02
4	2630						■	ST-05 -04

TRENCH TERMINATED AT 5 FEET IN FILL

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

GEOCON

TRENCH NUMBER: T24-06

PROJECT NAME Jada Windows
PROJECT NUMBER S2851-05-02
DATE STARTED 11/05/2024 **COMPLETED** 11/05/2024
LATITUDE / LONGITUDE 39.2219, -121.0365
CONTRACTOR C&D Contractors, Inc.
EQUIPMENT CAT 420D
METHOD Backhoe W/2-Foot Bucket
LOCATION -
LOGGED BY J. Muir
DEPTH 5' **SURFACE ELEVATION** ~2636'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	2636							
	2635			ML/MH	**FILL** Medium dense, damp, strong brown (7.5YR 5/6), **Sandy SILT**, with clay; dry and loose upper 18"	X		SS-06 -0.0
2							■	ST-06 -02
4				ML	**NATIVE** Firm, damp, strong brown (7.5YR 5/6), **Clayey SILT**; grades to completely weathered at Diabase Rock			

TRENCH TERMINATED AT 5 FEET

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

TRENCH NUMBER: T24-07

PROJECT NAME	Jada Windows	**PROJECT NUMBER**	S2851-05-02
DATE STARTED	11/05/2024 **COMPLETED** 11/05/2024	**LATITUDE / LONGITUDE**	39.221367, -121.034306
CONTRACTOR	C&D Contractors, Inc.	**EQUIPMENT**	CAT 420D
METHOD	Backhoe W/2-Foot Bucket	**LOCATION**	-
LOGGED BY	J. Muir	**DEPTH**	8' **SURFACE ELEVATION** ~2636'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	2636			ML/MH	**FILL** Medium dense, damp, strong brown (7.5YR 5/6), **Sandy SILT**, with clay	X		SS-07 -0.0
	2635							
2						X	■	ST-07 -02
								CB-07 -01
4							■	ST-07 -04
6	2630						■	ST-07 -06
8					TRENCH TERMINATED AT 8 FEET IN FILL			

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

TRENCH NUMBER: T24-08

GEOCON

PROJECT NAME	Jada Windows	**PROJECT NUMBER**	S2851-05-02
DATE STARTED	11/05/2024 **COMPLETED** 11/05/2024	**LATITUDE / LONGITUDE**	39.2219, -121.0343
CONTRACTOR	C&D Contractors, Inc.	**EQUIPMENT**	CAT 420D
METHOD	Backhoe W/2-Foot Bucket	**LOCATION**	-
LOGGED BY	J. Muir	**DEPTH**	3' **SURFACE ELEVATION** ~2641'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description
	2641				
	2640			ML	**FILL** Loose, dry, brown (7.5YR 4/4), **Sandy SILT**, with angular gravel to cobble size rock and variable wood debris
2					

TRENCH TERMINATED AT 3 FEET AT BASE OF STOCKPILE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

GEOCON

TRENCH NUMBER: T24-09

PROJECT NAME Jada Windows **PROJECT NUMBER** S2851-05-02

DATE STARTED 11/05/2024 **COMPLETED** 11/05/2024 **LATITUDE / LONGITUDE** 39.2218, -121.0341

CONTRACTOR C&D Contractors, Inc. **EQUIPMENT** CAT 420D

METHOD Backhoe W/2-Foot Bucket **LOCATION** -

LOGGED BY J. Muir **DEPTH** 4' **SURFACE ELEVATION** ~2639'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description
	2639				
				SM	**FILL** Dense, dry, light gray (GLEY 1 7/N), **Silty SAND**, with abundant angular cobble and gravel to 12" diameter
2				SM	**FILL** Medium dense, dry, yellowish brown (10YR 6/6), **Silty SAND**, with gravel and cobble, angular rock to 12" diameter
4	2635				

TRENCH TERMINATED AT 4 FEET IN FILL

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

TRENCH NUMBER: T24-10

PROJECT NAME	Jada Windows	**PROJECT NUMBER**	S2851-05-02
DATE STARTED	11/05/2024 **COMPLETED** 11/05/2024	**LATITUDE / LONGITUDE**	39.221078, -121.033967
CONTRACTOR	C&D Contractors, Inc.	**EQUIPMENT**	CAT 420D
METHOD	Backhoe W/2-Foot Bucket	**LOCATION**	-
LOGGED BY	J. Muir	**DEPTH**	7' **SURFACE ELEVATION** ~2624'

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	2624							
0				ML	Soft, dry, brown (7.5YR 4/4), **Clayey SILT**			
2				CH	Firm, moist, olive (2.5Y 5/6), **CLAY**	X		CB-10 -01
4	2620			ML	Firm, damp, light olive gray (5Y 6/2), **Clayey SILT**, with sand			
6								

TRENCH TERMINATED AT 7 FEET

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN APPLIES ONLY TO THE SPECIFIC BORING OR TRENCH LOCATION AT THE DATE INDICATED AND MIGHT NOT REPRESENT SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. THE STRATIGRAPHY PRESENTED REPRESENTS THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES; THESE TRANSITIONS COULD BE GRADUAL.

