APPENDIX D

PRELIMINARY GEOTECHNICAL REPORT, PROPOSED HEALTH PLAN OF SAN JOAQUIN – BEWELL

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PRELIMINARY GEOTECHNICAL REPORT
PROPOSED HEALTH PLAN OF SAN JOAQUIN – BEWELL
NORTHWEST CORNER OF W. HOSPITAL ROAD AND S. EL DORADO STREET
FRENCH CAMP, CA

Prepared for Boulder Associates

November 17, 2023

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STOCKTON
3428 Brookside Rd.
Stockton, CA 95219
t: 209.943.2021

SAN JOSE
111 N. Market St., #300
San Jose, CA 95113
t: 408.754.2021

SACRAMENTO
1164 National Ave, #20
Sacramento, CA 95834
t: 916.520.2777

MODESTO
101 Sycamore Ave, #100
Modesto, CA 95354
t: 209.762.3580

SIEGFRIED

November 17, 2023

To: Darci Hernandez, AIA, LEED AP | Principal
BOULDER ASSOCIATES
300 Spectrum Center Drive, Suite 730
Irvine, CA 92618

RE: PRELIMINARY GEOTECHNICAL REPORT FOR PROPOSED HEALTH PLAN OF SAN JOAQUIN – BEWELL NORTHWEST CORNER OF W. HOSPITAL ROAD AND S. EL DORADO STREET FRENCH CAMP, CA

Dear Darci,

We have completed our geotechnical report for the proposed Health Plan of San Joaquin – Be Well project located at the northwest corner of W. Hospital Road and S. El Dorado Street, French Camp, California. The purpose of our study was to explore the subsurface soil and groundwater conditions at the site to provide preliminary geotechnical engineering recommendations related to foundation design and earthwork construction.

Based on our study and on a preliminary basis during this validation phase, the site conditions are suitable for design and construction of the subject project from a geotechnical engineering perspective. The primary geotechnical features to be considered during final design and construction are:

- Site Conditions
  - The presence of an overgrowth of vegetation, tall brush, and trees across the site
  - The presence of organics in the upper 6 to 12 inches of the surface
  - The presence of undocumented fills in the upper 1 to 2 feet of the surface from prior agricultural use and backfill of previously demolished structures
  - The presence of trash and other non-deleterious matter (i.e., golf balls)
  - Stockpiles of soil, asphalt grindings, and concrete rubble on the western portion of the site
  - Imagery that suggested an unidentified feature was located at the northwest corner of the site
  - Remnants of flatwork from the prior golf course driving range usage at the site
- Building layouts are preliminary at this time but we assume typical single to two story structures will be light to moderately loaded with maximum column loads and wall loads of 100 kips and 2 kips per lineal foot, respectively. We are not aware of the building type which could change the structural load conditions assumed. Confirmation will be required during final design.

We make preliminary design and construction recommendations to address the adverse effects of these conditions with the design team during the validation Stage of the project. Once the Validation Stage is complete and the project moves into schematic design and design development, a final geotechnical report can be prepared when final site configuration and building type and loads are known. Additional field explorations and laboratory testing will be advanced within building footprints when they are finalized.

SIEGFRIED

We appreciate the opportunity to collaborate with you and the design team on this project. If additional information is needed or if there are inquiries in this report, please do not hesitate to contact me.

Sincerely,

REGISTERED PROFESSIONAL ENGINEER
BRADFORD LEE QUON
GE 2841
EXP. 6/30/2024
GEOTECHNICAL
STATE OF CALIFORNIA

Bradford Quon, GE
Geotechnical Manager | Principal
SIEGFRIED

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PART 1. INTRODUCTION

We have completed our geotechnical report for the proposed Health Plan of San Joaquin – Be Well project located at the northwest corner of W. Hospital Road and S. El Dorado Street, French Camp, California. The purpose of our study was to explore the subsurface soil and groundwater conditions at the site to provide preliminary geotechnical engineering recommendations related to foundation design and earthwork construction. The vicinity of the project is shown on Plate 1, Site Location Map.

1.1. PROJECT DESCRIPTION

The project site is located at the northwest corner of W. Hospital Road and S. El Dorado Street in French Camp, California.

The planned project will generally comprise the following:

- A series of single to two story structures
  - Building A – Community and Outpatient 22,300 sf (single story)
  - Building B – Urgent Care Services 27,700 sf (single story)
  - Building C – Residential Treatment Programs 42,500 sf (two story)
  - Building D – Future Residential 42,500 sf (two story) - Future
- Below grade infiltration facilities;
- Trash enclosures;
- Wet and Dry utilities;
- Landscaping;
- Exterior flatwork; and
- Flexible and Rigid pavements

No basements are planned currently. Load conditions are not known currently since the building types and configurations are not defined. We assume lightly loaded structures should not exceed 100-kip column or 2 kip per lineal foot wall loads. Actual load conditions should be verified during final design submittal.

1.2. SCOPE OF SERVICES

Our authorized scope of services was outlined in our proposal dated October 27, 2023, and authorized with Boulder Associates Consultant Service Order dated October 27, 2023. The scope of services generally included the following:

- Field exploration consisting of a series of drilled borings to maximum of approximately 51½ feet below the ground surface (bgs).
- Geotechnical testing to evaluate relevant index properties, engineering parameters (i.e., strength), corrosivity, and R value.
- Geotechnical engineering analysis to formulate conclusions and preliminary recommendations related to foundation design and earthwork construction.

1.3. SITE CONDITIONS

The site to be developed is located at the northwest corner of W. Hospital Road and S. El Dorado Street in French Camp, California as shown on Plate 1. The parcel is an irregularly shaped and relatively level parcel bounded by undeveloped land to the north, S. El Dorado Street to the east, W. Hospital Road to the south and the Interstate 5 embankment and right of way to the west. During our site exploration, we observed the following:

- Dense growth of brush approximately 4 to 5 feet tall blanketing the site and trees sporadic across the site as shown in Figure 1.
- Stockpiles of crushed rock, asphalt grindings and concrete rubble across the site (Approximate locations shown on Plate 2).
- A long stockpile of soil blanketed with dry grass east of the Interstate 5 embankment (Approximate location shown on Plate 2).
- Gravel surfaced access roads cut through the site.
- Existing single-story structure on the south portion of the site.
- Chain-link fencing around the perimeter of the site.
- Trash and miscellaneous non-deleterious materials around the surface of the site.
- Telephone and power poles around the site.

Figure 1: Typical growth of brush, vegetation, trees looking eastward on the site near the access road.

1.4. HISTORIC AERIAL IMAGES

We also reviewed historic aerial images provided at https://historicaerials.com from 1957, 1967, 1968, 1982, 1993, 1998, 2002, 2005, 2009, 2010, 2012, 2014, 2016, 2018, and 2020.

