GEOTECHNICAL ENGINEERING REPORT

Proposed PreFab Building
39.3354°, -120.1483°
Glenshire Drive
Truckee, California 96161
PSI Project No. 0587858

PREPARED FOR:

Five Nine Design Group
15925 Hargray Drive
Noblesville, Indiana 46062

May 1, 2024

BY:

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May 1, 2024

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Attn: **Guy Stumm**
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Re: **Geotechnical Engineering Report**
PROPOSED PREFAB BUILDING
Glenshire Drive
GPS Coordinates: 39.3354°, -120.1483°
Truckee, California 96161
PSI Project No. 0587858

Professional Service Industries, Inc. (PSI), an Intertek Company, is pleased to submit this Geotechnical Engineering Report for the referenced project. This report includes the results from PSI's recent field exploration, field testing, and laboratory testing along with recommendations for use in preparation of the appropriate design and construction documents for this project.

PSI appreciates the opportunity to provide this Geotechnical Engineering Report and looks forward to continuing participation during the design and construction phases of this project. PSI also has great interest in providing materials testing and inspection services during the construction of this project and will be glad to meet with you to further discuss how we can be of assistance as the project advances.

If there are questions pertaining to this report, or if PSI may be of further service, please contact us at your convenience.

Respectfully submitted,

Professional Service Industries, Inc.

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1.0 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

Professional Service Industries, Inc. (PSI), an Intertek Company, has completed field exploration, field and laboratory testing, and geotechnical evaluation for the proposed prefabricated building in Truckee, California at the referenced address. This work was performed in general accordance with PSI's proposal 0587-419810, dated March 7, 2024, and signed March 25, 2024.

1.2 SITE LOCATION

Currently the site consists of an undeveloped area south of Glenshire Drive (south of 12041 Highland Avenue). The area consists of vacant land with vegetation, small trees, shrubs, and boulders. A railroad track exists further south of the site.

1.3 PROJECT DESCRIPTION

Based on the Request for Proposal (dated February 15, 2024) and subsequent email discussions, the proposed project will consist of constructing a premanufactured building at the site for fiber optic utilities. The building will be placed on perimeter trench foundations with a raised structural slab. The building is proposed to weigh approximately 198,000 pounds and has a footprint of approximately 36 feet by 23⅓ feet generating a maximum foundation loading of 1700 psf. Minimal parking and drive aisles are also proposed. One soil boring extending to a depth of approximately 20 feet is requested. No grading plans were provided; however, PSI anticipates that ±2 feet of grading may be required. No geotechnical reports for this area were provided for our review.

The geotechnical recommendations presented in this report are based on the provided project information, structure locations, and the subsurface materials encountered during the field investigation. Should any of the above information or design basis made by PSI be inconsistent with the planned construction, it is requested that you contact us immediately to allow us to make any necessary modifications to this report. PSI will not be held responsible for changes to the project if not provided the opportunity to review the information and provide modifications to our recommendations.

2.0 SITE GEOLOGY

2.1 REGIONAL GEOLOGY

The subject site is located within the Sierra Nevada geomorphic province. The Sierra Nevada consists of a tilted fault block nearly 400 miles long. Its east face is a high, rugged multiple scarps, contrasting with the gentle western slope that disappears under sediments of the Great Valley. Deep river canyons are cut into the western slope (CGS, 2002).

2.2 SITE GEOLOGY

Based on geologic mapping of the area by Harwood, Fisher, and Hanson (2014), PSI anticipates the geology to generally consist of Donner Lake glacial till (Qdt) and outwash deposits (Qtio). No previous geotechnical reports were provided for our review nor were any geotechnical reports discovered within the California Geotracker database.

2.3 SOIL SURVEY

Based on a USDA custom soil survey report, the site is generally underlain by the Inville-Riverwash-Aquolls complex (EWB) consisting of cobble coarse sandy loam, very cobbly loam, and very gravelly loam. This soil is reported to have low potential for concrete and steel corrosion.

2.4 REGIONAL SEISMICITY

The project site is located in Northern California, which has undergone a complex multiphase structural history and remains an active tectonic region with documented historic earthquakes. Generally, the seismicity within California can be attributed to faulting due to regional tectonic movement. This includes the San Andreas Fault and other sub-parallel strike-slip faults, as well as normal and thrust faulting within the State. The area of the subject site is considered seismically active. Seismic hazards within the site can be attributed to potential ground shaking resulting from earthquake events along nearby or more distant faulting.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1.1 TOPOGRAPHY

Based on the USGS topographic map for the Truckee Quadrangle (2022), the site appears to have an elevation of around El 5,760 feet above Mean Sea Level.

3.1.2 FAULTING

Based on California Geological Survey mapping, no Alquist-Priolo fault zones exist on site and no known faults cross the site. Faults which may affect the project site include the active Polaris fault located just east of the site, the active Dog Valley fault zone located approximately 3½ miles northwest of the site, and the potentially active Wolf Creek fault zone located approximately 2 miles southwest of the site.

3.1.3 LIQUEFACTION

Soil liquefaction and seismically-induced settlement typically occur in saturated loose to medium dense cohesionless soils; and in clays and silts with low plasticity indexes and with moistures near their liquid limits, due to cyclic softening where the groundwater is relatively shallow (within 50 feet of the ground surface). During an earthquake, ground shaking causes a rapid increase in the porewater pressure within the soil mass under undrained conditions. The generation of excess porewater pressures causes a corresponding decrease in the soil's effective stress, which can result in a sudden loss of soil bearing strength and ground surface settlement within the liquefied (and softened) soil layers. Soil liquefaction potential is generally affected by soil types, mineral contents, ground acceleration, duration of shaking, and frequency content of the earthquake ground motion, among other factors.

Based on the California Geologic Survey website, the site area has not been delineated as having a potential for liquefaction. The Cal OES MyHazards website also does not delineate the site area as having any environmental hazards. Due to the dense to very dense silty sand and cobbles encountered onsite and lack of groundwater, we do not believe that liquefaction is a design concern at this site. Therefore, a liquefaction evaluation has not been included in the scope of work for this report.

3.1.4 LATERAL SPREADING

Due to the anticipated liquefaction potential at this site, the potential for lateral spreading or surficial disturbance (sand boils, grabens, etc.) is also considered negligible.

3.1.5 LURCHING AND SHALLOW GROUND RUPTURE

Evidence of active fault rupture was not observed within the explored areas of the site at the time of our subsurface exploration. PSI did not observe mapped faults crossing the site in readily available resources. As such, the potential for ground rupture from faulting at the site is considered to be negligible.