APPENDIX D

GEOCON

2024 Aerial Photograph, Google Earth, Imagery Date August 12, 2024
Approximate Scale: 1 inch = 200 feet

1939 Aerial Photograph, USGS
Approximate Scale: 1 inch = 200 feet

GEOCON

1947 Aerial Photograph, USGS
Approximate Scale: 1 inch = 200 feet

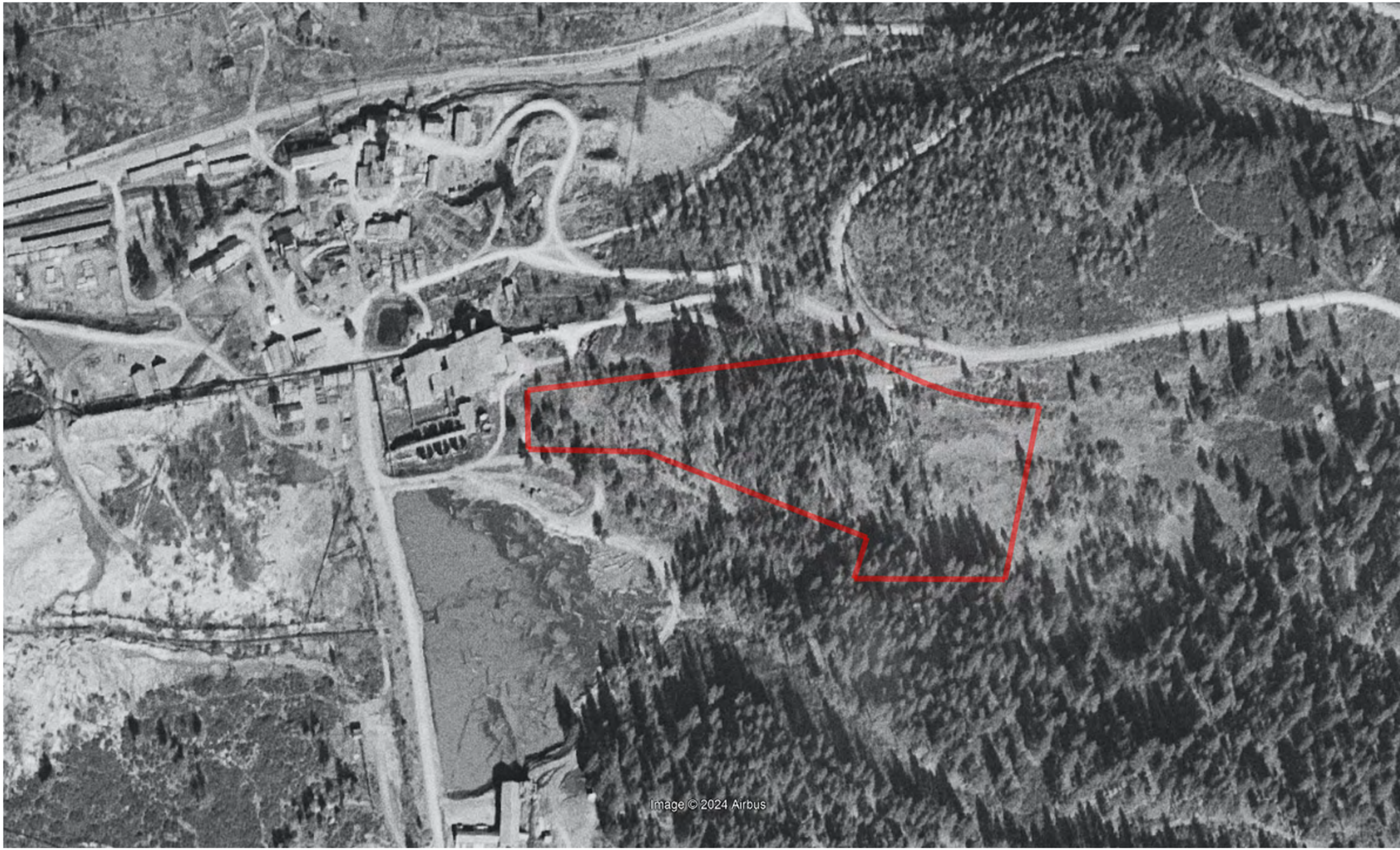
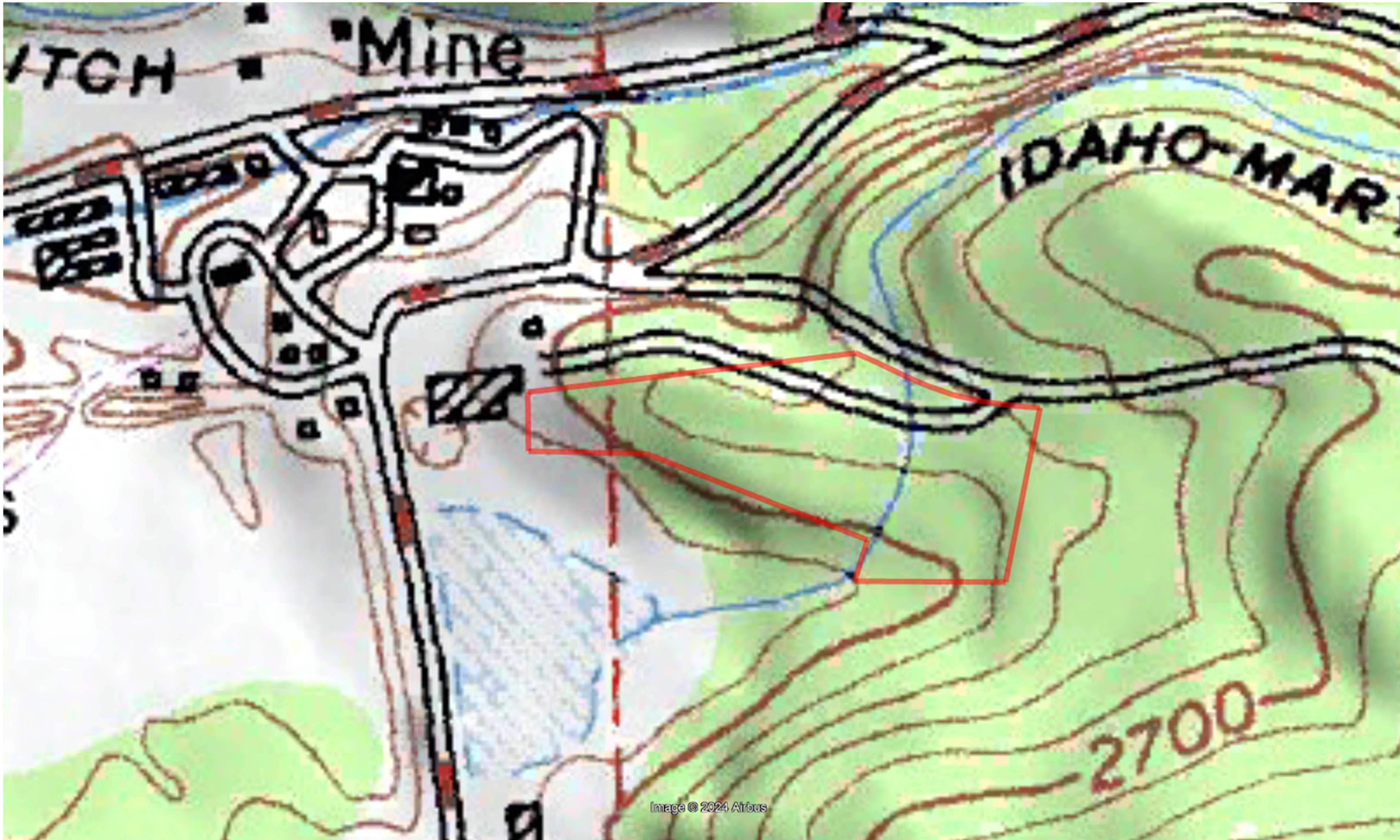
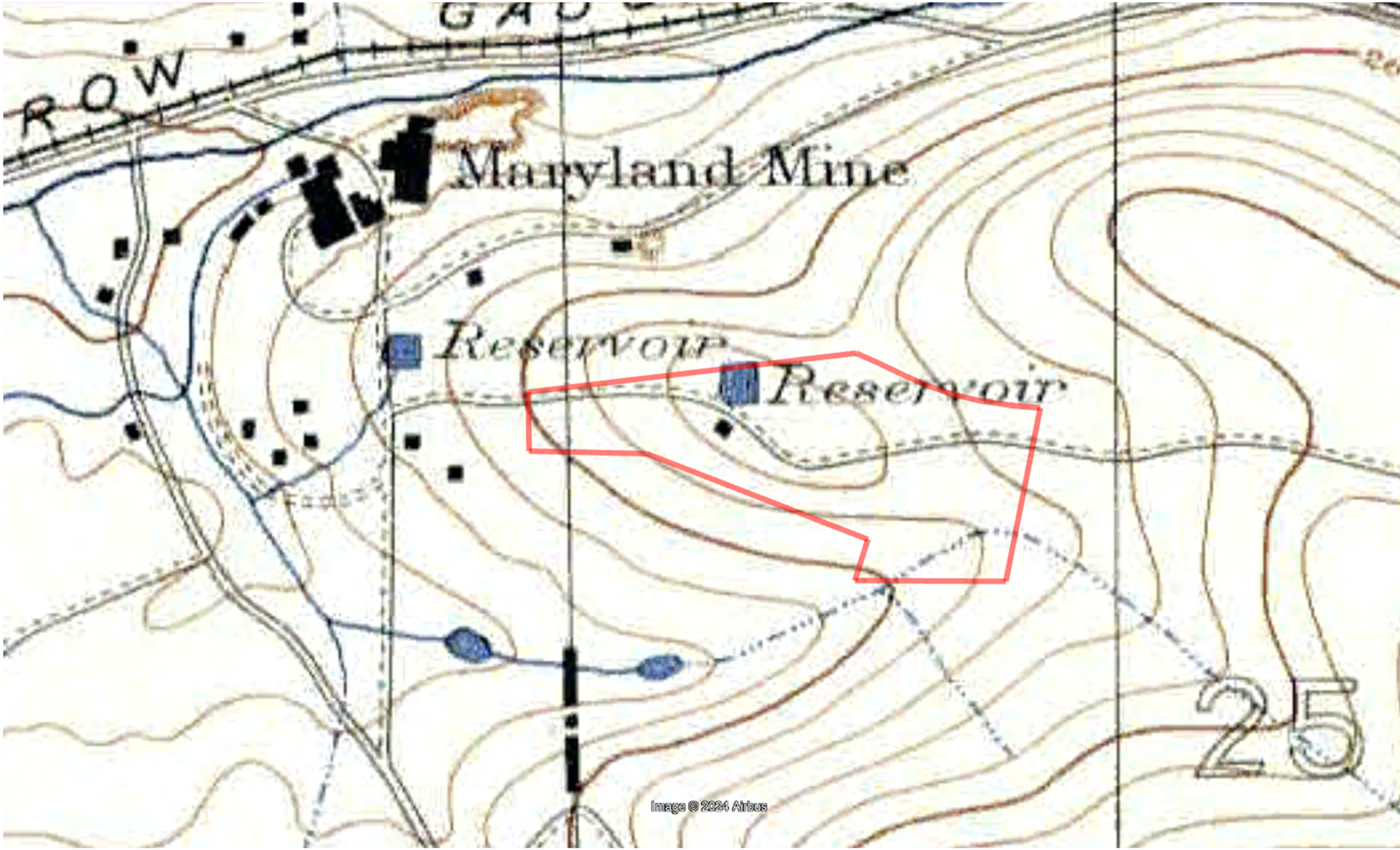
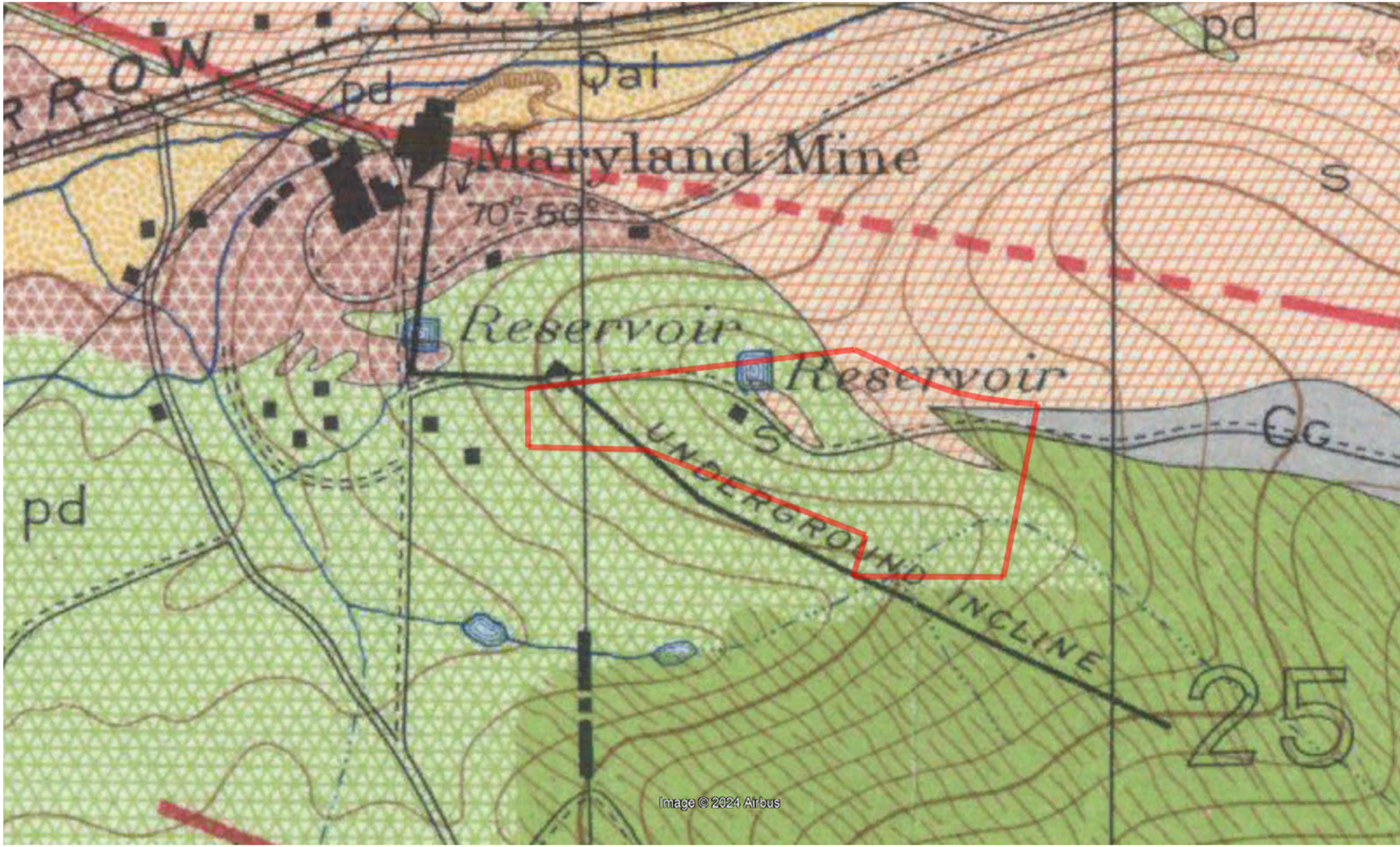
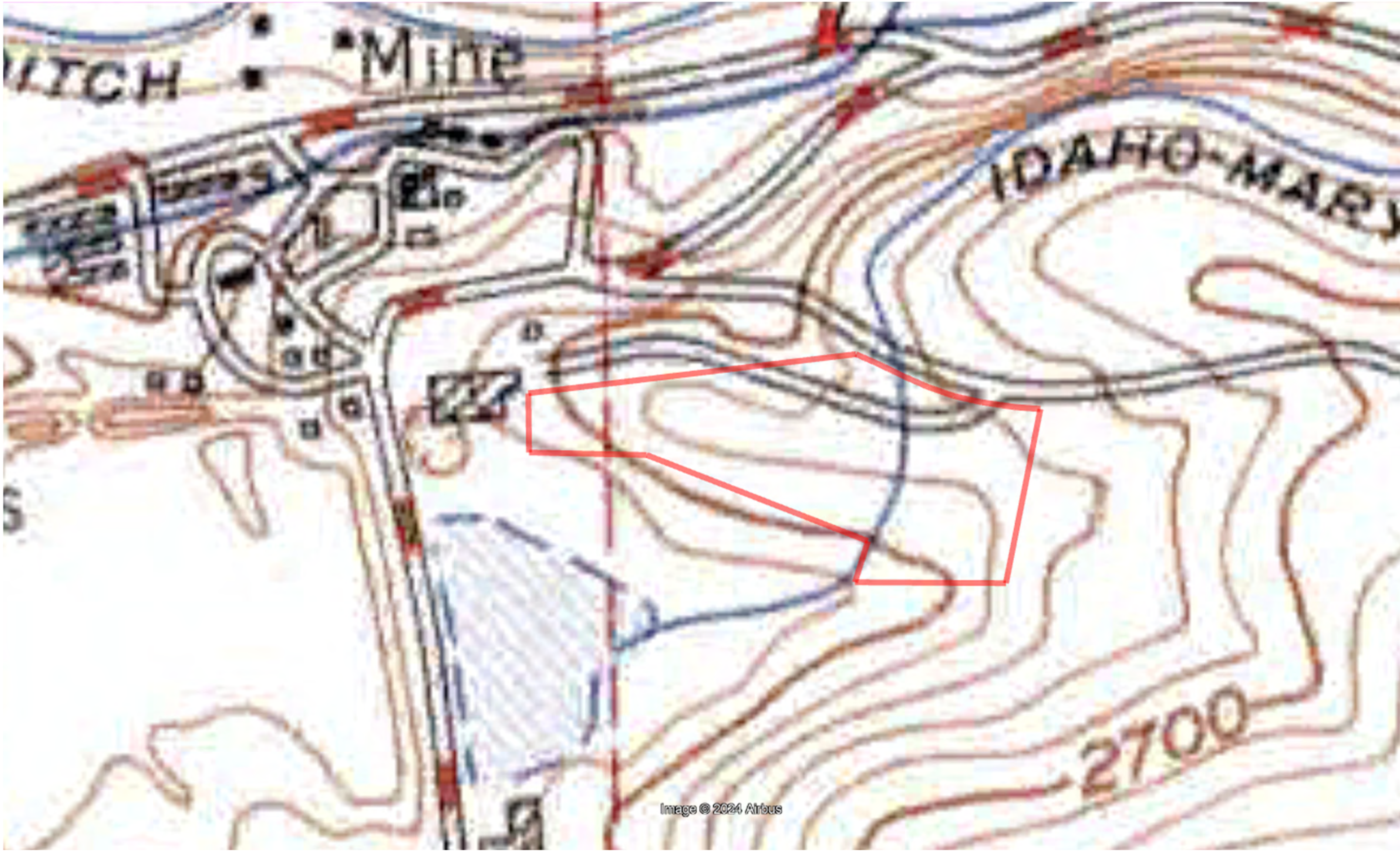