- The images from 1957, 1967, and 1968 show the existing site being relatively level and undeveloped, likely used for agricultural purposes
- The image from 1982 shows the first image where Interstate 5 appeared.
- The image 1993 indicates a darkened feature on the northwest corner of the site. The resolution of the image limits the clarity on the definition of the feature, but we interpret it as either a pond or a dense growth of brush and vegetation. See Plate 3.
- The image 1998 shows the feature identified in the image from 1993 as removed. A curved structure built just north of the existing structure was noted. See Plate 4.
- The image from 2002 shows the image noted from 1998 at a higher resolution with a concrete flatwork and structure on the south side of the site.
- The images from 2005 through 2012 still show the features noted in the image from 2002. See Plate 5.
- The image from 2014 does not show the features delineated in the 2002 through 2012 images.
- The images from 2018 show the access roads carved through the western side of the site. See Plate 6.
- No significant changes to the site were noted in the image from 2020.

PART 2. ENGINEERING GEOLOGY AND SEISMIC HAZARDS (GEOHAZARDS)

2.1. SITE CHARACTERIZATION

2.1.1. Local Geologic Conditions

Delattre, Graymer, Langenheim, and Knudsen, et. al. (2023) mapped the near surface deposits as Quaternary Modesto Formation (late Pleistocene) Upper Member, fine grained, map symbol, Qmub. This formation is commonly stratified alluvium of flood basins, lower fans, and interdistributary fan areas. The soil formed on these deposits are typically Dinuba, Landlow and Stockton Series.

2.1.2. Soil Survey

The Soil Survey of San Joaquin, California maps the western portion of the site as the Manteca fine sandy loam (Map symbol hhv2). The Manteca fine sandy loam soils are characterized as “moderately well drained” and a “very low” capacity to transmit water. The survey identifies these soils as Hydrologic Soil Group “C”. The eastern portion of the site is mapped as Veritas fine sandy loam (Map symbol hhxb). The Veritas fine sandy loam as “moderately well drained” and a “very low” capacity to transmit water. The survey identifies these soils as Hydrologic Soil Group “A”.

2.1.3. Geologic Hazard Zones

Geologic ground failures can occur within earthquake hazard zones. The California Geological Survey (CGS) Earthquake Zones of Required Investigation (https://maps.conservation.ca.gov) indicates the parcels to be developed:

- The parcel is NOT WITHIN an Earthquake Fault Zone
- The parcel has NOT been evaluated by CGS for liquefaction hazards
- The parcel has NOT been evaluated by CGS for seismic landslide concerns

2.2. GEOLOGIC HAZARDS

2.2.1. Expansive Soils

Expansive soils have the potential to impact the development where fluctuations in the moisture contents can cause unacceptable shrinkage and/or swell beneath buildings and/or flatwork. Controlling the moisture change will reduce this shrink-swell capability. Expansive soils are defined as having a Plasticity Index (PI) greater than 15 and an Expansion Index (EI) greater than 20. The near surface clay soils on the site were tested to have a PI of less than 15 indicating a low potential for expansion, thus we consider the expansive soils **not to be a design consideration.**

2.2.2. Weak/Soft Compressible Soils

Weak and soft, compressible soils are identified as having a very soft consistency. Soft compressible soils were not encountered in the borings advanced for this study. On this basis, weak/soft compressive soils are **not a design consideration.**

2.2.3. Corrosive Soils

We tested a bulk sample of soil for pH, minimum resistivity, chloride and sulfate presence, redox potential, and sulfides. The results are summarized in Table 2.1.

Table 2.1 – Soil Corrosivity

	CT643	CT643	CT422m	CT417	ASTM G200m	AWWA C105/A25.5
Sample Location	Soil pH	Min. Resistivity Ohm-cm (x1000)	Chloride ppm (%)	Sulfate ppm (%)	Redox Potential (mv)	Sulfides Presence
Bulk 1 (surface)	7.01	1.82	5.0 (0.00050%)	6.1 (0.00061%)	+ 268	negative
Bulk 2 (surface)	7.63	1.28	3.5 (0.00035%)	0.2 (0.00002%)	+255	negative

The Caltrans Corrosion Guidelines, Version 3.2 dated May 2021 considers a site to be corrosive if one or more of the following conditions exist:

Chloride concentration is 500 ppm or greater, sulfate concentration is 1500 ppm or greater, or the pH is 5.5 or less. Based on the Caltrans methodology, the site evaluated is **not considered corrosive.**

2.2.4. Flooding

The FEMA Flood Insurance Rate Map (FIRM) Map Number 06077C0470F indicates the entire parcel to be developed is mapped in an Area of Reduced Flood Risk due to Levee, Zone X. The potential for flooding is **not a design consideration for this project.**

2.2.5. Radon-222 gas

Radon is produced naturally as Radon-222 in gas form. Radon is a byproduct of the natural decay of uranium that is present in small quantities in several rock types such as granitic rocks of the Sierra Nevada and sediment derived rocks in the Sacramento Valley. Radon is soluble and can be transported in groundwater. When water-containing radon is exposed to air (by pumping or through a tap), radon can diffuse into the air where it can be inhaled.

The U.S. Environmental Protection Agency (EPA) (https://www.epa.gov/sites/default/files/2018-12/documents/radon-zones-map.pdf) lists San Joaquin County in Zone 3, the lowest potential radon hazard (less than 2 pCi/L) (U.S. EPA, n.d.). **Based on the zone assignment, we conclude that naturally occurring radon would not be considered a health hazard for this project.**

2.2.6. Naturally Occurring Asbestos

Naturally Occurring Asbestos (NOA) is hazardous to humans. Asbestos included six regulated naturally occurring minerals (actinolite, amosite, anthophyllite, chrysotile, crocidolite, and tremolite). In California, asbestos minerals are most associated with ultramafic rocks and their derivatives, including Serpentine rock. Ultramafic rock are igneous rocks composed mainly of iron-magnesium silicates minerals hat crystallize deep in the earth's interior. By the time they are exposed at the Earth's surface, ultramafic rocks have typically undergone metamorphism, a process in which the mineralogy or the rock changes in response to the changing chemical and physical conditions. Asbestos is classified as a known human cancer-causing substance by local, State, and Federal health agencies and is known to cause chronic respiratory diseases. Asbestos fibers may be released into the air because of activities which disturb NOA-containing rocks or soils. Asbestos minerals can fragment into small fibers that readily suspend in the air and are of a size visible only under a microscope. Breathing these small fiber fragments may result in an increased risk of respiratory disease or cancer in exposed individuals.

The Department of Toxic Substances Control (DTSC) has developed the Interim Guidance, Naturally Occurring Asbestos at School Sites, revised 9/24/2004. The guidance document provides a four-step process to assist school districts and their consultants in conducting environmental assessments, investigations, and response actions (if needed) at new or expanding school sites with potential NOA. Step 1 is the potential identification of NOA through the performance of a Phase I Environmental Site Assessment (Phase I ESA). If NOA is potentially identified, environmental sampling and analysis will be needed as part of the development of a Preliminary Environmental Assessment (PEA.) The guidance document continues to a mitigation phase and long-term operation and maintenance of the site.

