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SECTION H

GEOTECHNICAL REPORT

CAMP SWITZERLAND LIFT STATION PROJECT

FOR

LAKE GREGORY REGIONAL PARK
CRESTLINE, CALIFORNIA

PROJECT NO.: 30.30.0181

Geotechnical Engineering Services

Camp Switzerland Lift Station and Pipes
San Bernardino County, California

for
Kimley-Horn and Associates, Inc.

August 9, 2023

GeoEngineers
Earth Science + Technology

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File No. 26258-001-00

August 9, 2023

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GEOENGINEERS

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1.0 INTRODUCTION AND PROJECT DESCRIPTION

This report presents the results of GeoEngineers, Inc.'s (GeoEngineers) geotechnical engineering services in support of the Camp Switzerland Sewer Lift Station project located in San Bernardino County, California. The project site is shown relative to surrounding physical features on the Vicinity Map (Figure 1) and the Site Plan (Figure 2).

The proposed improvements consist of construction and installation of a sewer lift station servicing new cabin sites, installation of two new High Density Poly Ethelene (HDPE) pipes connecting to an existing polyvinyl chloride (PVC) sewer pipeline, and a Supervisory Control and Data Acquisition (SCADA) communication system with battery backup. Ancillary improvements will include surface structures such as equipment pads. We understand that the maximum depth of excavation associated with the sewer lift station will range between 10 to 15 feet with potential zones of excavation extending to depth of 20 feet below the existing ground surface (bgs).

The purpose of our services is to evaluate subsurface conditions at the site, evaluate potential geologic and seismic hazards that could impact the site and provide geotechnical recommendations in support of the proposed improvements. We understand that the proposed sewer lift station is not critical infrastructure, and therefore, detailed seismic analysis is not required.

GeoEngineers' geotechnical engineering services were completed in general accordance with the Services Authorization Number 195068123 between Kimley-Horn and Associates, Inc. (Kimley-Horn) and GeoEngineers dated November 09, 2022 and as clarified in email communication dated October 31, 2022. We understand that Kimley-Horn is performing this project under contract with The San Bernardino County, Department of Public Works - Special Districts.

2.0 FIELD EXPLORATIONS AND LABRATORY TESTING

GeoEngineers personnel performed a geologic site reconnaissance at the site on May 25, 2023 to identify geological conditions that could cause adverse effects on the lift station and proposed location of pipelines. The reconnaissance consisted of the following:

- Observation of surficial exposed soils within the project area;
- Observation of site topography and surficial slopes within the vicinity of the project
- Evaluation of access roads for drill rig and excavator
- Designating approximate location of test pits and boring.

2.1. Field Explorations

The exploration program for the project consisted of general site reconnaissance, excavation of three exploratory test pits along the proposed alignment of the proposed HDPE connecting pipes and advancement of one exploratory boring in the vicinity of the proposed sewer lift station.

The three exploratory test pits were excavated by GP Excavation on July 10, 2023 along the northern half of proposed sewer pipe alignment. Due to the steep slopes and access limitations, test pits were not excavated along the southern half of the proposed sewer pipe alignment.

The geotechnical boring (B-1) was advanced to a depth of 31.5 feet bgs (10 feet below the anticipated maximum depth of proposed site excavations for the sewer lift station) by Pacific Drilling on July 10, 2023, using a truck mounted rig equipped with hollow stem augers. Due to the presence of groundwater at a depth of 12 feet, a groundwater monitoring well was constructed to measure groundwater at the site. Soil samples were collected by driving Standard Penetration Test (SPT) samplers at 5-foot intervals.

Due to the variably weathered nature of the bedrock materials encountered within B-1, the boring was completed to the full planned depth using hollow stem auger drilling with a collection of drive samples; HQ-wireline rock coring with mud rotary was not required.

2.2. Laboratory Testing

Soil samples were obtained during the explorations and taken to a laboratory for further evaluation. Selected samples were tested for the determination of moisture content and fines content (material passing the U.S. No. 200 sieve). The tests were performed in general accordance with test methods of ASTM International (ASTM). Representative Uniaxial Compressive Strength (UCS) testing could not be conducted on the collected samples due to the high weathering rock. A description of the laboratory testing and the test results are presented in Appendix A.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1. Geology

3.1.1. Regional Geology

The project site is located in the Crestline area within the San Bernadino Mountains of greater San Bernadino County. The San Bernadino Mountains are in turn located within the eastern portion of the Transverse Ranges physiographic province of California. The Transverse Ranges are a long and narrow province characterized by east-west trending, steep side mountain ranges and intervening valleys. This generally east-west trending structure is oblique to the northwest physiographic-structural grain within coastal southern California. The San Garbriel Mountains form the western portion of the province. The San Bernadino Mountain Assemblage is bounded to the south by the San Andreas and Mill Creek Faults and is bounded by the southwestern portion of the Mojave Desert physiographic province to the north. The majority of the bedrock within the San Bernadino Mountains consists primarily of Cretaceous-aged granitic rocks.

3.1.2. Local Geology

Based on the regional geologic map by Morton and Miller (2006) the site is underlain by Mesozoic-aged Mixed Granitic rocks of Silverwood Lake (Geologic Map Unit: Mzsl). Mzsl is a heterogeneous unit that is known to contain a variety of granitic rock types including granodiorite, monzogranite, monzonite, monzodiorite, tonalite and monzonite. Additionally, this collective rock unit is described as being deeply weathered and locally decomposed. Colluvium and alluvium are also present on slopes of canyon walls and in base of alluvial valleys. The location of the site relevant to the local geology is shown on Regional Geologic Map, Figure 3.

3.2. Faulting and Seismicity

3.2.1. Regional Seismicity

The Transverse Ranges have experienced numerous large historical earthquakes, as the area is crisscrossed with faults, many of which are zoned by the Alquist Priolo Earthquake Fault Zoning Act. The eastern extension of the Transverse Ranges, the San Bernadino Mountains, has been historically displaced to the south along the San Andreas Fault. Additionally, intense north-south compression of the Transverse Ranges has resulted in rapid regional tectonic uplift throughout the physiographic province.

3.2.2. Fault Activity

The United States Geological Survey (USGS) have developed a Quaternary Fault and Fold Database of faults and associated folds that are believed to be sources of earthquakes with magnitudes greater than 6.0 that have occurred during the Quaternary (the past 1.6 million years). Class A faults have been categorized based upon the following distinctions:

- Historical faults (activity within last 150 years);
- Latest Quaternary faults (activity within last 15,000 years);
- Late Quaternary (activity within last 130,000 years);
- Middle to late Quaternary (activity within the last 750,00 years); and
- Undifferentiated Quaternary (activity within the last 1.6 million years).

The Class A faults are considered to have the highest potential to generate earthquakes and/or surface rupture, and the earthquakes and the potential for surface rupture increases from oldest to youngest. The evidence for Quaternary deformation and/or tectonic activity progressively decreases for Class B and Class C faults. When a fault is not of tectonic origin, it is considered to be a Class D structure.

The nearest known Historic or Latest Quaternary Class A faults include segments of the San Andreas Fault Zone which are located approximately 5 miles southwest of the project site area. Segments of the San Jacinto Fault Zone are approximately 8 miles southwest of the site.

The nearest known Late Quaternary Class A faults include segments of the Cleghorn Fault Zone, which are located approximately 1.2 miles north of the site, and segments of the Waterman Canyon Fault Zone, which are located approximately 2.1 miles south of the site. Regional faults are presented on Regional Seismicity Map (Figure 4).

The site could be subjected to significant shaking in the event of a major earthquake on any of the faults discussed above or other faults in the southern California area.

3.3. Site Conditions

The project site area is located within the north-south trending Houston Creek canyon, approximately one-quarter mile below the Lake Gregory Dam. The proposed sewer lift station is to be situated in a relatively flat lying area at the base of the canyon, immediately to the west of Houston Creek. The site can be accessed from two different roads—one from the Lake Gregory dam side, which is gated and fully paved until the bottom of the canyon, featuring a steel bridge with a posted 4,000 pounds (lbs) axle weight capacity. Beyond the bridge, the road leading to the lift station is unpaved. The other access road from Houston Drive is also gated and unpaved, with steep slopes leading to the bottom of the canyon where the proposed lift station is to be located.