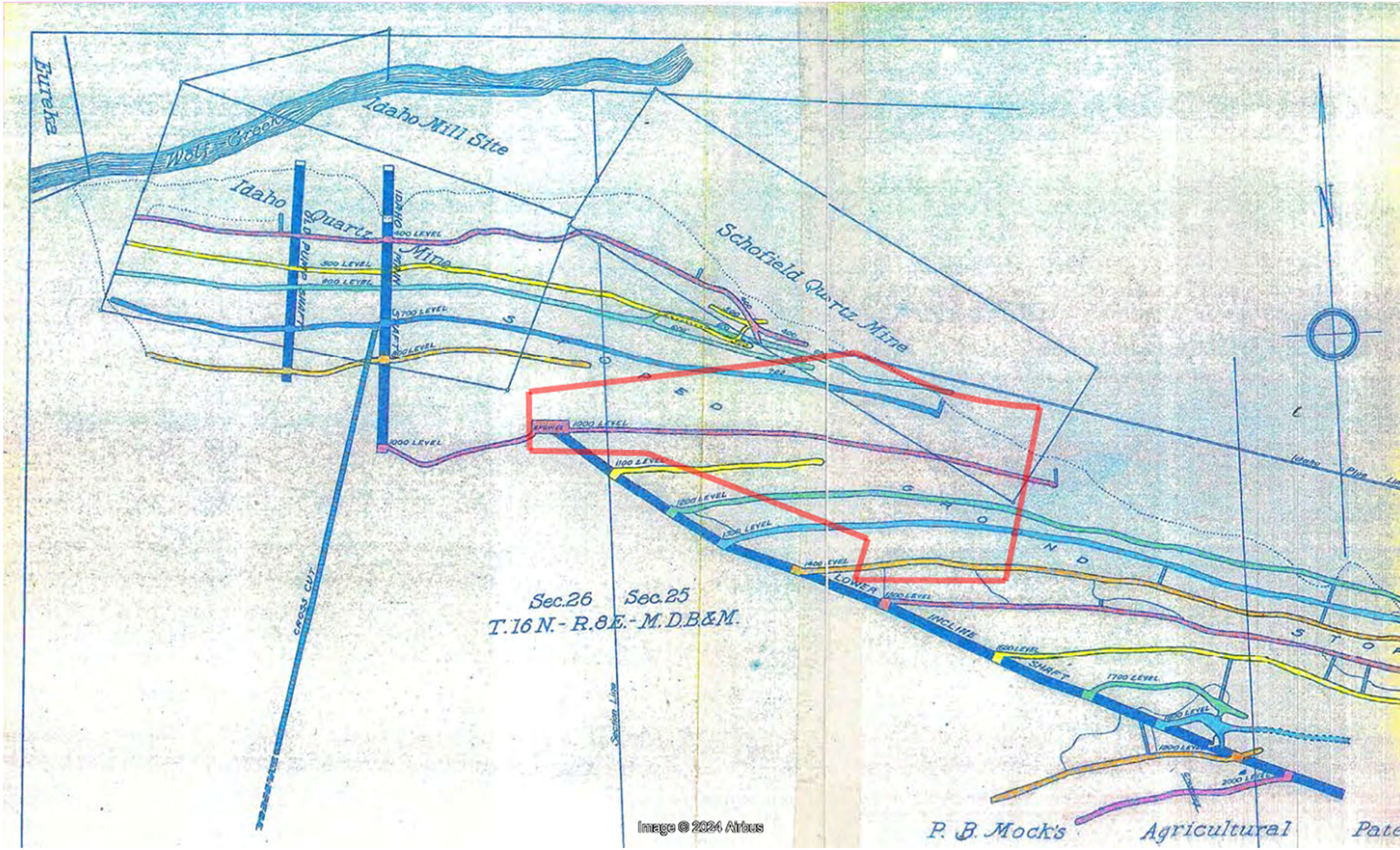

Image © 2024 Airbus

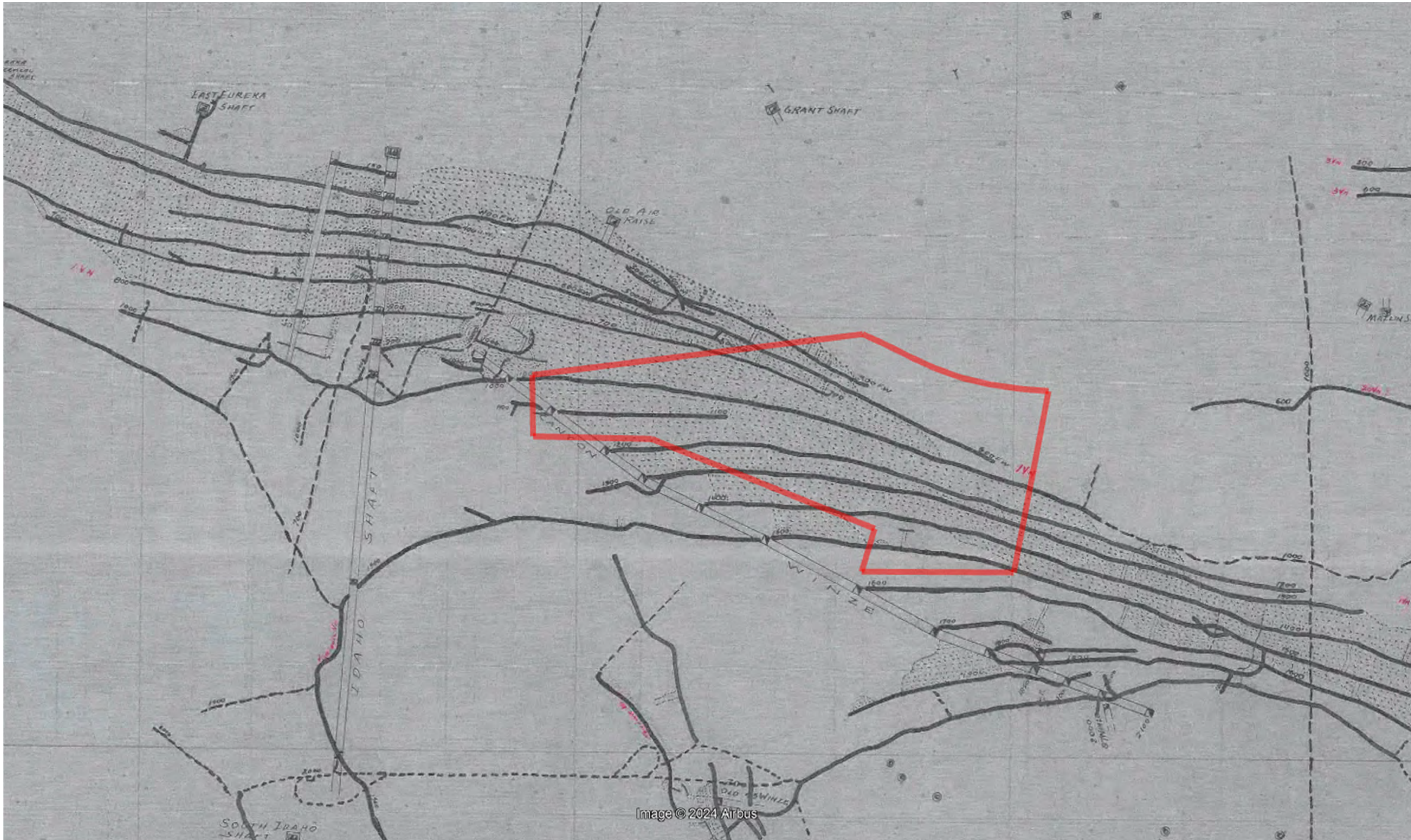

ITCH
Mine
IDAHO-MARY
2700
Image © 2024 Airbus


ROW
Maryland Mine
Reservoir