3.1.6 TSUNAMIS AND SEICHES

Based on the California Tsunami Map, elevation of the site, and distance to the ocean, the site is not located within a tsunami prone area. The site is not located near a large body of water; therefore, we do not consider seiches to be a design concern for the site.

3.1.7 LANDSLIDES AND SLOPE STABILITY

Seismically induced landsliding is not considered a hazard on or adjacent to the project site due to the absence of significant steep slopes in or around the project area.

3.1.1 Seismic Design Parameters

PSI has employed the 2022 California Building Code (CBC), the locally adopted version of the International Building Code, 2021 edition. As part of this code, the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site.

As part of the procedure to evaluate seismic forces, the code requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface. Our boring extended to a maximum depth of approximately 20¾ feet bgs, but to define the Site Class for this project, we have available geologic information and interpreted the results of a soil test boring drilled within the project site and estimated appropriate soil properties below the base of the boring to a depth of approximately 100 feet as permitted by the code. The estimated soil properties were based upon the soils encountered at the site, data available in published geologic reports, and our experience with subsurface conditions in the general site area.

Based upon our evaluation, the subsurface conditions at the site are consistent with the characteristics of a **Site Class "C"** (stiff soil) as defined in Chapter 20.3-1 of the ASCE 7-16. The associated probabilistic ground acceleration values and site coefficients for the general site area were obtained from the Structural Engineers Association web page (https://www.seismicmaps.org/) using ASCE 7-16 and are presented in the table below.

Table 3.1: Ground Motion Values

Period (sec)	Mapped MCE Spectral Response Acceleration (g)		Site Coefficients		Adjusted MCE_R Spectral Response Acceleration (g)		Design Spectral Response Acceleration (g)	
0.2	S_s	1.375	F_a	1.2	S_{Ms}	1.65	S_{Ds}	1.1
1.0	S_1	0.455	F_v	1.5	S_{M1}	0.682	S_{D1}	0.455

2% Probability of Exceedance in 50 years for Latitude, Longitude: 39.3354°, -120.1483°
MCE_R = Maximum Considered Earthquake

In accordance with ASCE 7-16, Chapter 11, for a site with S_1>0.2, the Fa value is governed by Section 11.4.8. Section 11.4.8 requires a ground motion hazard analysis be performed for structures with seismic isolation elements or damping systems on all sites having S_1>0.2. Section 11.4.8 includes an exception where ground motion hazard analyses "is not required for structures other than seismically isolated structures and structures with dampening systems where" the seismic response coefficient of the structure (Chapter 12) meets specific requirements. This report is based on no seismic isolation elements or dampening systems being installed and that exceptions can be applied.

3.2 Field Exploration and Laboratory Testing Program

Prior to initiation of field drilling activities, PSI marked the proposed exploration location with a white stake and paint, and USA North 811 was contacted a minimum of 48 hours prior to beginning work to locate any potential buried utilities. The USA North 811 inquiry identification number for the utility locate request was #2024041501071. A private utility location service was not used for utility searches at this site.

To evaluate soil conditions at the requested location, PSI advanced one (1) geotechnical boring to a depth of 20¾ feet bgs. The boring was drilled using a limited access track rig, using auger drilling methods. The approximate location of the boring is shown on Figure 2.

The exploration design element, approximate depth, and total footage are provided in the following table.

TABLE 3.2: FIELD EXPLORATION SUMMARY

Design Element	Number of Borings	Boring Depth (feet)	Total Footage (feet)
Proposed Pre-Fabricated Building	1	20¾	20¾
TOTAL:	**1**	**---**	**20¾**

During field activities, the encountered subsurface conditions were observed, logged, and initially visually classified in the field (in general accordance with ASTM D2488/D2487). Field notes were maintained to summarize soil types and descriptions, water levels, changes in subsurface conditions, and augering conditions. The terminology used in the soil classifications and other modifiers are depicted in the General Notes and Soil Classification Chart. Information gathered from this boring is described below.

Sampling procedures for the auger-drilled soil boring were performed in general accordance with applicable ASTM methods (ASTM D1586 and ASTM D3550). Samples were identified in the field, placed in sealed containers, and transported to the laboratory for further classification and testing. At the completion of drilling, the soil boring was backfilled with soil cuttings.

The boring location was located in the field by PSI estimating distances from existing site features. However, elevations of the ground surface at the boring location was not provided and should be surveyed by others prior to construction if needed. The references to elevations of various subsurface strata are based on depths below existing grade at the time of drilling. The approximate boring location is depicted on Figure 2.

PSI supplemented the field exploration with a laboratory testing program to determine additional engineering characteristics of the subsurface soils encountered. The laboratory testing program was conducted in general accordance with applicable ASTM Test Methods, and is included in Appendix A. Portions of samples not altered or consumed by laboratory testing will be discarded 30 days from the date shown on this report.

3.3 SUBSURFACE CONDITIONS

The results of the field and laboratory testing have been used to generalize a subsurface profile at the project site. The following subsurface descriptions provide a highlighted generalization of the major subsurface stratification features and material characteristics.

During our fieldwork, we encountered dense to very dense silty sand with gravel and cobbles to boring termination at 20¾ feet bgs. Weathered bedrock appeared to be located below a depth of approximately 7 feet bgs.

The boring log, included in Appendix A, should be reviewed for specific information at the boring location. These records include soil descriptions, stratifications, locations of the samples, and field and laboratory test data. The stratification shown on the boring log represents the conditions only at the actual boring location at the time of our exploration. The stratification that represents the approximate boundary between subsurface materials and the actual transition may be gradual. Variations will occur and should be expected across the site.

3.4 GROUNDWATER INFORMATION

Groundwater was not encountered in our boring which extended to a maximum depth of approximately 20¾ feet bgs. Groundwater level data from several California databases was not readily available in the immediate area.

The groundwater levels presented in this report were measured at the time of PSI's field activities. Based on our findings, groundwater is not expected to impact the proposed construction. It is possible, however, that transient, saturated ground conditions at shallower depths could develop at a later time during periods of heavy precipitation, landscape watering, leaking water lines, or other unforeseen causes. Variations in groundwater levels should be expected seasonally, annually, and from location to location.