Based on the review of the geologic maps, no ultramafic rocks are mapped near the property. We conclude that NOA is **not a design consideration.**

2.2.7. Hydrocollapse

Hydrocollapse occurs when loose, dry, sandy soils become saturated and settle. These materials are typically located in arid climates where wind and temperature have the greatest impact. The collapsible soils are prevalent in the Southern California area and in high desert areas. Loose granular soils were not encountered at the site; thus, we **consider hydrocollapse not to be a design consideration.**

2.3. SEISMIC HAZARDS

2.3.1. Historical Seismicity

The site is in low to moderate seismic region with most of the active faults located greater than 30 miles west of the project site within the San Francisco Bay Area. Toppozada, et. al., (2000) mapped the epicenters areas damaged by Magnitude (M) ≥ 5 Earthquakes. The mapping showed one significant earthquake located at the southwest corner of the county with a magnitude (M) 6.0 during the period around 1886.

The Unified States Geological Survey (USGS) maintains an interactive online portal at https://earthquake.usgs.gov/hazards/interactive to deaggregate the nearest earthquake faults that contribute the most towards the earthquake hazard. For this site, the deaggregated earthquake has a mean magnitude (M) of 6.24 occurring at a radius of 20.78 km (12.9 miles) west of the site. Table 2.2 provides some faults, the distance and direction from the site, and the magnitude it can generate. The faults presented are deaggregated contributors based on the Unified Hazard tool.

Table 2.2 – Faults, distance from site, and magnitude

Fault Name, Fault Model	Distance, km (miles)	Direction from Site	Magnitude (M)
Greenville (No.), FM32	41.27 (25.6)	West	7.14
Great Valley 07 (Orestimba), FM32	29.11 (18.1)	Southwest	6.45
Greenville (No.), FM31	41.33 (25.7)	West	7.14
Mount Diablo Thrust South, FM31	41.01 (25.5)	West	7.07
Great Valley 07 (Orestimba), FM31	29.11 (18.1)	Southwest	6.47

2.3.2. Fault Rupture

Fault rupture is a failure mechanism where the surface of the earth breaks along a fault. An active fault is defined as a fault that has ruptured in the last 11,000 years. There are no known active faults that trend and align towards the project site and the site is not located within an Alquist-Priolo Earthquake Fault Zone (formerly known as a Special Studies Zone). Therefore, we consider the potential for fault rupture at the site as negligible and **not a design consideration.**

2.3.3. Strong Ground Motion

For seismic design, mapped based spectral accelerations may be used provided the allowable exceptions are implemented in the project.

2.3.4. Liquefaction

Liquefaction is a phenomenon when saturated loose granular soils lose their strength and fail during a seismic event from an earthquake. The granular soils are typically clean and poorly graded and are typically younger deposits of Holocene age. For this project, groundwater was encountered in the boring B-4 at depth of about 24 feet bgs. The explorations encountered medium stiff to hard cohesive soils and medium dense silty sand and poorly graded sand. An approximate 5-foot-thick layer of loose silty sand layer encountered at depth of about 45 feet bgs was evaluated to have a potential liquefy. Since the site is of Pleistocene age and based on the findings of Ishihara (1985) where there is a sufficiently thick layer of dense to stiff material over a liquefiable layer, ground manifestations related to liquefaction are not likely. Therefore, we consider the potential for liquefaction at the site as negligible and **not a design consideration.**

2.3.5. Landsliding and Slope Stability

Landslides tend to occur in weak soil and rock on sloping terrain. The parcel to be improved is relatively level across the site, thus we consider the potential for landslides and slope instability as negligible and **not a design consideration.**

2.3.6. Tsunami and Seiche Inundation

A tsunami is a wave, or series of waves, generated by an earthquake, landslide, volcanic eruption, or even large meteor hitting the ocean. The sea floor experiences significant upward movement resulting in a rise of water at the ocean surface. The mound water moves away from the center in all directions as a tsunami (CGS, Note 55). The San Francisco Bay and Pacific Ocean is over 50 miles west of the French Camp. We conclude the risk of tsunami is negligible and **not a design consideration.**

A seiche is a temporary disturbance or oscillation in the water level of a lake or partial enclosed body of water, especially one caused by changes in atmospheric pressure. There are no known lakes or partial enclosed bodies of water located within a ½ mile of the site. We conclude the risk to seiche is negligible and **not a design consideration.**

PART 3. FINDINGS

3.1. SUBSURFACE CONDITIONS

3.1.1. Undocumented Fill

The borings B-1 through B-4 all encountered undocumented fill soils to depth of approximately 1 to 2½ feet bgs. The undocumented fill soils were likely a result of prior backfills and site grading during agricultural use. The fill was comprised generally of sandy lean clay (CL), silty sand (SM) with gravel. The undocumented fill soils were generally disturbed and loose in density.

3.1.2. Native Soil

Beneath the undocumented fills, the bores encountered native soils that were comprised of medium dense to dense silty sand (SM) and poorly graded sand (SP-SM), very stiff to hard silt and sandy silt (ML), and very stiff to hard sandy lean clay (CL) to the maximum depths explored at approximately 51½ feet bgs. The silt at 11 feet bgs was tested to have an unconfined compressive strength of 53.4 psi.

Details of the subsurface conditions are detailed on the boring logs presented in Appendix A.

3.2. GROUNDWATER CONDITIONS

Static groundwater was encountered during drilling of the bores advanced for the project as shown in the Table 3.1.

Table 3.1 – Measured Groundwater Depths Below Ground Surface (bgs)

Boring	Boring Depth, bgs (ft)	Depth to Groundwater, bgs (ft)	Comments
B-1	21½	---	Not encountered during drilling
B-2	16½	---	Not encountered during drilling
B-3	16½	---	Not encountered during drilling
B-4	51½	24	Encountered at time of drilling

Variations in groundwater levels may occur due to variations in ground surface topography, subsurface geologic conditions and structure, seasonal rainfall, local irrigation practices, new construction, and/or other factors beyond our control.

The California Department of Water Resources maintains a database of groundwater levels from well sites drilled in the vicinity for the Sustainable Groundwater Management Act (SGMA). The website https://storymaps.arcgis.com lists the following wells in proximity to the site with the corresponding depth to groundwater. Table 3.2 presents a summary of the DWR wells and corresponding groundwater depths.

Table 3.2 – Groundwater Levels from DWR Wells				
Well Site	Distance and Direction	Ground Elevation (ft)	Measured Depth to Water (ft)	Last Measurement Date
378972N1212936W003	0.8 miles NE	15.00	14.40	4/17/23
378787N1212825W001	0.5 miles SW	18.31	20.16	11/3/23

Based on the groundwater levels encountered during this study and the data reviewed from the available DWR wells, groundwater is not expected in the upper 14 feet of the surface and **not expected to be a design consideration unless deep excavations for utilities approach that depth.**

PART 4. CONCLUSIONS

Based on our understanding of the project and our findings, we conclude the project is feasible for design and construction from a geotechnical engineering perspective. Based on our findings, we conclude the following items should be addressed during final design and construction. Preliminary recommendations are presented in Part 5 of this report.

4.1. SITE CONDITIONS

The site conditions that should be addressed during design development and construction include the following presented in this section:

4.1.1. VEGETATION, TREES, BRUSH, AND ORGANICS NEAR THE SURFACE

During the time of our field study the site was blanketed with a dense growth of surface brush, trees, and vegetation. We also encountered organics scattered in the upper 1 to 2 feet of the surface. These deleterious materials should be removed outside of the construction limits and not be allowed for reuse within engineered fills.