Reservoir
25
Image © 2024 Airbus

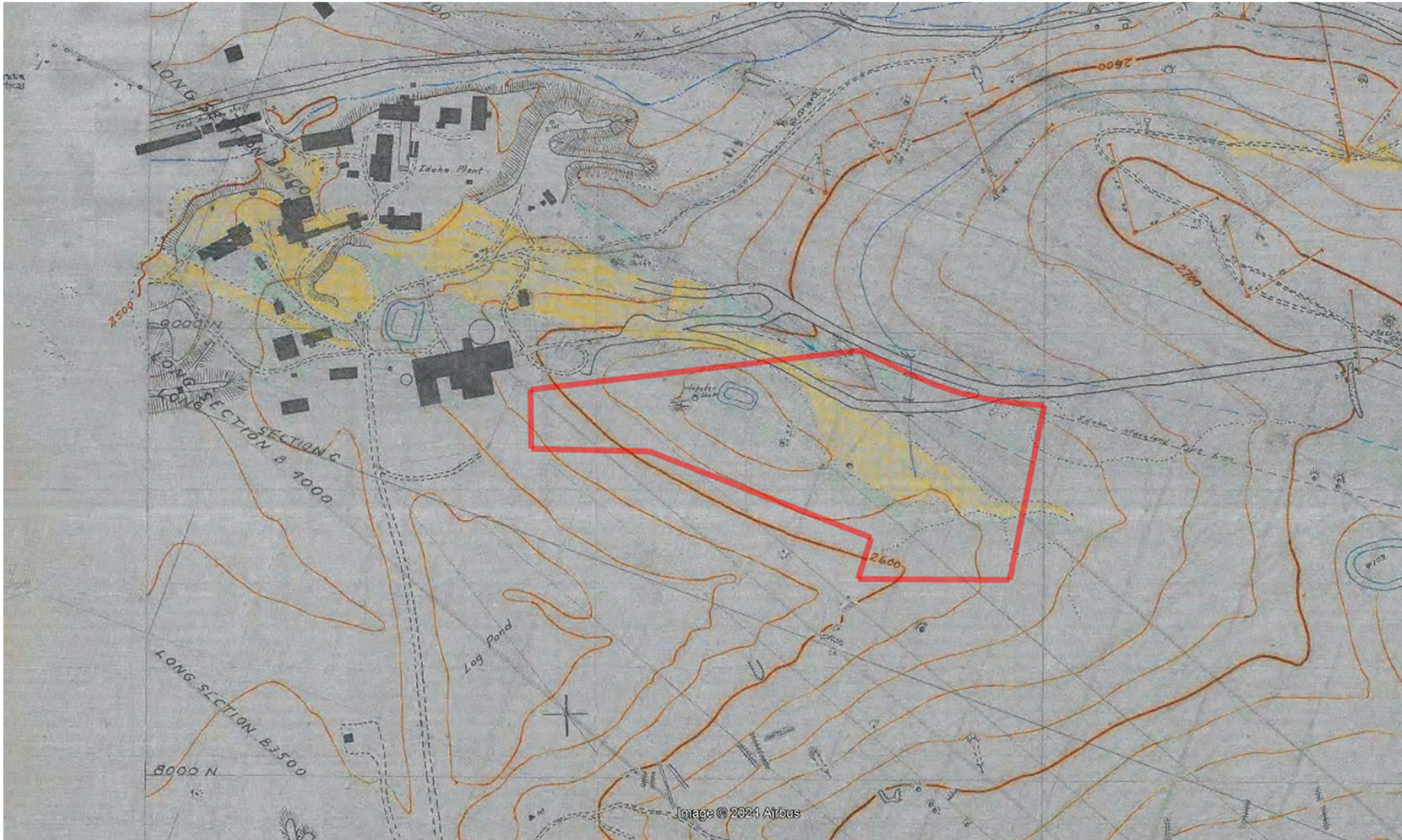

Maryland Mine
Reservoir
Reservoir
UNDERGROUND INCLINE
Qal
pd
pd
pd
S
S
Cg
70°-50°
25
Image © 2024 Airbus