4.0 GEOTECHNICAL ENGINEERING ANALYSIS

4.1 EXPANSIVE SOILS

The expansion index (potential) of the near-surface soils is anticipated to be **Very Low** based on their non-plastic, granular characteristics. Following site grading, testing of site soils should be performed by the project geotechnical consultant to confirm the basis of these recommendations if import soils are used. Depending upon the distribution of soil types and the expansion/swell characteristics, differing design recommendations may be developed to better suit the types of conditions present at the site.

4.2 CORROSION POTENTIAL

Both metal and concrete elements in contact with soil are subject to corrosion or degradation due to the chemical activity of the soil. Therefore, buried metal and concrete elements should be designed to resist this chemical activity. Soil samples recovered from the borings were evaluated for the corrosivity characteristics of the subsoils. The composite soil samples were submitted to an analytical lab to determine the Sulfate, Chloride, and pH characteristics. The test results of the laboratory testing are presented in the following table.

TABLE 4.1: SUBSOIL CORROSIVITY CHARACTERISTICS

Sample	Depth (feet)	Water-Soluble Sulfate (SO_4) in Soil ppm (mg/kg) [%]	Water-Soluble Chloride Content in Soil ppm (mg/kg) [%]	pH
B1	0 – 5	3 [0.0003]	<2 [<0.0002]	7.3

The concentration of water-soluble sulfates or dissolved sulfate in water is considered to be a good indicator of the potential for chemical attack on concrete. Based on ACI Manual of Concrete Practice (ACI 201.2R-10) or (ACI 318/318R-33, Table 19.3.1.1), the amount of water-soluble sulfate in soil can be used to evaluate the need for protection of concrete based on the following table.

TABLE 4.2: REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE

Water-soluble sulfate (SO_4) in Soil percent by mass (%)	Water-Soluble Sulfate (SO_4) in Water ppm (mg/kg)	Sulfate Exposure Class
$SO_4^- < 0.10$	$SO_4^- < 150$	S0 or Negligible Exposure
$0.10 \leq SO_4^- < 0.20$	$150 \leq SO_4^- < 1{,}500$	S1 or Moderate Exposure
$0.20 \leq SO_4^- \leq 2.0$	$1{,}500 \leq SO_4^- \leq 10{,}000$	S2 or Severe Exposure
$2.0 < SO_4^-$	$10{,}000 < SO_4^-$	S3 or Very Severe Exposure

Based on the test results of selected soil samples, the sulfate ion concentration and the potential for reactions within concrete exposed to sulfates is **negligible** with a **Class S0** Exposure. Evaluations of soluble sulfate content contained within the selected samples suggest no restriction on the type of concrete. The actual cement type to be used shall be determined by the project Structural Engineer. The soil survey report indicates that the soils have a moderate potential for corrosion to concrete.

The corrosion potential of the soils is also dependent on the chloride content present in the soil. The onset of corrosion of steel in concrete associated with the presence of chloride typically occurs when the values are above 463 mg/kg or ppm (1.5 pounds/yd3). The American Concrete Institute suggests maximum values of chlorides in conventionally reinforced concrete in moist environments be limited to 0.10% by weight of cement.

Soil pH can have a wide range of acidity, between 2.5 and 10. Concrete is alkaline in nature, meaning it has a pH of 7.0 or higher. pH levels below 5 can lead to extreme corrosion rates and premature pitting of metallic objects, a neutral pH of 7 is most desirable to minimize potential for damage. The intrinsic pH level of a soil can also be affected by precipitation. The pH value of the soil sample tested indicates that the soil has a normal pH and a **low** corrosive potential for concrete.

5.0 GEOTECHNICAL DISCUSSION AND RECOMMENDATIONS

5.1 GEOTECHNICAL DISCUSSION

The following geotechnical related recommendations have been developed on the basis of the subsurface conditions encountered and PSI's understanding of the proposed development. Should changes in the project criteria occur, a review must be made by PSI to determine if modifications to our recommendations will be required.

The following geotechnical design recommendations have been developed based on the previously described project characteristics and subsurface conditions encountered. The proposed construction should be performed in accordance with these recommendations and the applicable building code, and local governmental standards which have jurisdiction over this project. If there are changes in the project criteria, PSI should be retained to determine if modifications in the recommendations will be required. The findings of such a review would be presented in a supplemental report. Once final design plans and specifications are available, a general review by PSI is recommended to confirm that the conditions anticipated in preparing this geotechnical report are consistent with the earthwork and recommendations contained within the construction documents.

5.2 SITE PREPARATION

The proposed perimeter trench foundation areas should be stripped and grubbed of vegetation, organic laden materials, undocumented fill, loose soils, boulders, and other structures and utilities in conflict with the proposed construction.

5.3 EARTHWORK

5.3.1 FOUNDATION EXCAVATION

Following site preparation, the newly exposed subgrade soils in site improvement areas must be approved by the Geotechnical Engineering representative prior to construction above. Following approval, subgrade soils should be firm and unyielding. Areas which are not firm and unyielding, and which cannot be densified and moisture conditioned in-place, should be undercut to suitable soils and backfilled, or as recommended by the geotechnical engineering representative.

5.3.2 FILL MATERIALS

Engineered Fill (if needed) may include onsite and import soil and should not contain rock fragments greater than 3 inches in diameter or have greater than 30 percent retained on the ¾-inch sieve, and should not contain more than 3 percent (by weight) of organic matter or other unsuitable material. On-site or imported Engineered Fill soils should have an Expansion Index (EI) that does not exceed 20. Based on our subsurface investigation, existing onsite soils are generally suitable for use as Engineered Fill. The suitability of onsite materials for use as engineered fill should be confirmed by a PSI representative during grading. Fill materials meeting the above requirements should be approved by the Geotechnical Engineer several days prior to use at the site. Soils that are environmentally impacted should not be used as Engineered Fill onsite.

Engineered Fill should be compacted to at least 90 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557). The upper 12 inches of subgrade soils in pavement areas should be compacted to at least 95 percent. The moisture content of Engineered Fill should be maintained at approximately ±2 percent of the material's optimum moisture content for coarse-grained soil. If the Engineered Fill is too dry, water should be uniformly applied across the affected fill area. If the Engineered Fill is too wet, it must be dried. Engineered Fill should be thoroughly mixed by disking or other appropriate methods to obtain relatively uniform moisture content throughout the lift immediately prior to compaction. Due to the granular characteristics of the site materials, vibratory and/or tracked equipment is recommended for compaction.

Engineered Fill should be placed in maximum lifts of 8 inches of loose material. Each lift of Engineered Fill should be tested by a PSI soils technician, working under the direction of our Project Geotechnical Engineer, prior to placement of subsequent lifts. If smaller compaction equipment such as jumping jacks or plate compactors are used thinner lifts will be required to achieve compaction.