The proposed alignment for the HDPE sewer pipes runs northeast from the proposed lift station, ascending the slopes, and finally making a left turn of approximately 90 degrees, heading north towards Houston Drive. Half of this path along the proposed HDPE pipe alignment can be accessed using another unpaved road that descends from Houston Drive and covers the upper section of the proposed sewer pipes.

3.4. Subsurface Conditions

Based on our geologic site reconnaissance and recent site explorations, site subsurface conditions are generally consistent with those presented in the regional geologic maps, record drawings and reference documents. The project area is generally underlain by variably weathered granitic rocks. Alluvium soils are present in the generally flat lying area at the base of Houston Creek Canyon in the vicinity of B-1 and the proposed sewer lift station. Colluvial soils are present on canyon slopes and were locally encountered within the exploratory test pits. Both the alluvium and colluvium appear to be derived locally from the underlying weathered igneous rock.

The alluvial soils encountered within B-1 were approximately six to seven feet in thickness and consisted of silty to clayey sands. The underlying granitic bedrock consisted of decomposed (residual soil) to moderately weathered granitic rock, which generally excavated as poorly graded to silty fine to coarse grained sands, particularly within the heavily decomposed materials. As previously discussed, rock coring was not required to reach the maximum depth explored due to the weathered nature of the bedrock. The logs report the disturbed or excavated condition. The in-situ condition is expected to be weathered igneous rock.

The bedrock materials encountered within the exploratory test pits and generally present within the canyon side walls within the project area are also significantly weathered, and readily excavatable using a small to moderate sized excavator. Similar to the material encountered within B-1, the weathered granitic rock encountered within the test pits generally excavated as poorly graded to silty fine to coarse grained sands.

3.5. Groundwater Conditions

Groundwater was first observed within boring B-1 at approximate depth of 12 feet bgs during advancement of geotechnical explorations. A monitoring well was installed, and groundwater was measured at approximate depth of 10.5 feet bgs during pressure transducer sensor installation on July 29, 2023. Groundwater levels likely vary with season and in response to precipitation. The transducer can be retrieved at a later date to provide more detail on seasonal fluctuations.

4.0 GEOLOGIC HAZARDS

Geologic hazards associated with earthquakes include ground rupture, ground shaking, tsunamis, seiches, seismic-induced flooding, liquefaction, seismic-induced ground settlement and seismic-induced slope instability. In addition to geologic hazards associated with earthquakes and faulting, there are other potential geologic hazards that may impact the site. These include landslides, expansive soils, collapsible soils and groundwater. It appears from our research and observations that geologic hazards at the site are limited to those caused by shaking from earthquake-generated ground motions. The following comments are provided with respect to these hazards.

4.1. Surface Fault Rupture

Based on the geologic site reconnaissance and review of referenced literature, the site is not within a State of California-designated Alquist-Priolo Earthquake Fault Zone. In addition, no known active or potentially active fault traces are mapped within the general limits of the recent investigation, nor trend in the direction of the project area. According to the California Geologic Survey (CGS), a fault is considered active if it has offset Holocene sediments less than approximately 11,700 years old. A fault is considered potentially active if fault offsets occurred 11,700 to 2.85 million years ago. As such, the geologic risk associated with ground surface rupture beneath the proposed lift station and surrounding site area is considered to be low.

4.2. Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and sand boils may result from liquefaction. In general, the soil that is susceptible to liquefaction include very loose to medium dense, clean to silty sands and some silts that are below the groundwater level.

Due to the presence of shallow, dense to very dense variably weathered granitic bedrock, within the project site area, the potential for liquefaction, seismic settlement, lateral spreading and related effects is considered to be low.

4.3. Landsliding

According to regional geologic mapping by Morton, D.M. and Miller, F.K., 2006, no landslides are mapped in the site area. In addition, evidence of landslides or landslide potential was not observed during the geologic site reconnaissance. As such, landsliding is not considered to be a significant geologic hazard at the subject site.

4.4. Compressible and Expansive Soils

Based on the geologic site reconnaissance, the alluvial and colluvial soils, and the variably weathered granitic bed rock within the site vicinity do not appear significantly compressible or subject to hydro-collapse based on the anticipated loading. Based upon regional geologic map relationships, it is our opinion that the potential for the existence of expansive soils at the site is low.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1. Construction and Earthwork Recommendations

5.1.1. Construction Excavations

5.1.1.1. General Considerations

We anticipate that construction of the lift station will require excavation and dewatering down to maximum depth of 20 feet over an approximate 45.5-foot by 36.5-foot area that encompasses the footprint of the lift station. Sewer pipelines will extend approximately 550 feet to the north of the lift station.

There are two general methods to accomplish excavation and provide worker protection: (1) passive shield systems, and (2) positive shoring systems. Shields are systems such as trench boxes that are placed in an excavation to protect workers from cave-ins. Positive shoring systems are structures that are designed to provide lateral support to the sides of the excavation and prevent lateral movement or cave-ins.

With shield systems, the excavation is completed before the shield is in place and the shield is removed before the excavation is backfilled. The excavation sides are unsupported and can be prone to sloughing during construction. Even if the sides of the excavation do not slough, the sidewalls may squeeze and move laterally towards the trench. Typically, the potential for movement is limited to areas directly adjacent to the excavation within a distance equal to the depth of the excavation.

Trench boxes or other shield systems will only be suitable where the excavation can safely remain open and unsupported while the trench box is placed. This could limit the use of trench boxes to shallower pipeline installations. Additionally, the surface soils appear to be loose and will be prone to sloughing. Some benching or sloping could be required to advance to the denser layers that can remain vertical while the trench boxes are placed.

Although the decomposed and weathered rock is dense and could likely hold a steep or vertical cut for an extended period of time, the proposed lift station is large and deep and will likely extend below the groundwater level. These factors make open and unsupported temporary cuts a greater risk for sloughing or caving. We recommend that a shoring system be considered. In our opinion, an internally braced soldier pile and lagging temporary shoring system is likely the most practical method. If the contractor proposes a shoring method that requires portions of the excavation to be temporarily unsupported, we recommend that a contingency be carried for additional dewatering and/or staged construction that only allows smaller portions of the excavation to be unsupported at any time.

Shoring and dewatering systems are interdependent, and their design must be coordinated. We anticipate that an external dewatering system, including dewatering wells installed and operating prior to excavation will likely be most practical. If an internal dewatering system is used, one that uses sumps and pumps to clear water that seeps into the excavation, the shoring system will need to consider hydrostatic forces.

It is the responsibility of the contractor to ensure that excavations deeper than 4 feet for all parts of the construction conform to the provisions of Occupational Safety and Health Administration (OSHA), Section 1926.651 “Specific Excavation Requirements.” Shoring, trench boxes or sloped sidewalls will be required for excavation at the site.

The contract documents must specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety and providing shoring, as required, to protect personnel and structures. We recommend that all shoring be designed to accommodate at least 2 feet of overexcavation of the subgrade. We recommend that excavation shoring and dewatering systems be designed by a qualified engineer in general accordance with State of California Department of Transportation (Caltrans) *Trenching and Shoring Manual.*

The Caltrans *Trenching and Shoring Manual* requires a shoring plan to be submitted to the Engineer for review prior to excavation where depth of excavation exceeds 5 feet. The submittal must clearly describe the subsurface soil and groundwater conditions assumed in the design and indicate the location and elevation of any shoring elements and include the extent and limit of any slopes. The submittal must specifically indicate what the maximum design groundwater level is, how the groundwater will be monitored, and what actions will be taken to ensure the safety of the excavation should the groundwater approach or rise above the design groundwater level. We recommend GeoEngineers to be retained to review the proposed shoring and dewatering plan before construction.

5.1.1.2. Construction Shoring

Due to the presence of weathered rock, we anticipate that drilled soldier piles and lagging shoring system will be used for support of excavation. The shoring will need to support both the active soil pressures imposed on the wall by the retained soil and any surrounding construction surcharges. Based on the anticipated depth of the excavation (maximum 20 feet), we expect that a cantilevered system might not be practical or cost effective and that some form of a braced system will be required.