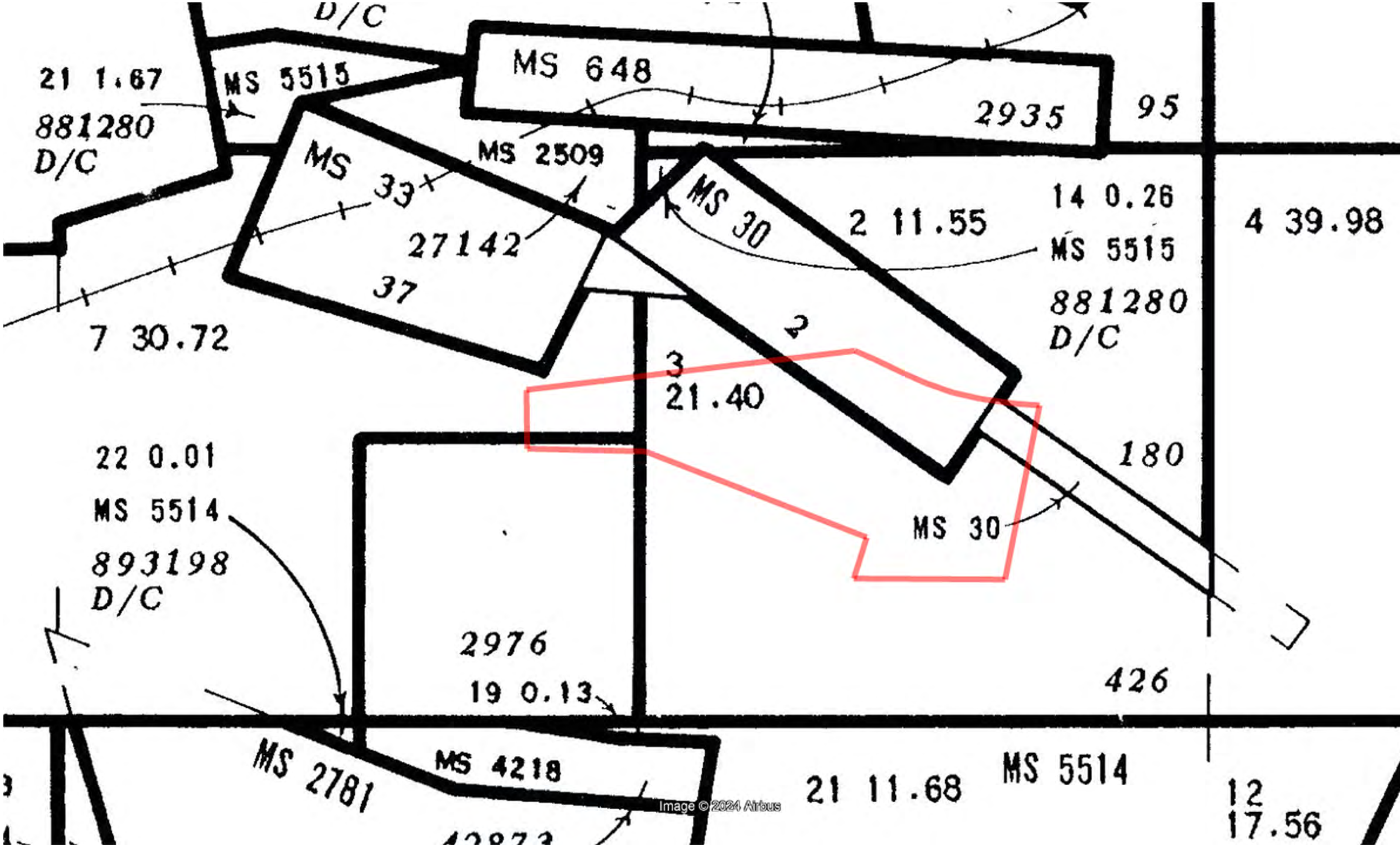

ITCH
Mine
IDAHO-MARY
2700
Image © 2024 Airbus

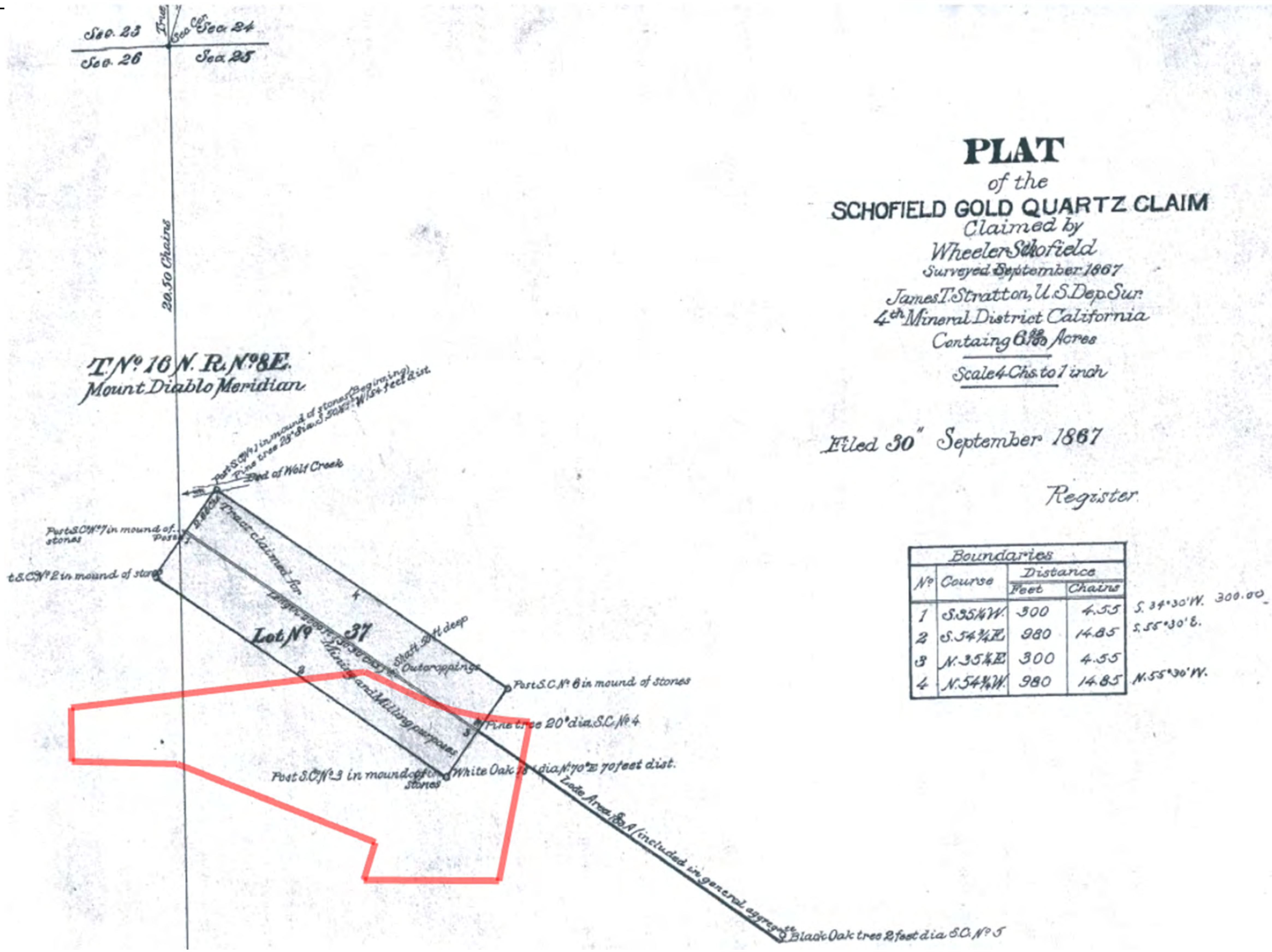

Eureka
Wolf Creek
Idaho Mill Site
Idaho Quartz Mine
Schofield Quartz Mine
OLD PUMP SHAFT
IDAHO MAIN SHAFT
400 LEVEL
500 LEVEL
600 LEVEL
700 LEVEL
800 LEVEL
1000 LEVEL
1100 LEVEL
1200 LEVEL
1300 LEVEL
1400 LEVEL
1600 LEVEL
1700 LEVEL
1800 LEVEL
1900 LEVEL
2000 LEVEL
LOWER INCLINE SHAFT
CROSS CUT
Section Line
Sec.26 Sec.25
T.16 N.-R.8 E.-M.D.B.&M.
Idaho Pipe Line
N
P. B. Mock's Agricultural Pater
Image © 2024 Airbus