Compaction of the Engineered Fill should be verified with a sufficient number of density tests, as determined by the Geotechnical Engineer, to determine if adequate compaction is being achieved by the contractor. The properly compacted Engineered Fill should extend horizontally outward beyond the proposed site improvement area by a distance equal to the height of newly placed fill below.

TABLE 5.1: COMPACTION CRITERIA AND TESTING FREQUENCY

Material Type (Location)	Per Modified Proctor Test (ASTM D1557)			
	Minimum Compaction (%)	Moisture Content Range (ref. to optimum moisture content)		Testing Frequency (min. 3 per lift)
		Minimum	Maximum	
Engineered Fill (General)	90	-2%	+2%	1 per 1,000 sf
Engineered Fill (Top 12-inches of Pavement Subgrade)	95	-2%	+2%	1 per 1,000 sf

5.3.3 EXCAVATIONS

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document was issued to better ensure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavation or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or other parties' compliance with local, state, and federal safety or other regulations.

5.3.4 UTILITIES

Utility trenches may be backfilled with suitable onsite native soils or imported soil above the pipe zone. Trench backfill should be moisture conditioned to within +2 percent above the optimum moisture content (coarse-grained soils), compacted in 6- to 8-inch lifts to a minimum of 90 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557). In pavement areas, the top 12-inches of soil subgrade should reach a minimum of 95 percent. If rocks larger than 3-inches in maximum size are encountered, they should be removed from the backfill material prior to placement in the utility trenches. Pipe zone backfill requirements should be in conformance with the requirements of the local agencies having jurisdiction, but should consist of clean granular sand material having a sand equivalent equal to or above 30. Jetting or flooding of utility backfill is not recommended. If smaller compaction equipment such as jumping jacks or plate compactors are used, thinner lifts will be required to achieve compaction.

5.4 SHALLOW FOUNDATIONS

In our opinion, the structural loads of the proposed development can be supported on conventional spread footing foundations constructed in accordance with the following design criteria following site preparation as described above. Additionally, PSI recommends that foundation type and bearing strata be consistent below each structure. Once ***SITE PREPARATION*** has been performed, the following structural elements may be constructed.

Shallow spread and continuous footings founded at a depth of at least 18 inches below lowest adjacent finished grade can be designed for a maximum net allowable soil bearing pressure of 3,500 pounds per square foot (psf). Minimum widths of 18 inches for column footings and continuous footings should be used in foundation design to reduce the possibility of a local bearing capacity failure. Bearing capacity may be increase by 1/3rd for short-term seismic and wind loadings.

If unsuitable soils or undocumented fill are encountered at footing excavation bottoms, the unsuitable material should be over excavated to suitable subgrade material and replaced with Engineered Fill or granular structural fill. The total width of the over-excavation beneath the design footing elevation should increase by 1 foot (on each side) for each foot of over-excavation. The over-excavated areas should be backfilled with Engineered Fill or clean crushed rock and compacted in accordance with the ***FILL MATERIALS*** section of this report. Alternatively, upon approval by the Geotechnical Engineer, the foundation bearing surface could be lowered to the suitable subgrade material, backfilled with lean concrete, or improved through the use of open graded, clean aggregate if conditions dictate.

Based on the anticipated loads and the recommended site preparation, we estimate that post-construction total static settlement will be less than 1 inch. Differential settlement is estimated to be less than ½ inch over a 40-foot span. Potential seismic dry sand settlements should also be incorporated into structural designs (<½-inch) as discussed previously. These magnitudes of estimated settlements are anticipated to be within tolerable limits but should be confirmed by the project architect and structural engineer.

We recommend the use of a smooth-edged excavator to make the footing excavations. The foundation excavation bottoms should be compacted and observed by a representative of PSI prior to steel or concrete placement to assess that the foundation materials can support the design loads and are consistent with the materials and recommendations discussed in this report.

The base frictional resistance and the passive soil resistance will counteract the horizontal loads on shallow foundations. Footings cast against natural competent soil or compacted soil may be designed using a frictional coefficient between the concrete and soil of 0.4. An ultimate equivalent fluid pressure of 350 pounds per cubic foot (pcf) may be used to compute the ultimate passive resistance. Passive resistance within the upper 1 feet of soil should be neglected if the footings are placed using form boards. If the footings are cast against competent natural soils or properly compacted fill soils and the soils above the footings are paved or consist of concrete floor slabs, the passive resistance within the upper 1 feet can be taken into account. The passive resistance of any un-compacted fill material or loose natural soils should be neglected. It is recommended that the overturning moments on the foundations be resisted by the weight of the foundation system. A minimum factor of safety of 2 should be used for sliding resistance.

Following excavation, footing bottoms should be compacted, observed, and documented by the geotechnical representative, and concrete should be placed as quickly as possible to avoid exposure of the excavations to wetting and drying. Surface run-off water should be drained away from the excavations and not be allowed to pond within 10 feet of the open excavation during or after construction. When possible, the foundation concrete should be placed during the same day the foundation excavation is made. If it is required that footing excavations be left open for more than one day, they should be protected to reduce moisture loss or gain.

PSI should be consulted during the design of the foundations to verify that the appropriate parameters are utilized. PSI should provide periodic observation during construction of the foundations to verify that the design parameters and the soil materials used during construction correspond.

5.5 Pavement Design

In designing the proposed pavement areas, the following conditions were considered:

1. Subgrade support characteristics are typically represented by an R-Value for the design of flexible pavements in this region.
2. Vehicular traffic volumes, in terms of the number and frequency of vehicles and their range of axle loads should be considered.
3. Likely changes in vehicular use over the life of the pavement should also be considered. We have anticipated that the pavement areas will not experience additional traffic.
4. Pavement life cycle was considered to be 20 years using Caltrans and Portland Cement Association Design Methods.

All site preparations and grading should be performed as discussed above.

Since an evaluation of the characteristics of the actual soils present at pavement subgrade can only be provided at the completion of grading, the following pavement sections should be used for planning purposes only. Final pavement designs should be evaluated after R-value tests have been performed on the actual in-place subgrade materials exposed for use during construction.

It should be noted that additional earthwork and/or ground improvement efforts may be required during grading on the actual subgrade material encountered, in order to achieve the aforementioned design parameters and assumptions. These design thicknesses anticipate that a properly prepared subgrade has been achieved.