The soil pressures on a shoring wall are dependent on the type of wall, the soil retained, the method of construction, and the extent of dewatering. For preliminary purposes, we suggest that loads against a shoring system be estimated using the soil properties in the following table. These values are based on our explorations at the site.

PRELIMINARY SOIL PARAMETERS

Soil Type	Friction Angle (degrees)	Total Unit Weight (pcf)	Active Equivalent Fluid Weight for Un-Saturated Soil[1] (pcf)	Allowable Passive Equivalent Fluid Weight[2] (pcf)
Fill, Alluvium and Colluvium	30	120	40	115
Decomposed to intensely weathered igneous rock	40	135	29	225

Notes:

[1] Active earth pressures should be used in accordance with pressure distribution presented in Caltrans *Shoring and Trenching Manual* Section 7.1.

[2] Soil pressure based on saturated (buoyant) soil and Coulomb earth pressure theory. These values include a factor of safety of 1.5.

pcf = pounds per cubic foot

Our suggested values are for preliminary planning purposes and assume that a yielding shoring system is used. Soil and water pressures used in final design must be appropriate for the specific shoring system that will be constructed and should be determined by the shoring design engineer.

5.1.1.3. Dewatering

We anticipate constructing the below-ground structure will require temporary lowering of the groundwater table by approximately 10 feet within the excavation if the excavation occurs during the summer months. Temporary dewatering may be accomplished using a variety of means. We anticipate a soldier pile and lagging system with dewatering wells will be used at this site. Wells are anticipated to be installed outside the temporary shoring system. The type of temporary dewatering system will depend on the depth of excavation, type of temporary shoring system, constructability considerations, and other factors. It is our understanding that the trenching for HDPE pipes will not extend below groundwater table and do not require dewatering.

The dewatering system should be design by a Professional Engineer experienced in design of such team and constructed by an experience dewatering contractor who is licensed in State of California. The temporary dewatering system should be designed to maintain the groundwater level at least 1 foot below the bottom of excavation or as required to implement the contractors proposed shoring system.

Surface water from rainfall can contribute significantly to the volume of water that needs to be removed from the excavation during construction and will vary as a function of season and precipitation.

5.1.1.4. Open Trench Excavations

The proposed HDPE pipes and electrical line will be installed within a trench. We understand the proposed lines will be installed in a general northeast-southwest alignment and will be connected to the existing manhole and electrical vault in the vicinity of Houston Drive. Temporary trench excavations should be stabilized by sloping back the sides of excavation or using a temporary shoring or shield system. OSHA guidelines allow temporary slopes for excavations less than 20 feet deep, from 0.75H:1V (Horizontal to Vertical) to 1.5H:1V depending upon soil type. The guidelines assume that surface loads such as construction equipment and storage loads will be kept a sufficient distance away from the top of the cut so that the stability of the excavation is not affected. The guidelines also assume that no groundwater is present. Based on our explorations the near surface soils are anticipated to fall under OSHA "Type C" classification and should have a maximum a temporary maximum slope angle of 1.5H:1V. The contractor will be responsible to determine and confirm the actual OSHA soil type during the construction process. It is important to understand that the trench extends approximately 550 feet in length, and the soil conditions could vary along the trench due to the inherent spatial variability of the soil.

The risk of sloughing will increase the longer the trench remains open. Stockpiles of soil or heavy equipment adjacent to the excavation will also increase the risk of sloughs. The potential for sidewall sloughs must be considered in the work plan. The contractor's work plan should include methods for removing sloughed soil from the trench and methods for stabilizing localized unstable areas. In our opinion, methods for stabilizing localized unstable areas could include: dewatering localized perched groundwater, shoring, laying back unstable slopes, or a combination of these or other methods. The surface of the trenched area shall be restored to generally match existing conditions at the other locations.

5.1.1.5. Excavation Methods

We anticipate that the majority of the excavations can be achieved with conventional earth moving equipment such as large excavators. The contractor should be prepared for less weathered areas where excavation will be significantly slower. The contractor should expect to use excavator buckets with reinforced teeth and ripper claws. Excavators with mounted hydraulic breaker attachments could be required in some areas for efficient excavation.

The contractor should also be prepared to remove and/or break apart isolated boulders of relatively unweathered rock, which could be present. This could require the use of mechanical means (i.e., hydraulic breakers) or chemical agents (i.e., drilling and placing expansive grout). Use of explosives will not be permitted without the express consent and approval of the engineer and contracting agency and should not be assumed as an available method for excavation.

5.1.2. Earthwork for Structures and Backfill

5.1.2.1. Footing Subgrade Preparation

The area to be developed with structures should be stripped of all debris, topsoil, sod, vegetation, existing uncontrolled fill and otherwise unsuitable material. Roots larger than about ½ inch in diameter should be grubbed out. The subgrade must be in a firm and unyielding condition prior to the construction of footings. In areas where the subgrade is soft or yielding, overexcavation and replacement with structural fill will be required.

5.1.2.2. Structure Backfill Compaction

Fill and backfill placed around the subsurface structures must consist of structural fill compacted to at least 95 percent of the maximum dry density (MDD) as determined by ASTM Test Method D 1557 (Modified Proctor) with the exception that backfill placed within 2 feet of the wall is compacted by hand-operated equipment to a density of 90 percent of the MDD. In general, structural fill should be placed in loose lifts not exceeding 12 inches in thickness for import material and lifts not exceeding 8 inches in thickness for on-site material. Each lift should be conditioned to near the optimum moisture content and compacted to the specified density before placing subsequent lifts.

5.1.2.3. Structural Fill Materials

Fill and backfill used to support structural elements must consist of a well-graded sand or gravel compacted to a dense condition. Fill should be free of debris and organic soils with fines contents limited to no more than 20 percent. Fill material used should have very low to low expansion index (EI = 20 or less) as defined by ASTM D4829. Other materials or gradations can be considered on a case-by-case basis.

5.1.2.4. On-site Soils

The on-site soil may be considered for use as fill, provided that it can be properly moisture-conditioned and compacted to a dense non-yielding condition. On-site soils that will be used in structural areas must meet the above criteria for structural fill materials. This should be evaluated on-site as the excavation occurs. The on-site soil does contain a significant amount of fines and will be moisture sensitive. Material that is excavated from below the groundwater table could require drying before it can be compacted as structural fill.

5.1.2.5. Select Granular Fill

If imported fill is needed during wet weather conditions or to backfill within wet excavations, we recommend using fill consisting of well-graded sand and gravel or crushed rock with a maximum particle size of 6 inches and less than 5 percent fines by weight based on the minus ¾ -inch fraction. This material will be less moisture sensitive but still must be compacted at or near an optimum moisture content. It will not be suitable for placement underwater or when saturated.

5.1.2.6. Pipe Subgrade, Bedding, and Backfill

Pipe subgrades must be firm and unyielding. If subgrades are soft and cannot be compacted in place, overexcavation and replacement as described for footing subgrade preparation should be used.

Pipe bedding must conform to the pipe manufacturer's recommendations for pipe bedding and support. In the absence of specific recommendations from the pipe manufacturer, we recommend pipe bedding consist of Caltrans Class 2 Permeable material. Pipe bedding material should be compacted to 90 percent of the MDD according to ASTM D1557.

Where the pipe alignment is located in areas that are not sensitive to settlement at the ground surface, for example, in the open natural areas, the trench can be backfilled above the bedding with native soil from the trench spoils. The native soil should be free of debris or large organic material such as tree stumps. The backfill should be placed in lifts not thicker than 12 inches and at a moisture content similar to the in-place moisture content. Each lift of the native backfill should be uniformly compacted. We recommend that the initial lift of backfill over the pipe be thick enough to reduce the potential for damage during compaction. Backfill material should be compacted to about 90 percent of the MDD per to ASTM D1557. Some settlement of the trench backfill may occur and we recommend that the surface be crowned 6 to 12 inches over the trench to account for this settlement.

Where the pipe alignment is located in structural areas or areas that are settlement sensitive, for example adjacent to or under paved roads, the trench should be backfilled with structural fill. All backfill in structural or settlement sensitive areas should be compacted to at least 90 percent of MDD 3 feet and below finish grade and to at least 95 percent of MDD in the upper 3 feet.