EAST EUREKA SHAFT
GRANT SHAFT
OLD AIR RAISE
IDAHO SHAFT
WINZE
SOUTH IDAHO SHAFT
OLD WINZE
Image © 2024 Airbus

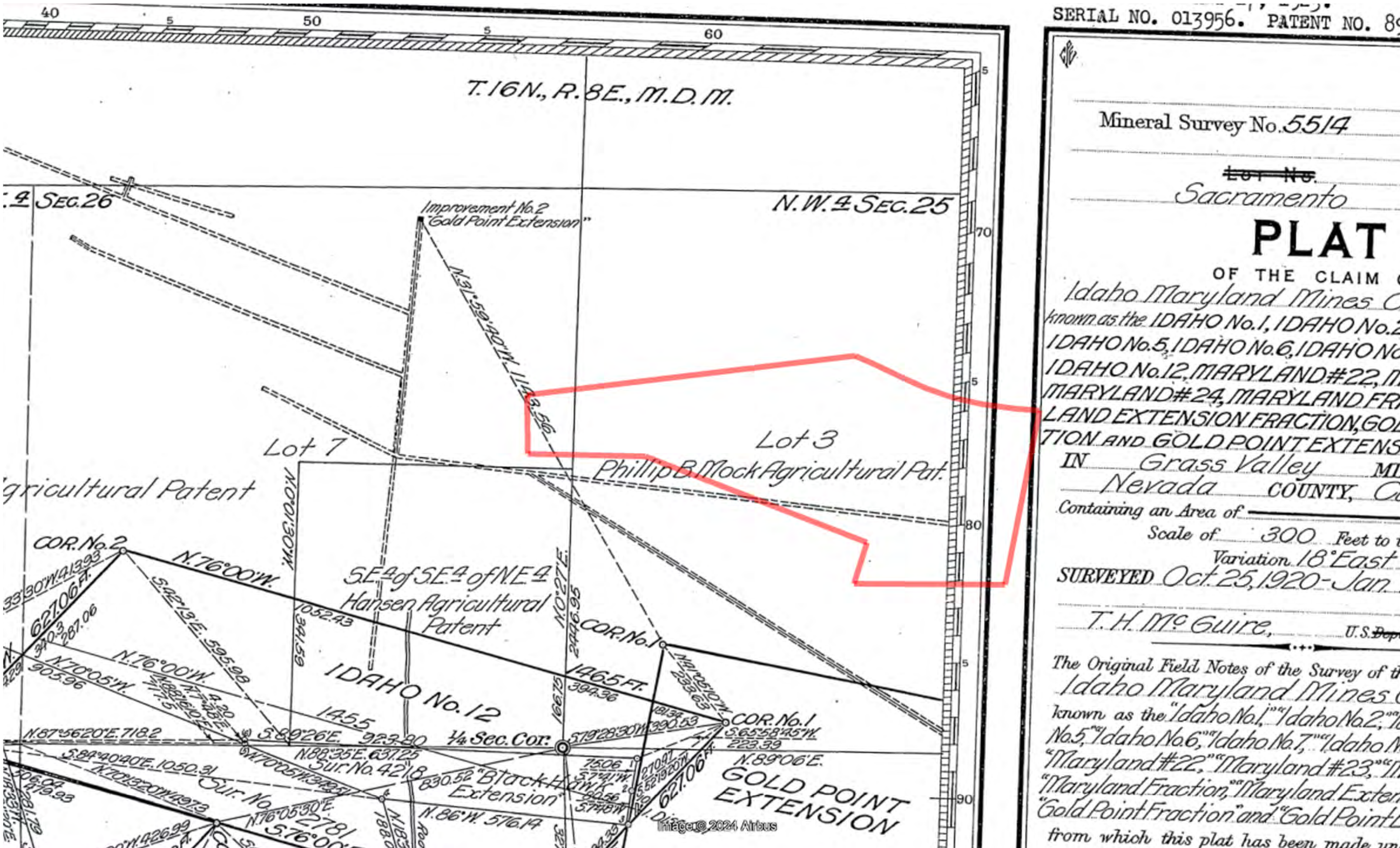
Map of near-surface workings labeled "Surface, L-50," Idaho Maryland Mine Corporation, Undated
Approximate Scale: 1 inch = 200 feet




D/C
MS 5515
MS 648
2935
95
21 1.67
881280
D/C
MS 33
37
MS 2509
27142
MS 30
2
2 11.55
14 0.26
MS 5515
881280
D/C
4 39.98
7 30.72
3
21.40
180
22 0.01
MS 5514
893198
D/C
MS 30
2976
19 0.13
426
MS 2781
MS 4218
21 11.68
MS 5514
12
17.56
Image © 2024 Airbus


Sec. 23
Sec. 24
Sec. 26
Sec. 25
20.50 Chains
T. Nº 16 N. R. Nº 8 E.
Mount Diablo Meridian
Bed of Wolf Creek
Post S.C. Nº 7 in mound of stones
Lot Nº 37
Tract claimed for Mining and Milling purposes
Outcroppings
Post S.C. Nº 6 in mound of stones
Pine tree 20" dia. S.C. Nº 4
Post S.C. Nº 3 in mound of stones
Lode Area Nº A (included in general aggregate)
Black Oak tree 2 feet dia. S.C. Nº 5
PLAT
of the
SCHOFIELD GOLD QUARTZ CLAIM
Claimed by
Wheeler Schofield
Surveyed September 1867
James T. Stratton, U.S. Dep. Sur.
4th Mineral District California
Scale 4 Chs. to 1 inch
Filed 30" September 1867
Register
Boundaries
Nº | Course | Distance Feet | Distance Chains
1 | S. 35¼ W. | 300 | 4.55
2 | S. 54¾ E. | 980 | 14.85
3 | N. 35¼ E. | 300 | 4.55
4 | N. 54¾ W. | 980 | 14.85
S. 34°30' W. 300.00
S. 55°30' E.
N. 55°30' W.


T.16N., R.8E.
Mount Diablo Meridian.
P.B.Mocks Agricultural Claim.
Agricultural Claim.
"N.W.Cor." Locn. Cor.
Sec. 26
Sec. 25
South Section Line.
Black Hawk Quartz Claim.
LODE LINE
West 11.38 chs.
West 8.62 chs. or 568.9 ft.
N.69¼°W. 15.92 Chs. or 1050.7 ft.
Image © 2024 Airbus


SERIAL NO. 013956. PATENT NO. 89
T. 16 N., R. 8 E., M.D.M.
Sec. 26
N.W. ¼ Sec. 25
Improvement No. 2 "Gold Point Extension"
Lot 7
Lot 3
Phillip B. Mock Agricultural Pat.
Agricultural Patent
SE ¼ of SE ¼ of N.E ¼ Hansen Agricultural Patent
COR. No. 2
COR. No. 1
N.76°00'W.
IDAHO No. 12
¼ Sec. Cor.
"Black Hawk Extension"
Sur. No. 4218
Sur. No. 2781
GOLD POINT EXTENSION
Image © 2024 Airbus
Mineral Survey No. 5514
Sacramento
PLAT
OF THE CLAIM O
Idaho Maryland Mines Co
known as the IDAHO No. 1, IDAHO No. 2,
IDAHO No. 5, IDAHO No. 6, IDAHO No.
IDAHO No. 12, MARYLAND #22, MA
MARYLAND #24, MARYLAND FRA
LAND EXTENSION FRACTION, GOL
TION AND GOLD POINT EXTENSI
IN Grass Valley MIN
Nevada COUNTY, Ca
Containing an Area of
Scale of 300 Feet to th
Variation 18° East
SURVEYED Oct. 25, 1920 - Jan. 7
T. H. McGuire, U.S.
The Original Field Notes of the Survey of the
Idaho Maryland Mines C
known as the "Idaho No. 1," "Idaho No. 2," "Id
No. 5," "Idaho No. 6," "Idaho No. 7," "Idaho No
"Maryland #22," "Maryland #23," "Ma
"Maryland Fraction," "Maryland Extens
"Gold Point Fraction" and "Gold Point E
from which this plat has been made unde

APPENDIX E

Sample ID	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Water Content (%)	Dry Density (pcf)
CB-04-01 (1-7')	1-7				---			
ST-04-02 (2')	2				---		18.8	78.5
ST-04-04 (4')	4				---		18.8	73.3
ST-04-06 (6')	6				---		35.1	71.9
ST-05-02 (2')	2				---		17.8	70.5
ST-05-04 (4')	4				---		18.4	90.6
ST-06-02 (2')	2				---		14.6	84.7
CB-07-01 (1-7')	1-7				---			
ST-07-02 (2')	2				---		15.9	97.4
ST-07-04 (4')	4				---		21.2	102.9
ST-07-06 (6')	6				---		24.9	97.0