4.1.2. UNDOCUMENTED FILLS

Undocumented fills from backfills of prior structures or abandoned utilities especially during demolition activity at the site may cause undesirable settlement unless they are mitigated. We recommend that potentially loose/soft soils (undocumented fill) be overexcavated to expose firm native soils and recompacted to provide an engineered fill for support of building slabs-on-grade or pavements.

- Within the proposed main building, we recommend all foundations and abandoned utilities be completely removed during demolition and replaced with compacted engineered fill. Foundations should bear in recompacted engineered fill or undisturbed native soil. Refer to Section 5.10.
- For lightly loaded, nonstructural elements such as trash enclosures, exterior flatwork, and pavements overexcavation is not necessary. However, the contractor should adhere to the grading requirements presented in Section 5.10.

4.1.3. TRASH AND NON-DELETERIOUS MATTER

Trash and non-deleterious matter from homeless encampments was observed throughout the site near the surface. These materials should be removed completely outside of the construction limits and not be allowed in fills.

4.1.4. STOCKPILES OF SOIL, ASPHALT GRINDINGS, CRUSHED ROCK, CONCRETE RUBBLE

We observed stockpiles of soil, asphalt grindings, crushed rock, and concrete rubble at the site and as shown on Plate 2.

Interviews with personnel maintaining the site indicated this material was reportedly placed during demolition of the former football field at the University of the Pacific. The stockpile is currently covered in weeds. The soils encountered appear to meet the requirements of engineered fill and may be used onsite provided the surface weeds/vegetation are removed and the material is moisture conditioned as engineered fill. Further testing will be required onsite to confirm applicability for reuse as engineered fill.

The stockpile of asphalt grindings will not meet the requirements of aggregate base, aggregate subbase, or engineered fill. It may likely be spread and uniformly blended with the soil stockpile provided it meets the requirements of engineered fill outside of the building footprints. Testing should be performed to verify it meets the requirements of engineered fill prior to usage. Deleterious or non-deleterious materials within the grindings should be removed prior to usage.

The stockpile of crushed rock will not meet the requirements of aggregate base, aggregate subbase, or engineered fill. It may likely be spread and uniformly blended with the soil stockpile provided it meets the requirements of engineered fill outside of the building footprints. Testing should be performed to verify it meets the requirements of engineered fill prior to usage. Deleterious or non-deleterious materials within the grindings should be removed prior to usage.

The concrete rubble stockpile is not suitable for reuse as engineered fill. The rubble should be removed outside of the construction limits.

4.1.5. UNKNOWN FEATURE AT NORTHWEST CORNER OF SITE

Review of historic aerial images as shown on Plate 3 indicated the presence of what appears to be defined as an undefined feature at the northwest corner of the project site. We did not advance any bores in this area other than a shallow infiltration test near the boundary of the limits. Based on the resolution of the imagery, we interpret this feature to be either a pond that was backfilled or a dense grown of brush and vegetation. Future studies should advance borings or test pits in the area to better define the subsurface conditions in that area specifically if there are building structures located in that area.

4.1.6. PRIOR STRUCTURES ONSITE

Review of historic aerial images as shown on Plate 3, 4, and 5 indicated the presence of some surficial structures on the south side of the parcel. These structures appeared as flatwork and single-story structures. Interviews with personnel maintaining the site indicated the site was formerly used as golf driving range facility. This supports the presence of golf balls scattered around the site. Due to the dense growth of brush around the site, the current flatwork is not apparent or visible, so we presume it was either demolished and backfilled, or buried. Future studies should verify if these features are present on site with test pits. If buried structures are exposed during construction, they should be completely removed and replaced with engineered fill.

4.2. BUILDING TYPE AND LAYOUT

The building layouts are preliminary as of preparation of this report. The construction type is not known but we assume typical single to two story structures will be light to moderately loaded with maximum column loads and wall loads of 100 kips and 2 kips per lineal foot, respectively. Taller structures may be heavier than that assumed and thus will require specific studies to evaluate the subsurface conditions at those areas to determine the appropriate subsurface preparation and foundation type.

4.3. EXPANSIVE SOILS

Expansive soils have the potential to shrink and swell due to fluctuations in the moisture content. This is prevalent especially when expansive soils are left untreated at the surface and may potentially cause undesirable movement and distress within flatwork areas or foundations. The materials tested are non to low plasticity based on Atterberg Limits testing.

During rough and finish grading, it is possible that expansive soils are encountered elsewhere onsite. If encountered, we recommend these “expansive soils” where exposed adhere to the moisture conditioning and compaction requirements recommended in this report. This would require site expansive soils to be moisture conditioned to at least 3 percent above the optimum moisture content and compacted to a minimum of 88 percent and a maximum of 92 percent relative compaction based on the ASTM D1557 test method. **It is essential that the moisture content be maintained until it is covered by the next layer of engineered fill, baserock, flatwork, or other material. Additionally, we recommend expansive soils, if encountered, not be allowed in the upper 12 inches of building pads. The upper 12 inches of building pads should consist of non-expansive soils or lime treated subgrade.**

For specific earthwork recommendations, refer to Section 5.10 through 5.12.

4.4. OTHER CONCLUSIONS

A sample was tested for pH, minimum resistivity, chloride, and sulfate presence. The sample was also tested for redox potential and the presence of sulfides. The test results on the single sample indicate that the site soil is not in a corrosive environment. Groundwater was encountered at about 15 to 16 feet below the ground surface of the explorations advanced. Based on the review of the existing available groundwater elevation data and that obtained from this study, we conclude that groundwater is not likely to impact design unless excavations approach 15 feet bgs in depth.

PART 5. PRELIMINARY RECOMMENDATIONS

5.1. SHALLOW SPREAD FOUNDATIONS

5.1.1. Allowable Design Criteria

Shallow spread foundations may be incorporated when designed according to the following parameters presented in Table 5.1. Shallow spread foundations could apply for light to moderately loaded buildings with maximum column and wall loads of 100 kips and 2 kips per lineal foot, respectively. Heavier structures may require analysis for viability of shallow spread foundations or consideration of deepened foundations.

Table 5.1 – Shallow Foundation Design Criteria for Light to Moderately Loaded Structures

Criteria	Variable	Design Criteria	Comments
Minimum Continuous Foundations Depth	D	18 inches	Note 1
Minimum Spread Foundations Depth	D	18 inches	Note 1
Minimum Width	B	12 inches	
Allowable Bearing Capacity	q_a	3,000 psf	Note 2 and 3
Estimated Total Settlement	S_{total}	1 inch	
Estimated Differential Settlement	S_{diff}	½ inch in 20 feet	Based on Risk Category IV
Allowable Passive Pressure	P_p	270 pcf	Note 5
Allowable Friction Factor	μ	0.45	Note 5

[1]Depth of footing is measured from the lowest ground elevation to the base of the footing and does not include under slab materials (i.e., capillary break gravel and sand, or aggregate base).
[2]Allowable bearing capacity may be increased by 500 psf for each additional foot of embedment to a maximum of three times the designated value. The allowable bearing capacity is a net value so the weight of the foundation extending below grade may be disregarded when computing dead loads. The allowable bearing capacity is based on a factor of safety of 3 and is applicable to dead plus live load combinations. This value may be increased by 1/3 for short-term loading due to wind or seismic forces.
[3]Based on footings bearing over a recompacted engineered fill or firm native soil for the light to moderately loaded structures. Footings for non-structural uses such as for signs or trash enclosures, etc., do not require overexcavation but instead recompaction underneath footings per this report.
[4]Total settlement is anticipated to occur rapidly and should be essentially complete following initial application of the loads.
[5]Passive pressure and friction factor are allowable values based on a safety factor of 1.5. The upper 1 foot of soil should be neglected for passive pressure, unless it is confined by exterior slabs, slabs on grade, or pavements. The structural engineer should evaluate if additional safety factors are applicable.