Based on the results of our field exploration and laboratory testing, we recommend the pavement designs presented in the table below. The pavement design criteria are based on the soil conditions present at the site, an R-value of 40 and the anticipated Traffic Index indicated below based on the estimated traffic for the site. Final pavement designs should be based on R-value testing of the as-graded subgrade and actual traffic volumes.

TABLE 5.2: PAVEMENT DESIGN PARAMETERS

Location	Traffic Index (TI)	Asphalt Thickness (inches)	+	Aggregate Base Thickness (inches)	Portland Cement Concrete Thickness (inches)	+	Aggregate Base Thickness (inches)
Standard Drive Aisles/Parking Spaces	5	3	+	4	5½	+	4
		-	+	12			
Heavy Duty Entrances and Drive Aisles	7	3	+	10	6	+	4
		0	+	17			

We recommend rigid pavements (Portland Cement Concrete, PCC) be constructed at all areas requiring heavy braking and turning such as intersections, entrances, docking bays, trash truck loading areas, etc. PCC pavement sections should incorporate appropriate steel reinforcement and crack control joints if needed and as designed by the project structural engineer. We recommend that sections be as nearly squared as possible and no more than 15 feet on a side. Contraction joints may be constructed by saw cutting to a depth of 30 percent of the slab thickness. Expansion/cold/construction joints may be used in lieu of contraction joints. Such joints should be properly sealed.

The presented estimated pavement sections are based on the field and laboratory test results for the project, local pavement design practice, design assumptions presented herein and previous experience with similar projects. The project Civil Engineer should verify that the Traffic Index and other design values are appropriate for the expected traffic and design life of the project. PSI should be notified in writing if the assumptions or design parameters are incorrect or require modification.

Pavements can be expected to crack due to environmental factors. Periodic maintenance to reduce damage to the pavement structure should be planned throughout the life of the pavement. During the paving life, maintenance to seal surface cracks within concrete or asphalt paving and to reseal joints within concrete pavement should be undertaken to achieve the desired paving life. Perimeter drainage should be controlled to prevent or retard influx of surface water from areas surrounding the paving. Water penetration leads to paving degradation. Water penetration into base or subgrade materials, sometimes due to irrigation or surface water infiltration leads to pre-mature paving degradation. Curbs should be used in conjunction with asphalt paving to reduce potential for infiltration of moisture into the base course. Curbs should extend the full depth of the base course and should extend into the underlying subgrade. The base layer should be tied into the area inlets to drain water that may collect in the base.

Pavement material requirements for the above pavement sections are presented below:

- **Compacted Subgrade:** Pavement subgrade preparation should be performed in accordance with the ***FILL MATERIALS*** section. Subgrade soils should be proof-rolled and scarified to a depth of 12 inches. The upper 12 inches of pavement subgrade should be compacted to 95 percent of the laboratory standard.

- **Aggregate Base Course:** Base materials should consist of Class II Aggregate Base. Aggregate base should be placed in maximum 6-inch compacted lifts. The base materials should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Aggregate base materials should be moisture conditioned to between ±2 percentage points of the optimum moisture content.

- **Asphalt Concrete:** Asphalt mix designs should be provided by others and should include a minimum of 1½ inch surficial wear coarse (fine grained) above a base coarse (coarse-grained) layering.

- **Portland Cement Concrete:** Concrete used for paving should have a minimum compressive strength of 4,000 psi at 28-days. Concrete should have a water/cement ration of 0.5 or lower. The air content at the point of placement should range from 2 to 4 percent. The concrete pavements should be reinforced and jointed per current ACI recommendations.

All materials should conform to and be placed in accordance with the latest revision of the Standard Specifications for Public Works Construction (Greenbook), the American Concrete Institute (ACI), the Portland Cement Association (PCA) and the requirements of the City Public Works.

5.6 DRAINAGE CONSIDERATIONS

Site grading should be carefully planned to promote positive drainage away from structures and to divert surface water away from or into stormwater systems. Water should not be allowed to collect near foundations either during or after construction.

Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend that ground surfaces adjacent to buildings be sloped to facilitate positive drainage away from the buildings.

A positive slope gradient of 5 percent down and away from the building perimeter should be applied to the finished subgrade (inclusive of topsoil). This slope should extend no less than 10 feet away from the outside building perimeter, with drainage swales provided to remove runoff from around the structure. Any utility trench that enters the perimeter of a structure should be excavated with a slight slope down and away from the perimeter of the structure.

The soils encountered at this site maybe sensitive to excessive disturbances caused by construction traffic and changes in moisture content. During wet weather periods, increases in the moisture content of the soil may cause significant reduction in the soil strength and support capabilities. In addition, soils which become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundations construction activities during dry weather.

If grading occurs in a period of increased rainfall, unstable subgrade conditions may be present. These conditions may require stabilizing the subgrade with admixtures, such as cement kiln dust or a coarse aggregate. Isolated areas may be stabilized using a geogrid, such as Tensar BX-1200 or equal, with one foot compacted aggregate base over the geogrid. Additional recommendations can be provided, as required, during construction.

5.7 PLAN REVIEW AND CONSTRUCTION OBSERVATION

After final plans and specifications are complete, PSI should review the design and specifications so that the earthwork recommendations are properly interpreted and implemented. It is considered imperative that the Geotechnical Engineer and/or their representative be present during earthwork operations to observe the field conditions with respect to the design documents and specifications. If PSI is not retained to observe and document geotechnical construction activities, others must assume geotechnical engineer of record (GEOR) and be responsible for such earthwork construction. PSI will not be responsible for changes in the project design or project information it was not provided, or interpretations and field quality control observations made by others. PSI would be pleased to provide these services for this project. Future reviews, responses to reviewers, conference calls, or additional scopes of work beyond those discussed herein will require additional budget.

6.0 GEOTECHNICAL RISK AND REPORT LIMITATIONS

The concept of risk is an important aspect of the geotechnical evaluation. The primary reason for this is that the analytical methods used to develop geotechnical recommendations do not comprise an exact science. The analytical tools which geotechnical engineers use are generally empirical and must be used in conjunction with engineering judgment and experience. Therefore, the solutions and recommendations presented in the geotechnical evaluation should not be considered risk-free and, more importantly, are not a guarantee that the interaction between the soils and the proposed structure will perform as planned. The engineering recommendations presented in the preceding sections constitute PSI's professional estimate of those measures that are necessary for the proposed structure to perform according to the proposed design based on the information generated and referenced during this evaluation, and PSI's experience in working with these conditions.