US LAB SUMMARY GEOTECH 2 S2851-05-02 JADA WINDOWS.GPJ US LAB.GDT 11/13/24

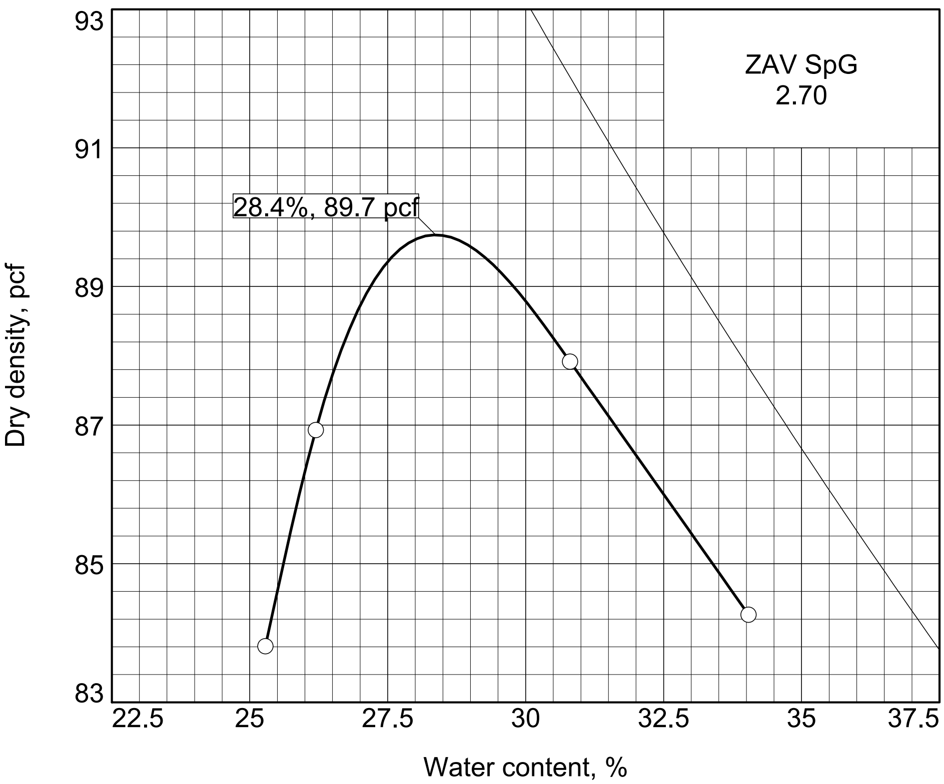
GEOCON

Geocon Inc
3160 Gold Valley Drive, Suite 800
Rancho Cordova, CA 95742
Telephone: 858-558-6900

Summary of Laboratory Results

Project: Jada Windows
Location: Nevada County,
CA Number: S2851-05-2
Figure: E1
Date: 11/24

COMPACTION TEST REPORT



Curve No.
T24-04-CB-04-01

Test Specification:
ASTM 1557 Method A 2024 Mold PM9

Preparation Method

Hammer Wt. 10

Hammer Drop 18

Number of Layers 5

Blows per Layer 25

Mold Size 0.0333 cu. ft.

Test Performed on Material Passing #4 **Sieve**

NM **LL** **PI**

Sp.G. (ASTM D 854) 2.7

%>#4 **%<No.200**

USCS **AASHTO**

Date Sampled

Date Tested 11/11/24

Tested By JF

TESTING DATA

	1	2	3	4	5	6
WM + WS	3779.0	3748.0	3628.0	3699.0		
WM	2042.0	2042.0	2042.0	2042.0		
WW + T #1	1891.0	1947.0	1908.0	2156.0		
WD + T #1	1481.0	1513.0	1574.0	1811.0		
TARE #1	150.0	238.0	253.0	494.0		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	30.8	34.0	25.3	26.2		
DRY DENSITY	87.9	84.3	83.8	86.9		

TEST RESULTS

Maximum dry density = 89.7 pcf

Optimum moisture = 28.4 %

Material Description

Red Elastic SILT

Remarks:

Project No. S2851-05-02 **Client:**

Project: Jada Windows

○ **Depth:** 1-7' **Sample Number:** T24-04-CB-04-01

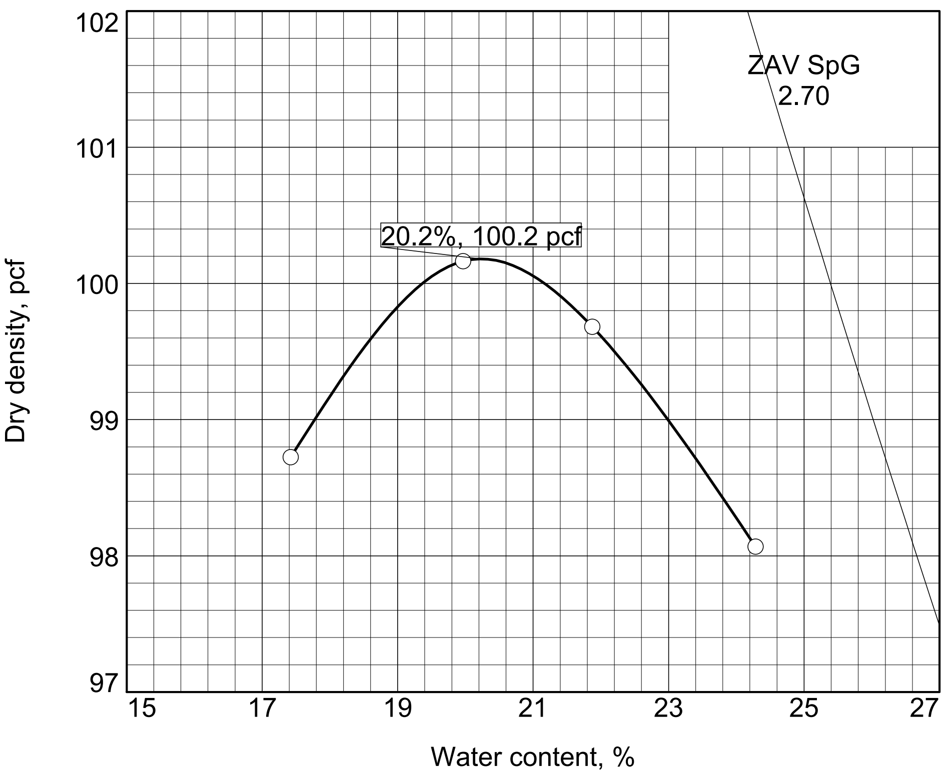
GEOCON CONSULTANTS, INC.

Checked by: MR

Title: LM

Figure E2

COMPACTION TEST REPORT



Curve No.
T24-07-CB-07-01

Test Specification:
ASTM 1557 Method A 2024 Mold PM9

Preparation Method
Hammer Wt. 10
Hammer Drop 18
Number of Layers 5
Blows per Layer 25
Mold Size 0.0333 cu. ft.
Test Performed on Material Passing #4 **Sieve**
NM **LL** **PI**
Sp.G. (ASTM D 854) 2.7
%>#4 **%<No.200**
USCS **AASHTO**
Date Sampled
Date Tested 11/12/24
Tested By JF

TESTING DATA

	1	2	3	4	5	6
WM + WS	3857.0	3877.0	3883.0	3793.0		
WM	2042.0	2042.0	2042.0	2042.0		
WW + T #1	1997.0	2041.0	2050.0	2062.0		
WD + T #1	1693.1	1710.4	1689.5	1802.2		
TARE #1	171.0	199.0	205.0	311.0		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	20.0	21.9	24.3	17.4		
DRY DENSITY	100.2	99.7	98.1	98.7		

TEST RESULTS

Maximum dry density = 100.2 pcf

Optimum moisture = 20.2 %

Material Description

Brown Elastic SILT

Remarks:

Project No. S2851-05-02 **Client:**
Project: Jada Windows

○ **Depth:** 1-7' **Sample Number:** T24-07-CB-07-01

Checked by: AD
Title: LC

GEOCON CONSULTANTS, INC.

Figure E3

APPENDIX F

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

Jada Windows

Latitude, Longitude: 39.2218, -121.0354

Date	11/22/2024, 11:11:41 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	0.56	MCE_R ground motion. (for 0.2 second period)
S_1	0.234	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.715	Site-modified spectral acceleration value
S_{M1}	0.35	Site-modified spectral acceleration value
S_{DS}	0.477	Numeric seismic design value at 0.2 second SA
S_{D1}	0.234	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.276	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.243	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.292	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	0.56	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.608	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.234	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.252	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)

Type	Value	Description
PGA_{UH}	0.243	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.921	Mapped value of the risk coefficient at short periods
C_{R1}	0.928	Mapped value of the risk coefficient at a period of 1 s
C_V	0.973	Vertical coefficient

DISCLAIMER

While the information presented on this website is believed to be correct, SEAOC /OSHPD and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in this web application should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. SEAOC / OSHPD do not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the seismic data provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the search results of this website.