5.1.2. Lateral Resistance

Resistance to lateral loads may be provided from frictional forces between the bottom of the footing and the underlying soils, and by passive soil resistance against the sides of the foundations. If moisture barriers or other substances are placed beneath footings, the coefficient of friction can be significantly lower. The passive pressure should be neglected to a depth of 1 foot where the ground adjacent to the foundation is not covered by a slab or pavement. Lateral resistance parameters presented in Table 5.1 are allowable with a safety factor of 1.5 applied. The appropriate factor of safety should be determined by the project Structural Engineer. **We assume passive pressure and friction would occur simultaneously so may be combined without reduction.**

5.1.3. Seismic Ties

As outlined in CBC 1809A.13, where a structure is assigned to Seismic Design Category D, E, or F, individual spread footings founded as Site Class E or F shall be interconnected by ties. Unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade, ties shall be capable of carrying, in tension or compression, a force equal to the lesser of the product of the larger footing design gravity load times the seismic coefficient, S_{DS}, divided by 10 and 25 percent of the smaller footing design gravity load.

5.1.4. Construction Considerations

Foundation excavations should be firm, neat, and clean of debris, loose or soft soil, or water prior to placing any reinforcement. All footings excavations should be observed by the project Geotechnical Engineer or their designated representative just prior to placing reinforcing steel or concrete to verify the recommendations presented herein are implemented during construction.

Additionally, footings may experience an overall loss of bearing capacity or an increased potential for settlement when located near existing or future utility trenches. Further, stresses imposed by the footings on the utility lines may cause cracking, collapse, and/or a loss of serviceability. To reduce this risk, open or backfilled trenches parallel with a footing shall not be below a plane having a downward slope of 2 horizontal to 1 vertical (2:1) slope from a line 9 inches above the bottom edge of the footing and not closer than 18 inches from the face of the footing. When pipes cross under footings, the footings shall be specially designed. This may require encasement of the pipe with lean concrete. Pipe sleeves shall be provided where pipes cross through footings or footing walls and sleeve clearances shall provide for possible footing settlement but not less than 1 inch all around the pipe.

5.2. RETAINING WALLS

We recommend retaining structures be designed for active pressures (i.e., cantilever conditions) or at-rest pressure if it is braced at the top (as in a roof connection) presented in Table 5.2.

5.2.1. Active and At-Rest Pressure

Table 5.2 – Lateral Earth Pressures

Condition	Lateral Earth Pressure	Drained Case[1,3]	Undrained Case[2,3]
Active Case	P_a	35	n/a, deep groundwater
At – Rest Case	P_o	55	n/a, deep groundwater
Seismic Increment	P_{AE}	$9 \times H^2$ (psf)	

[1]Drained case assumes fully drained conditions and level backfill. Undrained cases assume hydrostatic conditions.
[2]Undrained cases assume hydrostatic conditions based on buoyant unit weights of soil.
[3]Lateral earth pressures are presented as ultimate.

No additional surcharge stresses were included in the pressures noted above. Surcharge pressures will depend on the load conditions (i.e., equipment and construction loads such as material or soil stockpiles, and distance from wall where load is applied, etc.) If specific surcharge pressures need to be considered, additional analysis will be required with the load conditions given.

In general, walls subject to surcharge loads should be designed for an additional uniform lateral load pressure equal to one-third the anticipated surcharge loads for unrestrained walls and one-half the anticipated surcharge loads for restrained walls. The project engineer should be consulted with to confirm applicable values.

5.2.2. Seismic Design for Retaining Walls

Section 1807A.2.2 of the 2022 California Building Code notes for structures assigned to Seismic Design Category D, E or F, the design of retaining wall supporting more than 6 feet of backfill height shall incorporate the additional seismic lateral earth pressure.

Under seismic conditions, the active incremental seismic force along the face of a retaining wall should be added to the static active pressures, and can be calculated as follows:

$$\Delta P = 9 \times H^2$$

H is the design height of the wall (in feet) and ΔP is the active incremental seismic force in pounds per foot of wall. This force has a horizontal direction and should be applied at 0.6 x H from the base of the wall.

5.2.3. Wall Drainage

Where retaining walls are designed to be drained, drainage may be provided using a 4-inch-diameter perforated pipe embedded in Caltrans Class 2 permeable material, or free-draining gravel surrounded by synthetic filter fabric. The thickness of the drain blanket should be at least 12 inches. As an alternative, prefabricated synthetic wall drain panels can be used. The drain blanket should extend from the bottom of the wall to about one foot below the finished grades at the top of the wall. The upper one foot of wall backfill should consist of onsite compacted clayey soils. Drainage should be collected by a perforated pipe and directed to an outlet approved by the Civil Engineer. Subdrain pipe, drain blanket and synthetic filter fabric should meet the minimum requirements presented herein. Clay soils should not be incorporated into retaining wall fills.

5.3. SEISMIC DESIGN CRITERIA

The structural engineer should confirm the design of the proposed improvements is in accordance with the requirements of governing jurisdictions and applicable building codes in addition to the appropriate values to use for this structure. Map-based design criteria presented in this section are based on entering the site coordinates (latitude and longitude), the risk category, and the Site Class. Based on our experience in the area, the site may be classified as Site Class D. Table 5.3 presents the seismic design parameters for the site in accordance with the 2022 CBC and ASCE7-16 guidelines using the SEAOC/OSHPD Seismic Design Maps Tool.