Services performed by PSI for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in this area. No warranty, expressed or implied, is made.

The recommendations submitted are based on the subsurface information obtained by PSI, and information provided by the client, client's representative and client's design consultants. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation and/or other recommendations are required. If PSI is not retained to perform these functions, PSI cannot be responsible for the impact of those conditions on the performance of the project.

The Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations.

This report has been prepared for the exclusive use of the Client and their design consultants.

7.0 REFERENCES

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2. California Building Code (CBC), 2022.
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4. California Department of Water Resources, SGMA website.
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7. Harwood, D.S., Fisher, G.R., and Hanson, R.E., “Geologic Map of Part of Eastern Placer County, California”, MS-061, 2014.
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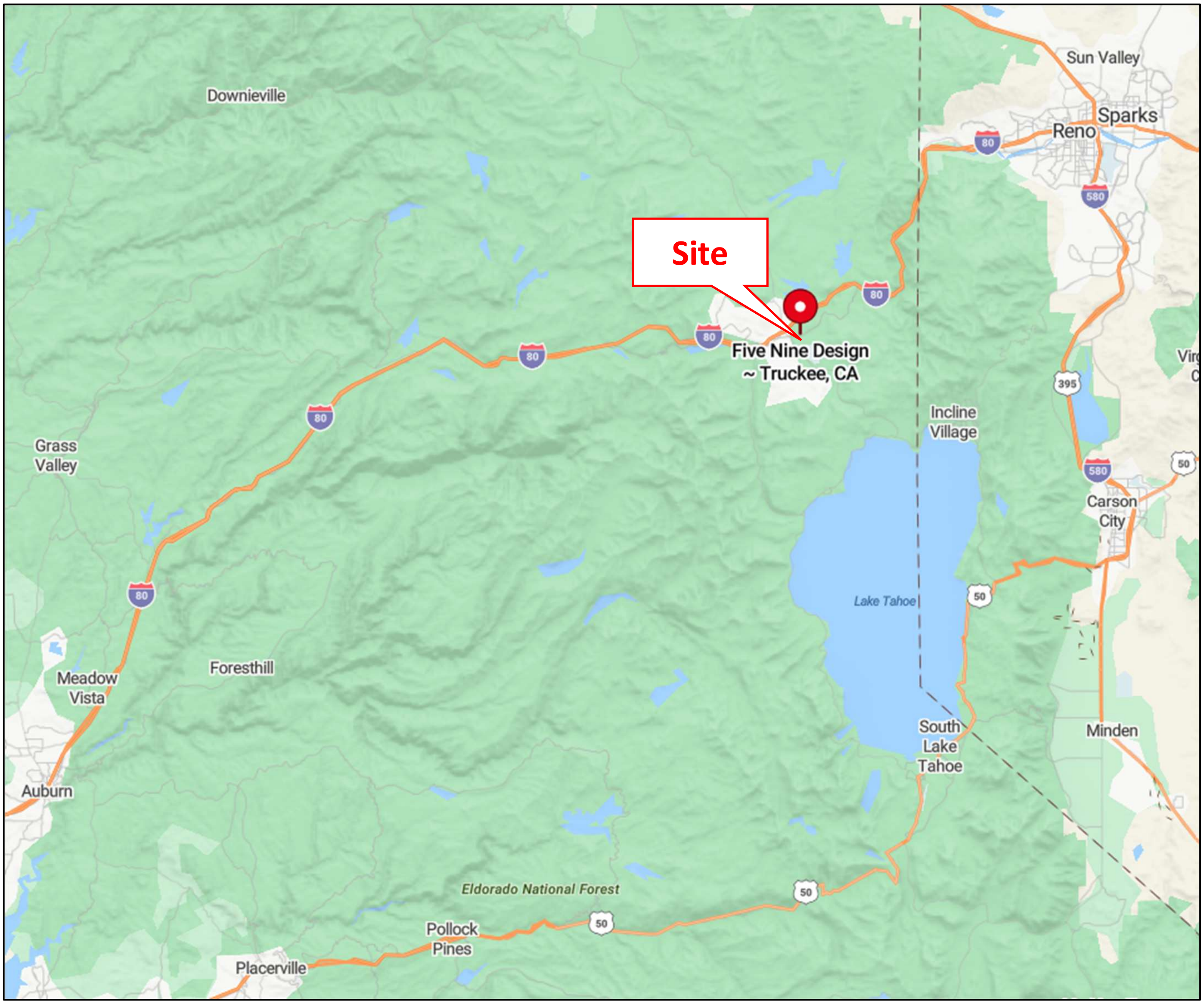
10. Structural Engineers Association, OSHPD Seismic Design Maps, https://www.seismicmaps.org/
11. Terzaghi, K. and Peck, R. B., “Soil Mechanics in Engineering Practice”, 2nd edition.
12. USDA NRCS Soil Survey, “Custom Soil Resource Report for Tahoe National Forest Area, California”, February 27, 2024.
13. USGS, “Truckee Quadrangle, California, 7.5-minute series”, 2022.