Where the pipe alignment is located in areas that are not settlement sensitive, but do require a firm working surface, for example under gravel roads or in maintenance areas, the trench can be backfilled with on-site soil 3 feet and below finish grade and with structural fill compacted to at least 90 percent of MDD in the upper 3 feet. Some settlement of the ground surface should be expected and could occur after construction.

5.2. Structure Design Recommendations and Analysis

5.2.1. Earthquake Engineering

2018 IBC/ASCE 7-16 MAPPED SEISMIC DESIGN PARAMETERS

ASCE 7-16 Parameter	Recommended Value
Site Class	C
Short-period mapped MCE_R spectral response acceleration, S_S (g)	2.242
Long-period mapped MCE_R spectral response acceleration, S_1 (g)	0.762
Short-period site coefficient, F_A	1.2
Long-period site coefficient, F_V	1.4
Design Spectral Acceleration at 0.2 second period (S_{DS})	1.794
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.712
$T_s = S_{D1}/S_{DS}$	0.397

Notes:

Parameters developed based on latitude 34.246686 and longitude -117.268408 using the Applied Technology Council (ATC) Hazards online tool.

5.2.2. Below Ground Structure Buoyancy

Below ground structures should be evaluated and designed to prevent floatation or uplift, which can be caused when the lift station is empty, and the outside groundwater is present. Resistance to uplift can be developed by the dead weight of the structure, friction along the sides of the structure, and the weight of any soil that is located above a floor slab that protrudes beyond the permanent exterior walls. For design purposes, we recommend that hydrostatic uplift pressures be considered for groundwater up to the ground surface which could occur during flood conditions. When calculating the weights available to resist floatation, the submerged unit weight of 57 pcf can be used for fill, backfill, alluvium, and colluvium. Additionally, a frictional resistance can be computed using a friction 0.36 applied to the lateral soil pressure.

Several procedures are available to provide additional uplift resistance, such as tie-downs, adding concrete weight to the structure and/or extending the bottom slab beyond the structure walls to take advantage of the weight of soil above the slab. If the bottom of the slab is extended, the soil directly above the extended slab can be used with the buoyant soil weight given above.

The floor slab of the well must also be designed to resist this uplift force in flexure.

5.2.3. Structure Foundations

Footings for the lift station can be designed using an allowable soil bearing pressure of 3,000 pounds per square foot (psf) provided the bearing surfaces are prepared as recommended. If smaller ancillary structures founded near the ground surface are required, these structures can be supported on spread footings designed with an allowable soil bearing pressure of 2,000 psf.

These bearing pressures should be applied to the total of dead and long-term live loads and may be increased by one-third when considering total loads, including earthquake or wind loads. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes.

We estimate post-construction footing settlement of 1 inch or less under design load. Differential settlement between comparably loaded isolated column footings or along 50 feet of continuous footing is expected be less than ½ inch. Settlement is expected to occur rapidly as loads are applied. Increased settlement should be expected if subgrades are disturbed.

5.2.4. Lateral Earth Pressures for Subsurface Walls

We recommend that permanent below-grade structures be designed for exterior groundwater level at 8 feet below ground surface or higher for normal operating conditions. We also recommend that full hydrostatic pressure (up to the ground surface) be considered as an extreme case associated with flooding. We recommend that abutment walls backfilled with structural fill placed and compacted as previously recommended be designed using the soil parameters provided in the table below.

LATERAL SOIL PRESSURE PARAMETERS FOR SUBSURFACE WALLS

Soil Parameter	Structural Fill	Submerged Structural Fill	Submerged Highly Weathered Rock[1]
Soil Unit Weight	Total Weight = 130 pcf	Total Weight = 135 pcf Buoyant Weight = 73 pcf	Total Weight = 135 pcf Buoyant Weight = 73 pcf
Friction Angle	34 degrees	34 degrees	40 degrees
Cohesion	0 psf	0 psf	0 psf
Active Earth Pressure[2]	K_a = 0.28 Equivalent Fluid Pressure: K_a*Unit Weight = 36.8 pcf	K_a = 0.28 Total Equivalent Fluid Pressure: (K_a*Buoyant Unit Weight)+hydrostatic = 82.9 pcf	K_a = 0.22 Total Equivalent Fluid Pressure: (K_a*Buoyant Unit Weight)+hydrostatic = 78.2 pcf
At-rest Earth Pressure	K_o = 0.44 Equivalent Fluid Pressure: K_a*Unit Weight = 57.3 pcf	K_o = 0.44 Total Equivalent Fluid Pressure: (K_a*Buoyant Unit Weight)+hydrostatic = 94.4 pcf	K_o = 0.36 Total Equivalent Fluid Pressure: (K_a*Buoyant Unit Weight)+hydrostatic = 88.3 pcf

Notes:

[1] The values for Submerged Highly Weathered Rock are only appropriate where the native weathered rock is located within 3 feet of the exterior face of the wall, otherwise parameters for structural fill should be used.

[2] Active Earth Pressures should only be used where the wall is free to move up to 0.001 times the height of the wall.

If a seismic loading will be considered, we recommend a seismic loading be approximated using a uniform lateral pressure equal to 24*H psf, where H is the depth in feet below grade of the structure. If vehicles will be operated within one-half the height of the wall, a traffic surcharge should be added to the wall pressure. The traffic surcharge can be approximated by a uniform 70 psf horizontal pressure on the wall.

6.0 LIMITATIONS

We have prepared this report for the exclusive use of The San Bernardino County, Department of Public Works - Special Districts, Kimley-Horn, and their authorized agents for the Camp Switzerland Lift Station and Pipes project located in San Bernardino County, California. The data and geotechnical report should be provided to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

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

Any electronic form, facsimile, or hard copy of the original document (email, text, table and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix B, Report Limitations and Guidelines for Use, for additional information pertaining to use of this report.

7.0 REFERENCES

American Society of Civil Engineers, “ASCE 7-16, Minimum Design Loads for Buildings and Other Structures,” 2016.

California Geologic Survey, 2002, “California Geomorphic Provinces,” California Division of Mines and Geology, Note 36.

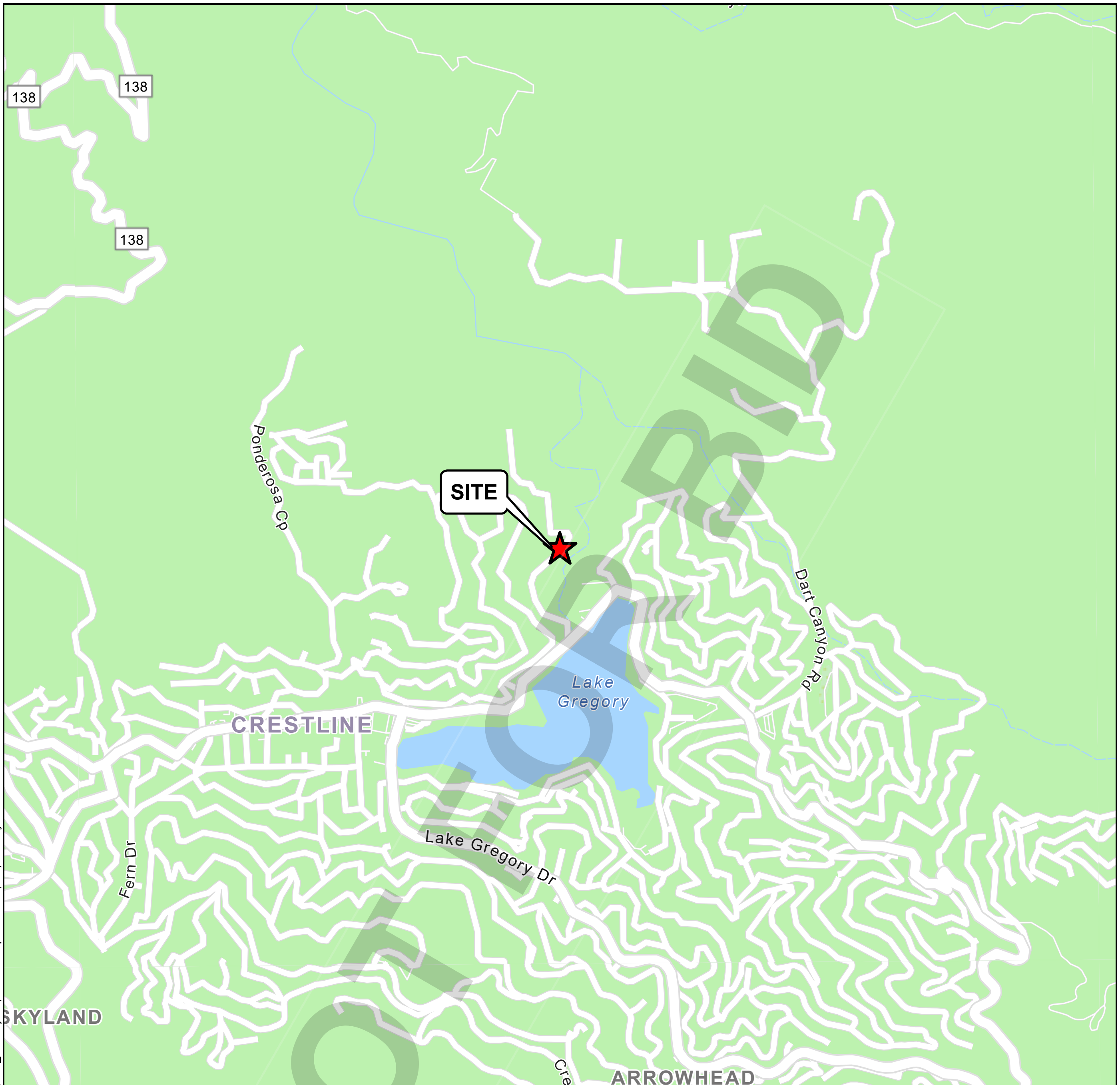
Hart, Earl W., and Bryant, William A., Revised 2007, “Fault-Rupture Hazard Zones in California, Alquist Priolo, Special Studies Zones Act of 1972,” California Division of Mines and Geology, Special Publication 42.