Table 5.3 – Seismic Design Criteria per 2022 California Building Code and ASCE 7-16

Reference	Seismic Parameter	Value
Google Earth	Latitude	37.887147
Google Earth	Longitude	-121.278142
Table 20.3-1	Site Class	D
Table 1.5-1	Risk Category	III
Table 11.4-1	Site Coefficient for Short Period, F_A	1.193
Table 11.4-2	Site Coefficient for Long Period, F_v	2.012*
Figure 22-7	Peak Ground Acceleration, PGA	0.32g

Table 11.8-1	Site Amplification Factor, F_{PGA}	1.28
Equation 11.8-1	Peak Ground Acceleration, PGA_M	0.409g
Figure 22-1	Mapped MCE_R Spectral Response Acceleration at 0.2-second period, S_s	0.767g
Figure 22-2	Mapped MCE_R Spectral Response Acceleration at 1.0-second period, S_1	0.294g
Equation 11.4-1	Site-Adjusted MCE_R Spectral Acceleration at 0.2-second period, S_{MS}	0.915g
Equation 11.4-2	Site-Adjusted MCE_R Spectral Acceleration at 1.0-second period, S_{M1}	0.887g**
Equation 11.4-3	Design Spectral Response Acceleration at 0.2-second period, S_{DS}	0.610g
Equation 11.4-4	Design Spectral Response Acceleration at 1.0-second period, S_{D1}	0.592g
Table 11.6-1	Seismic Design Category for Short Period Response Acceleration	D
Table 11.6-2	Seismic Design Category for 1-s Period Response Acceleration	D
	Long-period transition, T_L	12 sec
	Short-period transition, $T_S = S_{D1}/S_{DS}$	0.971 sec

[1]A site-specific response spectra and ground motion study was not performed for this study. The structural engineer should confirm the appropriate values for use on the project during foundation design. If a site-specific hazard analysis is required, please contact our firm.
[*]F_v was determined per ASCE 7-16, Supplement 3,Table 11.4-2, assuming the exceptions allowed by Section 11.4.8 are implemented.
[**]S_{M1} was determined per ASCE 7-16, Supplement 3, and increased by 50% for all applications of S_{M1} in the Standard.

5.4. CORROSIVITY

The American Concrete Institute (ACI) 318 code, Table 19.3.2.1 is reproduced in Table 5.4 and indicates the requirements for concrete by exposure class. Refer to the commentary in the referenced ACI for additional comments and notes included in the table.

Table 5.4 – Soil Corrosivity

Exposure Class	Maximum w/cm	Minimum f'c, psi	Cementitious Materials - Types			Calcium Chloride Admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	N. T. R.[1]	N. T. R.	N. T. R.	N. R.[2]
S1	0.50	4000	II	Types with (MS) designation	MS	N. R.
S2	0.45	4500	V	Types with (HS) designation	HS	Not permitted
S3 – Option 1	0.45	4500	V plus pozzolan or slag cement	Types with (HS) designation plus pozzolan or slag cement	HS plus pozzolan or slag cement	Not permitted
S3 – Option 2	0.40	5000	V	Types with (HS) designation	HS	Not permitted

[1] N. T. R. – No Type Restriction
[2] N. R. – No Restriction

Table 5.5 – Corrosivity Scale by AWWA[1] C-105 Standard				
Soil Parameter Resistivity (ohm-cm)	Assigned Points		Soil Parameter pH	Assigned Points
< 700	10		0-2	5
700-1000	8		2-4	3
1000-1200	5		4-6.5	0
1200-1500	2		6.5-7.5	0
1500-2000	1		7.5-8.5	0
>2000	0		>8.5	3
Soil Parameter Redox Potential	Assigned Points		Soil Parameter Sulfides	Assigned Points
>100	0		Positive	3.5
50-100	3.5		Trace	2
0-50	4		Negative	0
<0	5			
Soil Parameter - Moisture			Assigned Points	
Poor drainage, continuously wet			2	
Fair drainage, generally moist			1	
Good drainage, generally dry			0	

[1]American Water Works Association (AWWA)

Based on the testing performed, the soils evaluated would classify as a Class "S0" where there are no type restrictions for the cementitious materials used.

For cast iron alloy pipes, the American Water Works Association (AWWA) developed a numerical soil corrosivity scale to identify the severity by assigning points for different variables such as the resistivity, pH, Redox Potential, Sulfides, and Moisture. The AWWA C-105-point standard is reproduced for reference in Table 5.11.

Based on the corrosivity test performed and our assumption of "fair drainage, generally moist" conditions, we assign a point value of less than 10, indicating a low corrosive rating for the site. When total points on the AWWA scale are at least 10, the soil is classified as corrosive to cast and ductile iron pipes and use of cathodic protection is often recommended.

The results provided were based on a single sample tested on the site. Other soil on the site may be corrosive. We do not practice Corrosion Engineering and a complete assessment of the corrosion potential of the site soil was not within our scope. For long term, specific corrosion control design recommendations, we recommend a California-registered Corrosion Engineer evaluate the corrosion potential of the soil on buried concrete structures, steel pipe coated with cement mortar, and ferrous metals.

5.5. INTERIOR SLAB-ON-GRADE

Interior slabs-on-grade for normal pedestrian traffic and office use areas should be a minimum of 5 inches and verified by the designer. The slab-on-grade may be designed with a subgrade modulus of 50 pci assuming an engineered fill pad. Moisture barriers should be considered if moisture sensitive floor coverings are used. If a moisture barrier is to be laid to protect floor finishes, we recommend it be a flexible membrane at least 15 mils thick, such as Stego® Wrap, complying with ASTM E 1745-97 "Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill Under Concrete Slabs", and placed in accordance with ASTM E 1643-98 "Standard Practice for Installation of Water Vapor

Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs". A layer of crushed rock at least 4 inches thick should underlie the vapor retarding membrane. The rock shall be clean, crushed, and free-draining having a nominal 1-inch maximum size with less than 3 percent passing the No. 200 sieve.

5.6. EXTERIOR FLATWORK

Exterior flatwork for pedestrian traffic should be at least 4 inches thick and placed over 6 inches of road base materials over a subgrade prepared in accordance with the recommendations of this report. Lime treatment can be used to address the expansive soils. If lime treatment is not used, we strongly emphasize the subgrade preparation should be strictly adhered to specifically for moisture conditioning. For shrinkage control, we recommend the slabs be reinforced with minimum No. 4 bars at 18 inch-centers, both ways, centered on "dobies" or similar supports at middepth throughout the slab, and, due to the expansive site soils, bars should continue through joints. However, the slabs should not be pinned to the building walls. The civil engineer should determine the final slab thickness, reinforcing, and joint spacing based upon the anticipated loads.

5.7. FLEXIBLE AND RIGID PAVEMENTS

5.7.1. Flexible (Asphalt Concrete) Pavements

Laboratory testing from one (1) bulk soil sample taken from the proposed pavement area resulted in R-Values (Resistance Values) of 33. Asphalt and base course materials should meet the requirements of the *Caltrans Standard Specifications, latest edition*. Pavement sections per the empirical methods presented in the California Highway Design Manual are shown below. Pavement sections are based on a reduced subgrade R-value equal to 30 to account for potential variability across the site.

Table 5.6 – Recommended Flexible Pavement Sections

Traffic Index[1]	Asphalt Concrete (in)	Class 2 Aggregate Base (in)	Lime Treated Subgrade (in)	Geogrid
4	2½	4	---	---
5	3	6	---	---
6	3	9	---	---
7	3	12	---	---
8	4	13	---	---
9	5	14	---	---

[1]Traffic Indices were assumed.

If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. Subgrade materials should be processed to a minimum depth of 12 inches below the Class II aggregate base and compacted to a minimum 95 percent of ASTM D1557 laboratory maximum dry density at or near the optimum moisture content. Class II Aggregate Base material should be compacted to 95 percent of ASTM D1557 laboratory maximum dry density at or near optimum moisture content. The base should meet the quality requirements outlined in Section 26 of the Caltrans Standard Specifications.