FIGURES



Reference: Bing Maps

N

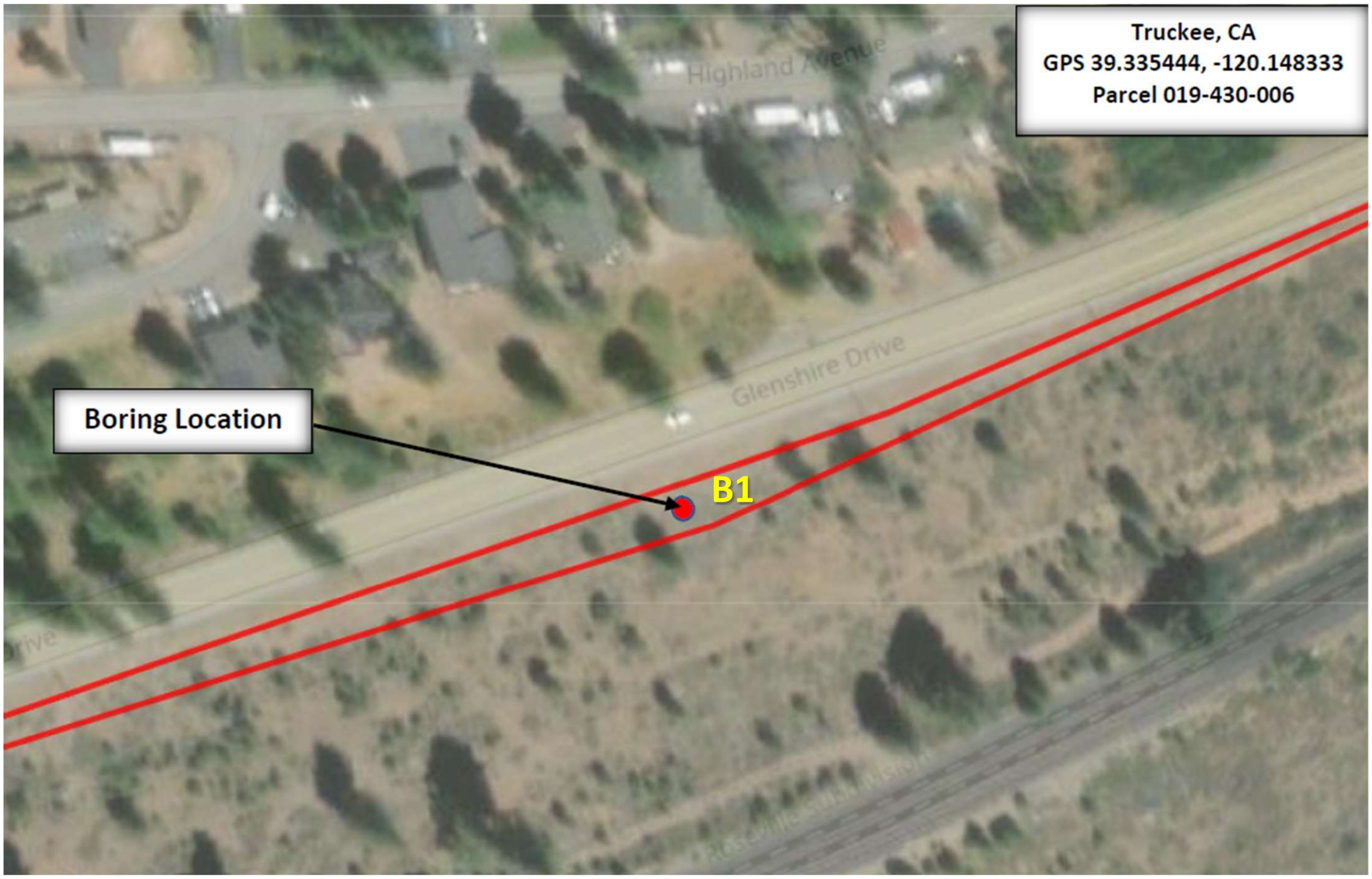
FIGURE 1

Professional Service Industries, Inc.
16170 Vineyard Blvd., Suite 170
Morgan Hill, CA 95037
(408) 669-5500

SITE VICINITY MAP

PSI Project No. 0587858

Five Nine Design Group
GPS 39.3354, -120.1483
Glenshire Drive
Truckee, CA 96161



Reference – Five Nine Design Group RFP, 2/15/24

Legend

B1 **Approximate Boring Location**

N

FIGURE 2

APPENDIX A

Field Exploration & Laboratory Testing Program

FIELD EXPLORATION PROGRAM

On April 18, 2024, the subsurface conditions at the proposed development area were explored by drilling one (1) soil boring to a maximum depth of approximately 20¾ feet below ground surface (bgs). The boring location is shown on Figure 2. Drilling was performed by H1 Drilling Company, LLC of Sacramento, California using a solid flight auger method of drilling with a Limited Access track mounted drilling rig. The soil types encountered at the specific boring locations are presented on the attached Boring Logs below.

During the boring sampling procedure, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 and relatively undisturbed samples were obtained in general accordance with ASTM D3550. The SPT for soil borings is performed by driving a split-spoon sampler, with an outside diameter (O.D.) of 2 inches, into the undisturbed formation located at the bottom of the advanced borehole with repeated blows of a 140-pound hammer falling a vertical distance of 30 inches. The number of blows required to drive the sampler the last 12 inches of an 18-inch penetration depth is a measure of the soil consistency (blow count). For ASTM D3550 (California Modified Sampler) the split barrel sampler possesses a 3-inch O.D. and is driven in the same manner as the SPT. The field blow counts obtained from the California Modified sampler should be adjusted to obtain a rough correlation to SPT blow counts (SPT-N value). SPT blow counts in gravel tend to over-estimate density. Samples were identified in the field, placed in sealed containers, and transported to the laboratory for further classification and testing. At the completion of drilling, the soil boring was backfilled with soil cuttings.

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

SPT: Standard Penetration Test sampler - 1 3/8" I.D., 2" O.D.
CMS: California Modified Sampler - 2 1/2" I.D., 3" O.D.
ST: Shelby Tube - 3" O.D.
RC: Rock Core
BS: Bulk Sample

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼,▽,▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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

intertek psi

BORING B1

DATE STARTED:	4/18/24	**DRILL COMPANY:**	H1 Drilling
DATE COMPLETED:	4/18/24	**DRILLER:**	H1
LOGGED BY:	C. Hoadley	**DRILL RIG:**	Limited Access Track Rig
COMPLETION DEPTH	20.8 ft	**DRILLING METHOD:**	4" Solid Flight Auger
BENCHMARK:	N/A	**SAMPLING METHOD:**	SS: CMS & SPT
ELEVATION:	N/A	**HAMMER TYPE:**	Automatic
LATITUDE:	39.3354°	**EFFICIENCY**	N/A
LONGITUDE:	-120.1483°	**REVIEWED BY:**	D. Abernathy
STATION:	N/A	**OFFSET:**	N/A
REMARKS:			

Water: Not Encountered

BORING LOCATION: See Borng Location Plan

Elevation (feet)	Depth, (feet)	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft	Additional Remarks
	0							Corrosion Series
		7	**Silty SAND**, light brown, dense, with coarse subrounded gravel and cobble, moist.		6-20-11 N=31	7		
	5	8	...becomes very dense.		18-50(4")	9		DD = 87 pcf Sieve Analysis 22.4% < #200
		14	...increase in sand and gravel. [Weathered Bedrock]		17-20-50(6")	2		DD = 129 pcf
	10	6	...becomes brown, trace cobbles.	SM	50(5") N=100	5	>>	
		0	No Recovery, cobbles.		50(0") N=100		>>	
	15	8	...with cobbles.		40-50(2") N=100	7	>>	Sieve Analysis 17.9% < #200
	20	8			17-50(3") N=100	23	>>	

Boring terminated at 20-3/4 feet below ground surface.
Borehole backfilled with soil cuttings.

intertek psi
Professional Service Industries, Inc.
16170 Vineyard Boulevard, Suite 170
Morgan Hill, CA 95037
Telephone: (408) 669-5500

PROJECT NO.: 0587858
PROJECT: Five Nine Design Group
LOCATION: Glenshire Drive
Truckee, CA

The stratification lines represent approximate boundaries. The transition may be gradual.