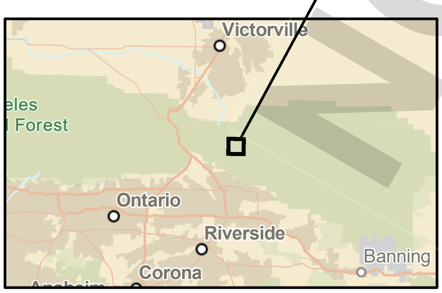
Morton, D.M. and Miller, F.K., 2006, “Geologic Map of the San Bernardino and Santa Ana 30’ x 60’ Quadrangle, California,” US Geological Survey.

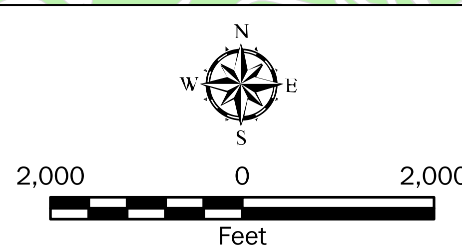
U.S. Geological Survey (California Geologic Survey), 2006, Quaternary fault and fold database for the United States, accessed July 2022, from USGS web site: http//earthquake.usgs.gov/hazards/qfaults/.

FIGURES

GeoEngineers







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

Data Source: ESRI

Projection: NAD 1983 StatePlane California V FIPS 0405 Feet

Vicinity Map

Camp Switzerland Lift Station and Pipes
San Bernardino County, California

GEOENGINEERS

Figure 1

P:\26\26258001\GIS\26258001_Project\26258001_Project.aprx\2625800100_F01_VicinityMap Date Exported: 07/20/23 by ccabrera

Legend

- Approximate Boring Number and Approximate Location
- Test Pit Number and Approximate Location

200 0 200
Feet

N
W E
S

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

Data Source: San Diego County GIS.

Projection: NAD 1983 StatePlane California V FIPS 0405 Feet

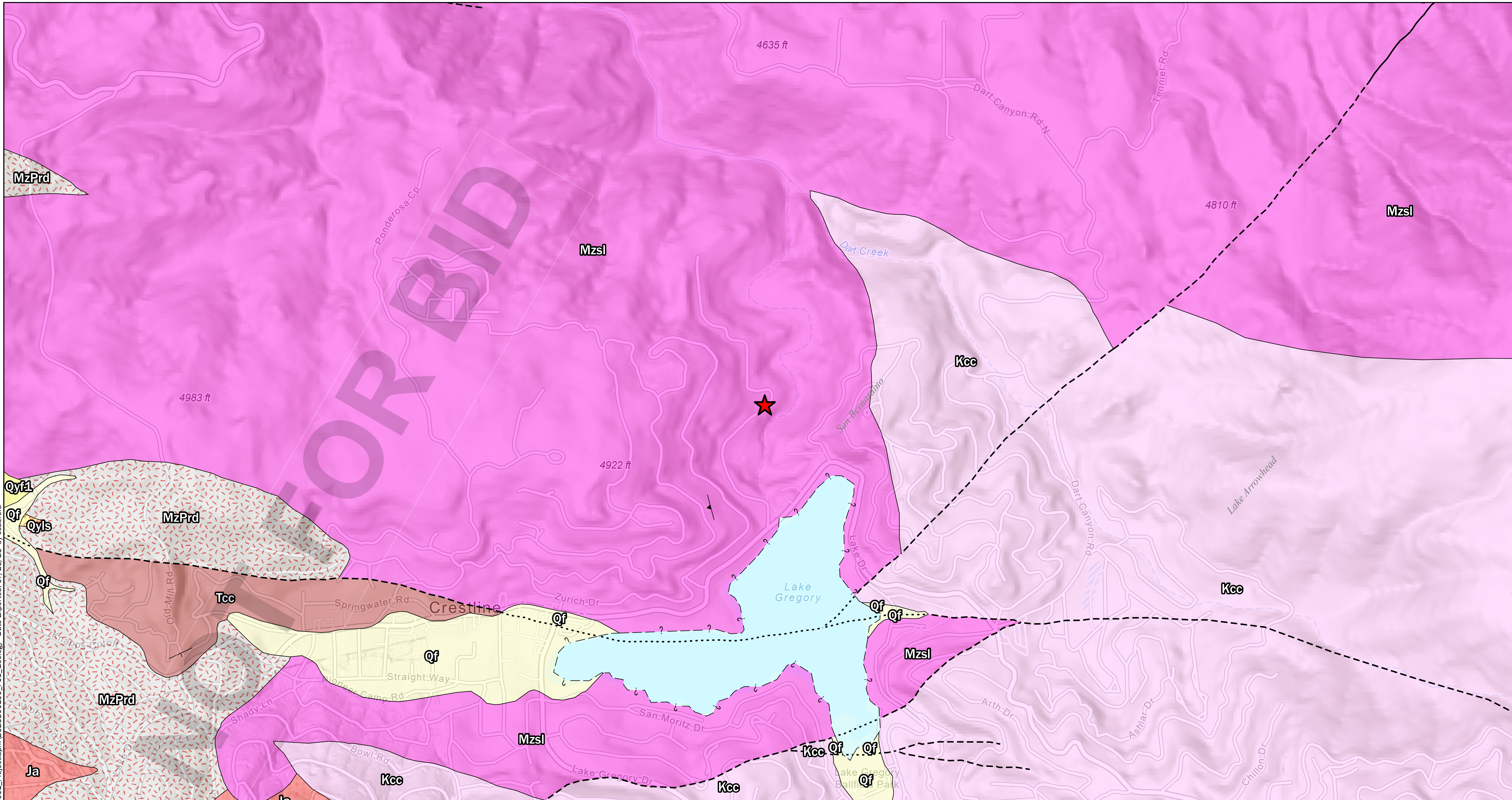
Site Plan

Camp Switzerland Lift Station and Pipes
San Bernardino County, California

GeoEngineers

Figure 2

P:\26\26258001\GIS\26258001_Project\26258001_Project.aprx\2625800100_F02_SitePlan Date Exported: 08/03/23 by ccabrera



NOT FOR BID

Camp Switzerland Lift Station

Surficial Geologic Unit

- Qf: Very young alluvial-fan deposits
- Qyf1: Young alluvial-fan deposits, Unit 1
- Qyls: Young landslide deposits
- Qvof: Very old alluvial-fan deposits
- Tcc: Conglomerate of Crestline
- Mzsl: Mixed granitic rocks of Silverwood Lake
- MzPrd: Gneiss of Devil Canyon
- Kcc: Monzogranite of City Creek
- Ja: Granodiorite of Arrowhead Peak
- water body

N
W E
S

1,000 0 1,000

Feet

Data Source: Morton, D.M. and Miller, F.K., 2006,
"San Bernardino and Santa Ana 30' x 60' quadrangles, California".
USGS.