The pavement section is intended as a minimum. Positive site drainage should always be maintained. Water should not be allowed to pond or seep into the ground. If the average daily traffic (ADT) increases beyond that intended, as reflected by the assumed traffic designation, increased maintenance could be required for the pavement section. The project Civil Engineer should determine the Traffic Index appropriate for the project.

5.7.2. Rigid (Portland Cement Concrete) Pavements

Where rigidity of pavement is desired for areas designed for, high volume vehicular traffic, heavy maintenance or equipment traffic, entry driveways or trash enclosure slabs, we recommend using Portland cement concrete paving. The rigid concrete pavement section presented in Table 5.7 is based on a composite subgrade modulus of 150 pci, for light, moderate, and heavy-duty sections, respectively. The composite subgrade modulus considers the native subgrade and a specified thickness of aggregate base. The concrete thickness is based on a minimum concrete modulus of rupture of 550 psi. In addition, the driveway slabs should be designed with thickened edges at least twice the slab thickness. The design, applicable section, and thickness of rigid pavement slabs should be confirmed by the design professional.

Table 5.7 – Recommended Rigid (Portland Cement Concrete) Pavement Sections

Traffic Classification[1]	Rigid Concrete (in)	Class 2 Aggregate Base (in)	Total Section (in)	Notes
Light – ADTT[2] = 3	5	6	11	Note 3, 4, 5, 6
Moderate – ADTT = 10	5½	9	14½	Note 3, 4, 5, 6
Heavy – ADDT = 50	6	9	15	Note 3, 4, 5, 6

[1]Classification per the American Concrete Pavement Association based on Portland Cement Association (PCA) EB109P, 1984 .
[2]ADTT is the Average Daily Truck Traffic for both lanes of travel, over all lanes of traffic, and includes trucks with six tires or more (excluding panel and pickup trucks and other four tire vehicles).
[3]Dowels are not recommended unless rigid concrete pavement is greater than 6 inches
[4]Concrete thickness is based on 30-year design life WITH concrete curb and gutter or concrete shoulders. Add one inch thickness to concrete if based on 30-year design life WITHOUT concrete curb and gutter or concrete shoulders. A concrete modulus of rupture of 550 psi (minimum) is assumed.
[5]Based on a firm and unyielding subgrade where the upper 12 inches are compacted as recommended in this report for pavement subgrade.
[6]If subgrade is lime treated, reduce concrete thickness by ½ inch.

5.7.3. Construction Considerations for Pavements

Additional requirements and/or assumptions for pavements are outlined below:

- Baserock materials used should comply with the requirements outlined in Section 26 of the State Standard Specifications. We strongly recommend that baserock be a virgin, crushed aggregate product.
- Baserock should be firm and stable prior to placing asphalt and compacted to a minimum of 95 percent based on the ASTM D 1557 test method.
- Subgrade beneath paved areas shall be compacted to a minimum of 95 percent based on the ASTM D 1557 test method.
- Proof rolling of subgrade and of baserock with fully loaded water truck, or equivalent, should be performed under observation of our field representatives to detect for any instabilities of pavement subgrade and baserock following final grading. Proof rolling of subgrade should occur immediately (i.e., less than 24 hours) before placement of baserock. Baserock should be proofrolled immediately prior to placement of tack coat.
- Subgrade preparation is performed as outlined in the Earthwork sections of this report.

5.8. EARTHWORK

5.8.1. Site Preparation

Prior to any site grading, the existing concrete slabs, foundations, and surficial deleterious materials from previous use should be demolished and removed outside of the construction limits. These materials should not be incorporated into any structural fills. Vegetation and organics within grading limits should be stripped and removed offsite. The stripping should be performed to provide a subgrade with organic content less than 3 percent of organics and to the satisfaction of the geotechnical representative. We estimate the depth of stripping is approximately 3 to 4 inches across the site and could be deeper in areas with denser growth of brush. Trees and their root foundation should be removed entirely. No measurements were made on the root layers but based on the dense growth of the vegetation brush and trees throughout, it is anticipated that excavation and removal of the brush and/or trees will create large void spaces and disturb the existing ground. The cavities created by complete removals of the root balls from the brush and trees should be replaced with compacted engineered fill.

5.8.2. Site Grading

Prior to placing any fills, the exposed subgrade should be scarified 12 inches, moisture conditioned and mechanically compacted. Once the exposed subgrade is moisture conditioned and compacted, the new fill meeting the requirements of in this report should be moisture conditioned and placed horizontally in 8-inch maximum lifts, then compacted. Moisture content and the level of compaction will vary according to the definable feature. The acceptance criteria are presented in Section 5.13.

5.8.3. Engineered Fill

Imported engineered fill may be used and should be free of organic or other deleterious debris, non-plastic, and less than 3 inches in maximum dimension. Onsite soil may be used as engineered fill material provided it is processed and compacted as recommended in this report. **Expansive soils, if encountered, should not be allowed within the upper 12 inches of building pads. Clay exposed in the soils in the upper 12 inches of building pads should be removed and replaced with non-expansive fill or lime treated, subject to the Geotechnical Engineer of Record.** Specific requirements for engineered fill including the applicable test procedures to verify suitability are presented in Table 5.8.

Table 5.8 – Materials for Engineered Fill (Imported)		
Gradation		
Sieve Size	Percent Passing	Test Procedures
3 inches	100	ASTM[1] D6913 or ASTM D1140
¾ inch	80-100	ASTM[1] D6913 or ASTM D1140
No. 4	40-70	ASTM[1] D6913 or ASTM D1140
No. 200	More than 10	ASTM[1] D6913 or ASTM D1140
Test	Criteria	Test Procedure
Liquid Limit	Less than 40	ASTM D4318
Plasticity Index	Less than 15	ASTM D4318
Swell Test	Less than 4%	
Organic Content	Less than 3%	ASTM D2974
Expansion Index	Less than 20	ASTM D4829
Sand Equivalent	Greater than 10	CT[2]217

Notes
[1] ASTM = American Society for Testing and Materials Standards
[2] CT = California Test Method

If fill is to be imported from off-site, it should meet the requirements of engineered fill above and be non-corrosive and free of deleterious material. Any imported fill should be sampled by the project Geotechnical Engineer prior to being imported to evaluate its suitability for its intended use and to perform confirmatory testing listed above, if necessary.

5.8.4. Wet Weather and/or Unstable Soil Conditions

The in-situ moisture content of the site soil may increase after extended periods of rainfall. Soil subgrades may become saturated due to exposure to wet weather conditions. When wet soils are encountered, they should be remediated by aeration, removing and replacing with drier material, and/or chemically treated with lime or cement combinations. We should be contacted if these conditions are encountered.

5.8.5. Rat Slab for Foundation Working Surfaces

An alternative for aeration or removal of wet soils and replacement with engineered fill for mat foundations may consist of construction of a lean concrete slab at least 2 inches thick placed over a subgrade prepared in accordance with this report. The lean concrete slab should have a minimum compressive strength of 1000 psi. This slab would provide a dry working surface for construction of foundations.