LABORATORY TESTING PROGRAM AND PROCEDURES

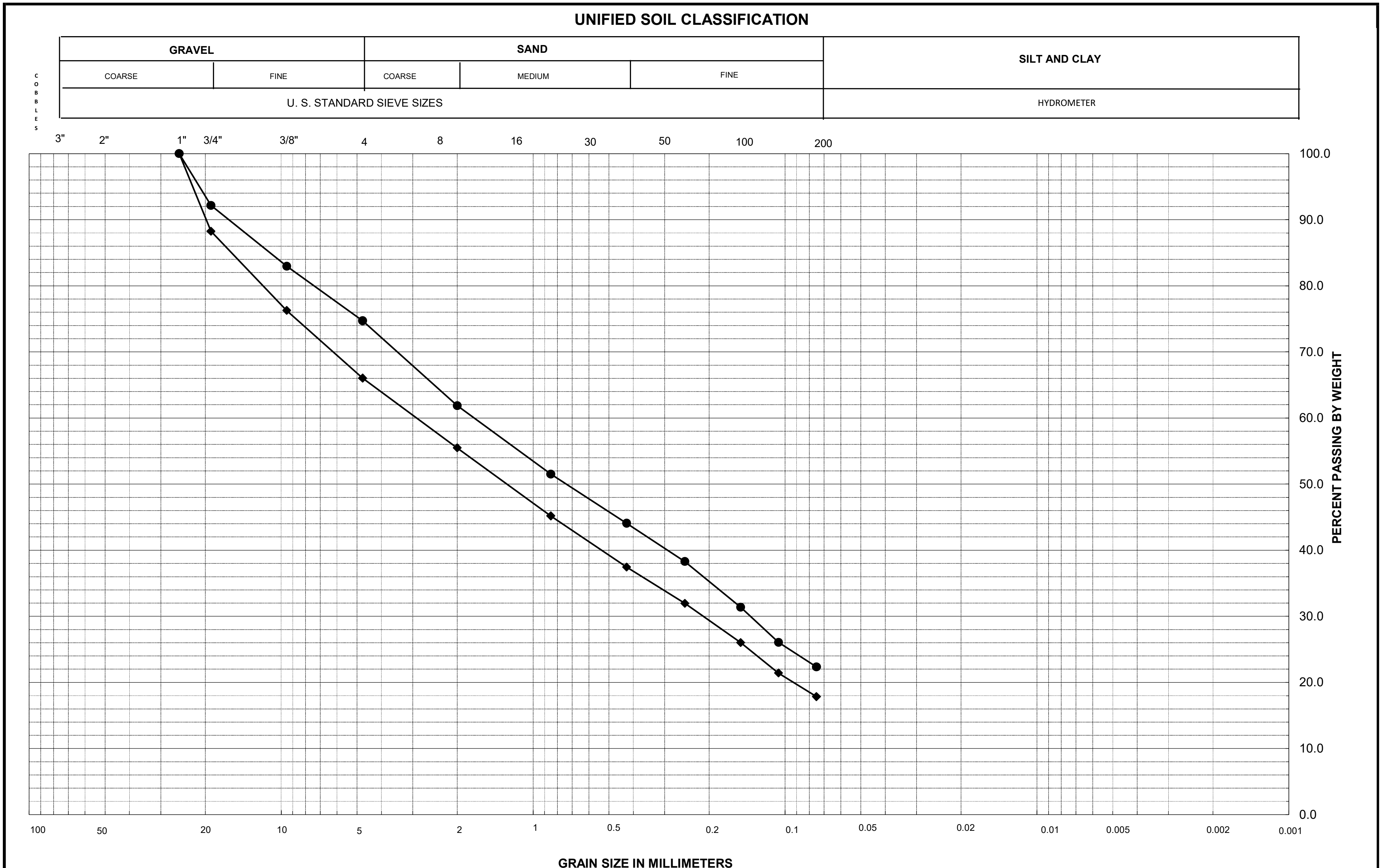
The soil samples obtained during the field exploration were transported to soil laboratories and selected soil samples were tested to determine the material properties for evaluation. Laboratory testing on selected samples included moisture content (ASTM D2216), density (ASTM D2937), and sieve analysis (ASTM D6913). Corrosion series for pH, Sulfate, and Chloride were tested according to ASTM D4327 and G51. Laboratory testing was performed in general accordance with ASTM procedures. Unless otherwise informed, the soil samples will be discarded 30 days from the issuance of this report. Laboratory test results are provided below.

Visual-Manual Classification

The soil samples were classified in general accordance with guidelines presented in ASTM D2487. Certain terminology incorporating current local engineering practice, as provided in the Soil Classification Chart, is included with, or in lieu of, ASTM terminology. The term which best described the major portion of the sample was used in determining the soil type (i.e., gravel, sand, silt or clay).

LABORATORY TEST RESULTS

Boring	Depth (ft.)	USCS Soil Type	Test	Result
B1	0 - 5	Silty SAND w/Gravel	Corrosion Series	See Attached
	6	Silty SAND w/Gravel	Sieve Analysis %<#200	See Plot 22.4
	16	Silty SAND w/Gravel	Sieve Analysis %<#200	See Plot 17.9



Exploration No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% Clay	Description and Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
B1		5	●	8.9				Silty SAND (SM) with Gravel					
B1		15	◆	7.0				Silty SAND (SM) with Gravel					
			■										
			❍										
			□										

PROJECT NAME: Five Nine Design Group ~ Truckee, CA
PROJECT NUMBER: 0587858

 PARTICLE-SIZE DISTRIBUTION CURVES
ASTM D6913

Corrosivity Tests Summary

COOPER TESTING LABORATORY

CTL # 781-062 **Date:** 4/23/2024 **Tested By:** PJ **Checked:** PJ
Client: Intertek-PSI **Project:** Zayo 59 Design Truckee **Proj. No:** 0587858
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
B1	Bulk	0-5	-	-	-	<2	3	0.0003	7.3	-	-	-	14.6	Dark Brown Clayey SAND w/ Gravel & organics