Regional Geologic Map

Camp Switzerland Lift Station and Pipes
San Bernardino County, California

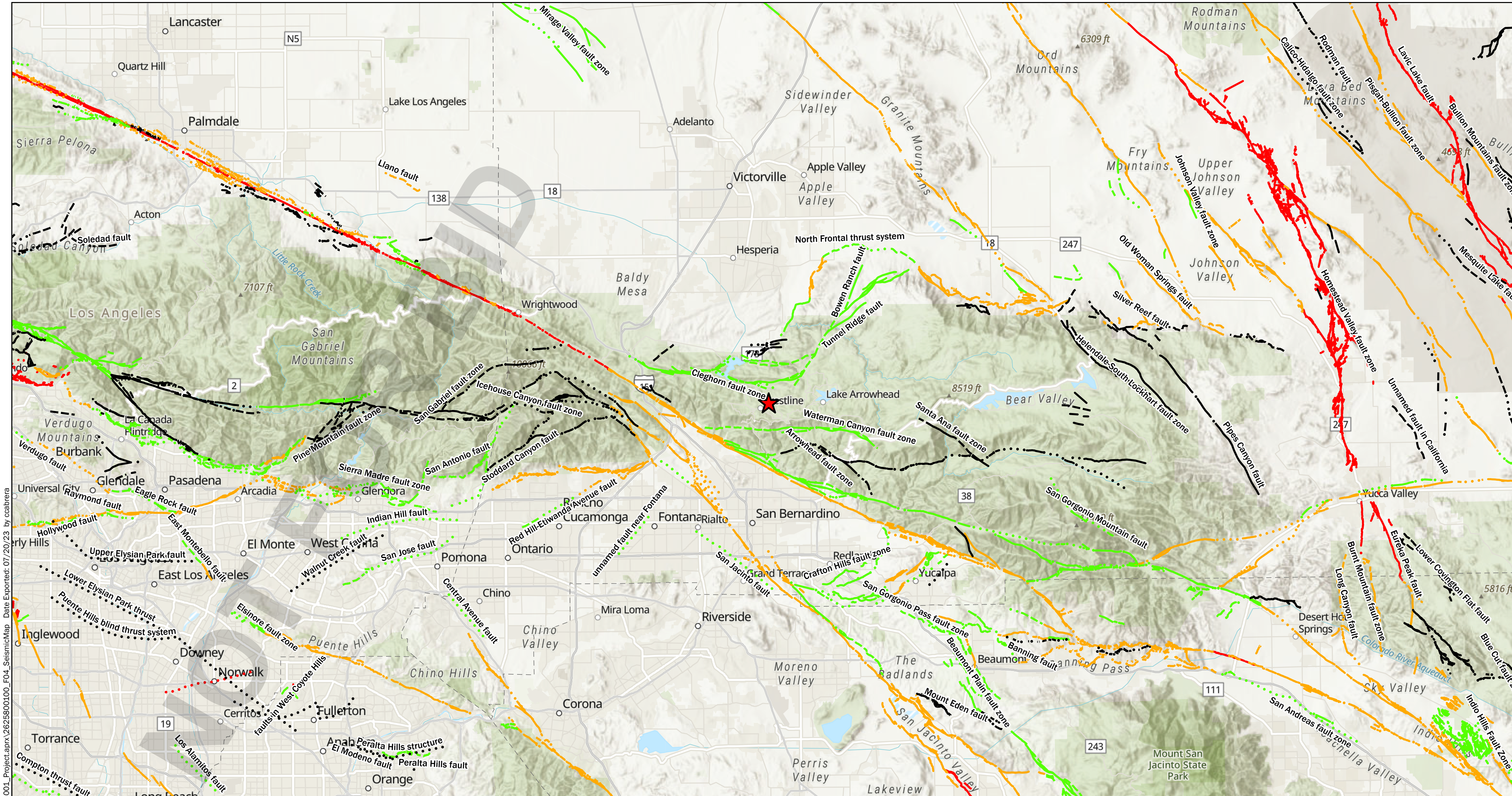
GEOENGINEERS

Figure 3

P:\26\26258001\GIS\26258001_Project\26258001_Project.aprx\2625800100_F03_Geology Date Exported: 07/25/23 by ccabrera

Notes:

Legend



★ Camp Switzerland Lift Station

Quaternary Faults

Based on time of most recent surface deformation

- historical (<150 years), well constrained location
- historical (<150 years), moderately constrained location
- historical (<150 years), inferred location
- latest Quaternary (<15,000 years), well constrained location
- latest Quaternary (<15,000 years), moderately constrained location
- latest Quaternary (<15,000 years), inferred location
- late Quaternary (<130,000 years), well constrained location
- late Quaternary (<130,000 years), moderately constrained location
- late Quaternary (<130,000 years), inferred location
- undifferentiated Quaternary(<1.6 million years), well constrained location
- undifferentiated Quaternary(<1.6 million years), moderately constrained location
- undifferentiated Quaternary(<1.6 million years), inferred location

Data Source: USGS. ESRI.

N
W E
S

8 0 8

Miles

Regional Seismicity Map

Camp Switzerland Lift Station and Pipes
San Bernardino County, California

GEOENGINEERS

Figure 4

P:\26\26258001\GIS\26258001_Project\26258001_Project.aprx\2625800100_F04_SeismicMap Date Exported: 07/20/23 by ccabrera

Notes:

Legend

APPENDICES

GeoEngineers

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface conditions were explored at the site by advancing one geotechnical boring (B-1) and three exploratory test pits (TP-1 through TP-3). The boring was completed to depth of 30.5 feet bgs and the test pits extended to 5 to 12 feet bgs. The explorations were completed on July 10, 2023.

The locations of the explorations were estimated by taping/pacing from existing site features. The approximate exploration locations are shown in the Site Plan, Figure 2.

Borings

Boring B-1 was completed using a truck-mounted, continuous-flight, 8-inch outside-diameter hollow-stem auger drilling equipment. The boring was continuously monitored by an Engineer from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions and prepared a detailed log of each exploration.

The soils encountered in the borings were continuously sampled in the top 10 feet and generally sampled at 5-foot vertical intervals below that. Samples were obtained using both a 2-inch outside diameter split-barrel standard penetration test (SPT) sampler, and a 3 inch outside diameter Modified California Sampler (ASTM D-3550) lined with twelve 2.5 inch diameter, one inch tall stainless steel rings, and one 2.5-inch-diameter 6-inch-tall waste sleeve in the upper portion of the sampler. The relatively undisturbed samples were obtained by driving the sampler 18 inches into the soil with a 140-pound automatic hammer free-falling 30 inches. The number of blows required for each 6 inches of penetration was recorded. The blow count ("N-value") of the soil was calculated as the number of blows required for the final 12 inches of penetration. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. If very dense soil conditions precluded driving the full 18 inches, the penetration resistance for the partial penetration was entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Soils encountered in the borings were visually classified in general accordance with the classification system described in Figure A-1. A key to the boring log symbols is also presented in Figure A-1. The logs of the borings/monitoring wells are presented in Figure A-2. The boring logs are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change may actually be gradual. If the change occurred between samples, it was interpreted. The densities noted in the boring logs are based on the blow count data obtained in the borings and judgment based on the conditions encountered.

Observations of groundwater conditions were made during drilling. The groundwater conditions encountered during drilling are presented in the boring logs. Groundwater conditions observed during drilling represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate.

Laboratory Testing

Soil samples obtained from the explorations were transported to Allied Geotechnical Engineers, Inc. (AGE) laboratory. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, fines content and sieve analyses. The tests were performed in general accordance with test methods of the ASTM International (ASTM) or other applicable procedures and are included within this appendix.

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel / Dames & Moore (D&M)

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GeoEngineers

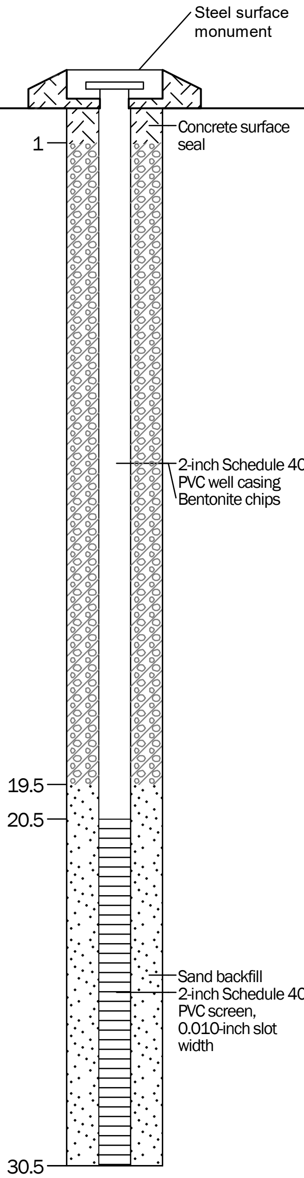
Figure A-1

Drilled — Start: 7/10/2023 — End: 7/10/2023	Total Depth (ft): 30.5	Logged By: AA Checked By: LJS	Driller: Pacific Drilling Co. — Drilling Method: Hollow-stem Auger
Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: Truck-mounted HSA Drill Rig, Saber Cab MTXD		A 2-in well was installed on 7/10/2023 to a depth of 30.5 ft.
Surface Elevation (ft) / Vertical Datum: Undetermined	Top of Casing Elevation (ft)		Groundwater — Date Measured: 7/29/2023 — Depth to Water (ft): 10.5 — Elevation (ft):
Easting (X) / Northing (Y)	Horizontal Datum		

Notes: A 2.5-inch I.D. split barrel/ Modified California Sampler was used for Sample 5B

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0					1			SM	Brown silty fine to medium sand with trace angular gravel (loose to medium dense) (Quaternary undocumented fill [Qudf])		
									SM	Dark brown silty to poorly-graded fine to medium sand with trace clay (alluvium [Qal])		
						2						
	5			5		3 MC				Becomes light brown with trace coarse gravel and sand	12.8	
									SM	Excavates as: olive brown to light brown silty fine to medium sand with trace coarse sand and trace clay (decomposed, igneous rock [mixed granitics])		
									SM	Excavates as: olive brown silty fine to coarse sand with trace clay (dense, moist) (decomposed to intensely weathered)		
	10			48		4 MC					8.85	
										Becomes wet		
										Becomes intensely weathered at 14 feet		
	15		6 5	50/5" 50/4"		5A 5B				Becomes very dense		
	20		6	50/4"		6			SM	Excavates as: olive brown silty sand (very dense, wet) (intensely weathered, igneous rock [mixed granitics])		
									SM	Excavates as: olive brown to olive gray poorly-graded sand with silt (very dense, wet) (intensely to moderately weathered)		
	25		5	50/4"		7						
	30		10	50/4"		8						

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Boring with Monitoring Well B-1

GEOENGINEERS

Project: Camp Switzerland Lift Station and Pipes
Project Location: San Bernadino County, California
Project Number: 26258-001-00

Figure A-2
Sheet 1 of 1

Date Excavated 7/10/2023	Total Depth (ft) 5	Logged By AA/MDM Checked By LJS	Excavator Pacific Drilling Co. Equipment John Deer 310 SL	Groundwater not observed Caving not observed

Surface Elevation (ft) Vertical Datum — Undetermined	Easting (X) Northing (Y)	Coordinate System Horizontal Datum

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0–2				SM	Light brown silty fine to medium sand (loose to medium dense, dry) (colluvium)			
	2–5				SM	Excavates as: olive brown to gray silty fine to medium sand with trace clay (medium dense, moist) (decomposed to intensely weathered, igneous rock [mixed granitics])			
	~4.5		1			Becomes moderately weathered			Difficult excavating

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-1

GeoEngineers

Project: Camp Switzerland Lift Station and Pipes
Project Location: San Bernadino County, California
Project Number: 26258-001-00

Date Excavated	7/10/2023	Total Depth (ft)	5.5	Logged By AA/MDM Checked By LJS	Excavator Pacific Drilling Co. Equipment John Deer 310 SL	Groundwater not observed Caving not observed

Surface Elevation (ft) Vertical Datum	Undetermined	Easting (X) Northing (Y)	Coordinate System Horizontal Datum

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0–2				SM	Light brown silty fine to medium sand (loose to medium dense, dry) (colluvium)			
	2–5.5				SM	Excavates as: gray brown silty fine to medium sand with trace clay (medium dense, moist) (decomposed, igneous rock [mixed granitics])			
	5					Becomes intensely weathered			
	5.5		1			Becomes moderately weathered			Difficult excavating

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-2

GeoEngineers

Project: Camp Switzerland Lift Station and Pipes
Project Location: San Bernadino County, California
Project Number: 26258-001-00

Figure A-4
Sheet 1 of 1

Date Excavated	7/10/2023	Total Depth (ft)	12	Logged By	AA/MDM	Excavator	Pacific Drilling Co.	Groundwater not observed
				Checked By	LJS	Equipment	John Deer 310 SL	Caving not observed

Surface Elevation (ft) Vertical Datum	Undetermined	Easting (X) Northing (Y)		Coordinate System Horizontal Datum

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0–4				SM	Brown silty fine to medium sand with trace angular gravel and cobble (colluvium)			
	3–4	X	1 / %F			Becomes slightly clayey, abundant rootlets, indurated		17	
	4–9				SM	Excavates as: light brown silty sand (medium dense, moist) (decomposed, igneous rock [mixed granitics])			
	9–12				SM	Light brown poorly-graded sand (medium dense, moist) (intensely weathered, igneous rock [mixed granitics])			
	10–11	X	2 / %F					7	
	11.5–12	X	3			Becomes intensely to moderately weathered			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-3

GeoEngineers

Project: Camp Switzerland Lift Station and Pipes
Project Location: San Bernadino County, California
Project Number: 26258-001-00

August 4, 2023

Mr. Matt Martinez, PG, CEG
Engineering Geologist
GeoEngineers, Inc.
13220 Evening Creek Drive South, Suite 115
San Diego, CA 92128

Subject: **LABORATORY TEST RESULTS**
GEOENGINEERS - CAMP SWITZER LAND LIFT STATION AND PIPES
AGE Project No. 221 GS-22-E/GEOENGINEERS Project No. 26258-001-00

Dear Matt,

As per your request, we have performed laboratory test to evaluate the moisture content (ASTM D2216), % passing #200 sieve and sieve wash analyses (ASTM D422) of the samples which was delivered to our office.

A summary of the moisture content and % passing #200 sieve test results is shown in Table 1 on the next page. The particle size distribution curves for the sieve wash analysis are attached.

Table 1
Summary of Laboratory Test Results

Sample ID	Sample Type	Moisture Content (%)	% Passing #200
B-1 - S-3 @5'	SPT	12.8	Not requested
B-1 - S-4 @10'	SPT	8.8	Not requested
TP-3 - S-1	Bulk	Not requested	17%
TP-3 - S-2	Bulk	Not requested	7%

We appreciate the opportunity to be of service on this project. If you have any questions regarding the contents of this letter or need further assistance, please feel free to contact our office.

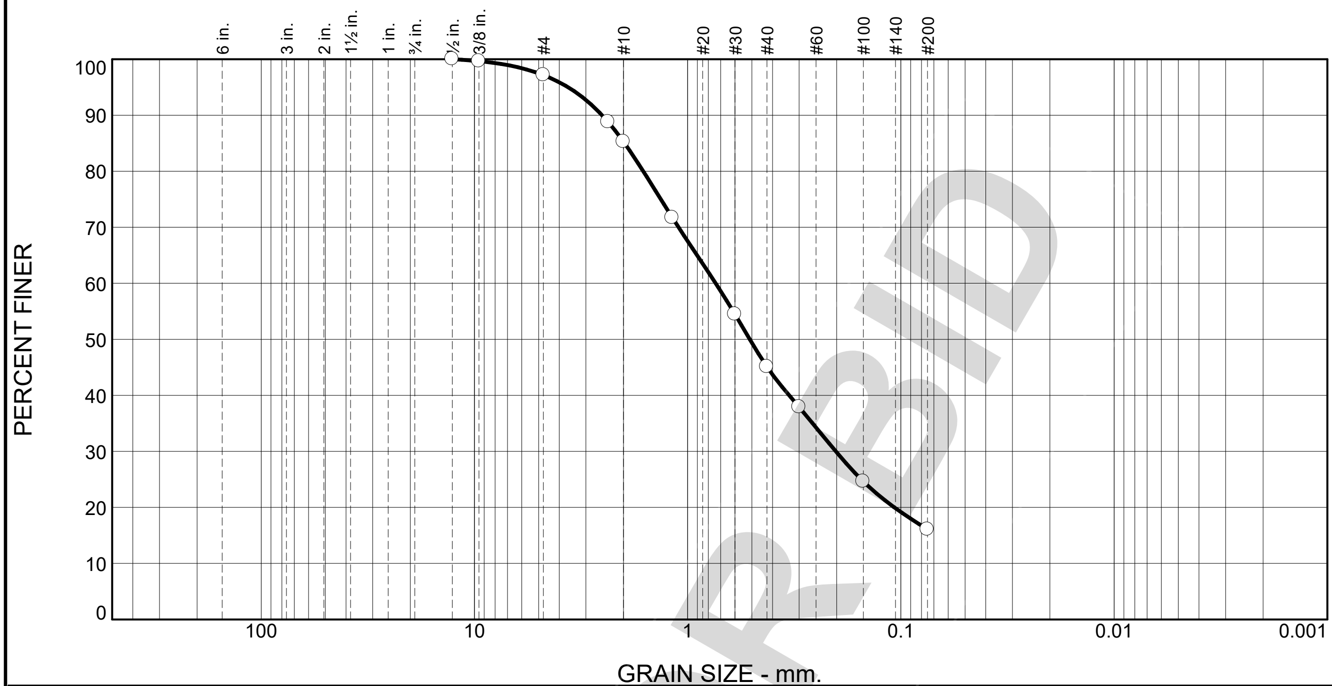
Sincerely,

ALLIED GEOTECHNICAL ENGINEERS, INC.

Sani Sutanto P.E., G.E.
Project Manager/Principal

NEB/SS/TJL:cal
Distr. (1 electronic) Addressee

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.8	11.9	40.2	29.0	16.1	

Test Results (ASTM D 422 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.5	100.0		
0.375	99.6		
#4	97.2		
#8	88.8		
#10	85.3		
#16	71.7		
#30	54.5		
#40	45.1		
#50	37.9		
#100	24.6		
#200	16.1		

* (no specification provided)

Material Description

Silty Sand (SM)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 2.5197 D_{85}= 1.9765 D_{60}= 0.7398
D_{50}= 0.5114 D_{30}= 0.2020 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 07/20/2023 **Date Tested:** 08/01/2023

Tested By: NEB

Checked By: SS

Title: PM

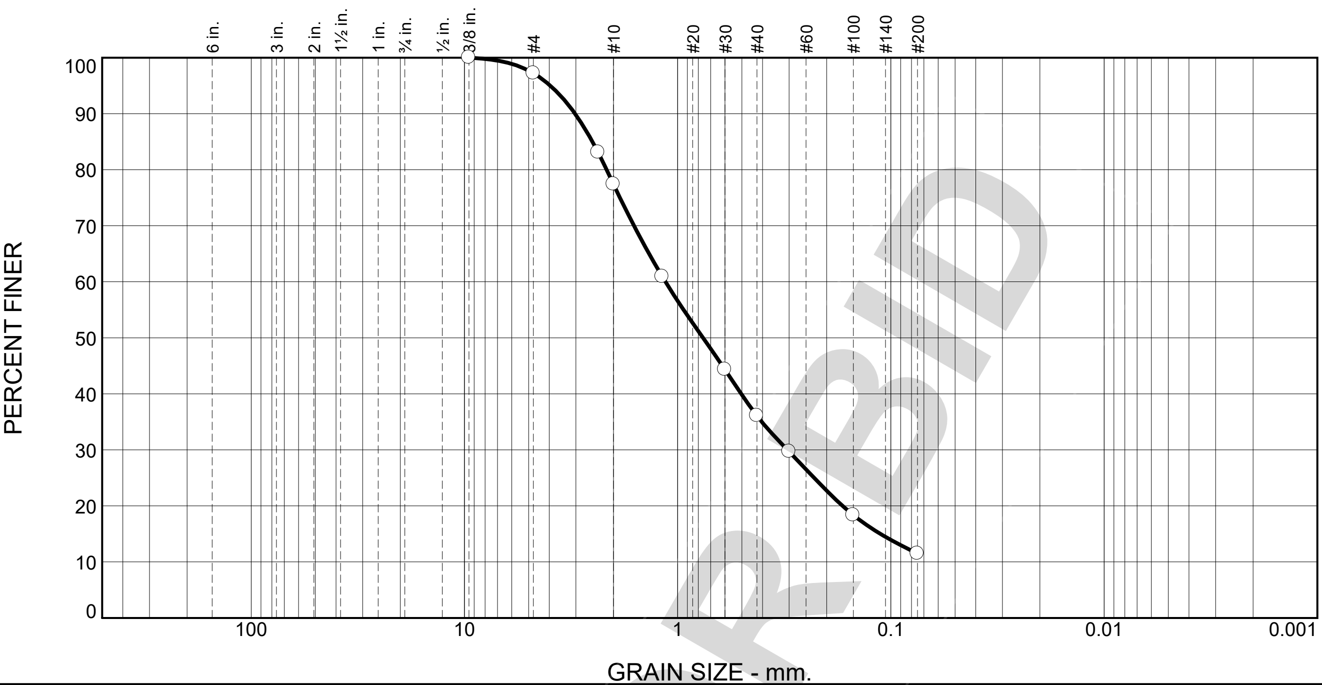
Sample Number: B-1, S-4 **Depth:** 10' **Date Sampled:** 07/20/2023

Allied Geotechnical Engineers, Inc.

Santee, CA

Client: GEOENGINEERS
Project: CAMP SWITZERLAND LIFT STATION AND PIPES
Project No: 221GS **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.8	19.8	41.3	24.6	11.5	

Test Results (ASTM D 422 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375	100.0		
#4	97.2		
#8	83.1		
#10	77.4		
#16	60.9		
#30	44.3		
#40	36.1		
#50	29.7		
#100	18.4		
#200	11.5		

* (no specification provided)

Material Description

Poorly Graded Sand With Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=**

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 3.0298 D_{85}= 2.5088 D_{60}= 1.1409
D_{50}= 0.7607 D_{30}= 0.3054 D_{15}= 0.1119
D_{10}= C_u= C_c=

Remarks

Date Received: 07/20/2023 **Date Tested:** 08/01/2023
Tested By: NEB
Checked By: SS
Title: PM

Sample Number: B-1, S-7 **Depth:** 20' **Date Sampled:** 07/20/2023

Allied Geotechnical Engineers, Inc.

Santee, CA

Client: GEOENGINEERS
Project: CAMP SWITZERLAND LIFT STATION AND PIPES
Project No: 221GS **Figure**

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of The San Bernardino County, Department of Public Works - Special Districts, Kimley-Horn, and their authorized agents. This report may be made available to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with which there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report Is Based on a Unique Set of Project-Specific Factors

This report has been prepared for the Camp Switzerland Lift Station and Pipes project located in San Bernardino County, California. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- Not prepared for you;
- Not prepared for your project;
- Not prepared for the specific site explored; or
- Completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- The function of the proposed structure;
- Elevation, configuration, location, orientation or weight of the proposed structure;
- Composition of the design team; or

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- Project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the geologic site reconnaissance and geophysical survey was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on the recent geologic site reconnaissance and geophysical survey at the site, as described herein. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of

Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.

GeoEngineers