5.9. EXCAVATIONS

5.9.1. Temporary Excavations and Excavatability

Pipelines, excavation, and earthwork following removal of paving and/or flatwork within trench zones can be performed with the typical conventional excavating and filling machines generally in use for such projects Soil on trench walls or bottoms should not be allowed to dessicate (dry out) or become saturated due to inclement weather. Ultimately, it is the Contractor's responsibility for implementing means and methods to protect exposed soil on the trench walls or bottom of excavations. If materials become saturated and cause sliding, toppling, subsidence and bulging, or heaving or squeezing conditions as defined by OSHA, remedial actions will be required to address the conditions. The Contractor and/or Geotechnical Engineer, or his representative shall periodically review the near-surface and subsurface materials when the conditions are encountered. As the site has variable materials, excavations should be addressed on an individual basis to meet the requirements established by OSHA. Temporary excavations may require shoring to meet these requirements.

5.9.2. General Considerations for Temporary Shoring (if needed)

During construction, the Contractor is responsible for maintaining safe excavations in accordance with OSHA guidelines. Where temporary shoring and internal bracing is used, it should be designed by a registered design professional experienced in shoring design.

A monitoring program should be implemented by the Contractor and set on existing permanent benchmarks or survey points as well as the installed shoring system to evaluate if any movement is occurring as the excavation continues. The instrumentation used, monitoring program workplan, and readings of survey points should be documented, submitted, and reviewed by the project team.

5.9.3. Bedding and Backfill Materials for Utility Trenches

Trench bedding and backfill should meet the meet stricter requirements outlined in the local jurisdictional requirements and the recommendations presented herein.

Trench backfills generally fall within two categories typically characterized as pipe zone backfill and trench zone backfill. The pipe zone backfill refers to the material in the immediate vicinity of the pipe and is often termed “shading”. Trench zone backfill refers to the material between the pipe zone backfill and the finished subgrade.

We do not recommend using coarse-grained sand and/or gravel for either pipe or trench zone backfill unless they are separated from the native soils by a non-woven geotextile fabric equivalent to Mirafi® 140N. This is due to the potential for soil migration into the comparatively large void spaces in these types of materials which will, over time, result in ground settlement.

5.9.4. Pipe Zone Materials

Pipe zone backfill should be placed loosely and then thoroughly tamped by hand-working the soil beneath the pipe’s spring line using shovels and by walking on three-inch loose lifts. It should extend to at least one foot above the crown of the pipe. We generally recommend against ponding or jetting or using mechanical compactors to densify pipe-zone backfill but requests for use in specific situations may be referred to the geotechnical engineer for consideration.

Piping with sensitive coatings should be designed to ensure that the outside dimension of the insulation or other coating is buried deeply enough below the road’s subgrade elevation and covered with appropriate thickness of shading that will protect the coating from construction damage. Pipes and their insulation should be located deeper than a foot below top of road subgrade/ underside of aggregate base course zone to minimize the possibility of insulation and pipe damage and/or corrosion when the road subgrade is scarified prior to compaction. Conflicts between these recommendations and the backfill requirements of pipe manufacturers should be referred to the project civil engineer for resolution.

Pipes should be encapsulated with clean sand at least 6 inches in each direction from the bottom of the trench to over the pipe.

5.9.5. Trench Zone Materials

The trench zone should be backfilled with onsite soil placed and compacted as recommended for engineered fill. As stated above, pipe manufacturers or design professionals may require special backfill materials. We recommend that the geotechnical engineer be included in the consideration of alternate backfill materials. Mechanical compaction is recommended; ponding or jetting of backfill should be avoided.

Based on the materials encountered during our investigation and the results of the laboratory test program performed on selected samples, the native materials appear to be suitable for use as backfill materials in the trench areas. However, consideration should be given if earthwork activities occur during the wet winter or early spring seasons where it is possible that moisture conditions could increase prior to trench excavations or earthwork which could render the materials difficult to compact. Consideration should be given for drying, mixing, and/or importing drier material or chemically treating the soil to facilitate compaction and meeting the requirements of engineered fill herein.

5.9.6. Protection of Existing Foundations and Buried Utilities

Where excavations are made next to foundations or buried utilities, the excavations should not be allowed to encroach to within a line projected downward at a slope of 2 horizontal to 1 vertical from a point 9 inches above the bottom of the foundation as outlined in CBC 1809.14. Each case may be specific, but the registered design professional shall determine the requirements for support and protection of the existing foundation and prepare site-specific plans, details, and sequence of work. Typical support means and methods may include underpinning, bracing, excavation retention systems, or other means. Where pipes cross under footings or encroach within the near surface of fills, the footings shall be specially designed. The existing utilities shall be protected. Pipe sleeves shall be provided where pipes cross through footings or footing walls and sleeve clearances shall provide for possible footing settlement but not less than 1 inch all around the pipe.

5.10. COMPACTION AND MOISTURE CONDITIONING SUMMARY

Site subgrade prior to placing fill, engineered fill and trench backfill, and pavement section materials meeting the criteria presented above should be placed in uniform, horizontal, loose lifts not exceeding 8 inches, and moisture conditioned and mechanically compacted as noted in Table 5.9. Jetting should not be allowed.

Table 5.9 – Compaction and Moisture Conditioning Summary

Area to be Compacted	Minimum Relative Compaction (RC) [1, 3]	Moisture Content[2] Required
Non-Expansive Engineered fill (Import)	≥95%	0 to 3% > optimum moisture
Subgrade prior to placing fill	≥95%	0 to 3% > optimum moisture
Expansive soils (in place compaction, if encountered)	88%<RC<92%	Min 3% > optimum moisture
Trench backfill[6]	88%<RC<92%	Min 3% > optimum moisture
Upper 12 inches of Trench backfill in paved areas	≥95%	0 to 3% > optimum moisture
Lime Treatment as Engineered Fill (if used)	≥95%[4]	Min 3% > optimum moisture[4]
Lime Treatment as Pavement Subgrade (if used)	≥95%[4]	Min 3% > optimum moisture[4]
Upper 12 inches of pavement subgrade	≥95%	0 to 3% > optimum moisture
Aggregate Baserock for pavement[5] section	≥95%	0 to 3% > optimum moisture

[1]Minimum relative compaction is a ratio of the in place dry density and the maximum dry density determined by the ASTM D1557 test method

[2]Moisture content is determined by ASTM D1557 for optimum moisture content and D6938, D1556, or D8167 for field determination by nuclear gauge. **Moisture content shall be maintained in its tested state until it is covered with the next lift of engineered fill, aggregate base, or flatwork. It shall not be allowed to dessicate or dry to below the moisture content requirements shown.**

[3]In place dry density and moisture content can be determined by ASTM D6938, D1556, or D8167.

[4]Optimum moisture content and maximum density determined by California Test Methods.
[5]The compaction requirement for aggregate baserock applies to both flexible (asphalt) and rigid (concrete) pavements.
[6]Fills greater than 5 feet should be compacted to a minimum of 95 percent for the entire depth.

5.11. DRAINAGE

To minimize moisture intrusion into foundation and slab subgrades, we recommend the ground surface slope away from the building pad and pavement areas in accordance with jurisdictional and/or local Building Code requirements toward the appropriate drop inlets or other surface drainage devices. These grades should be maintained for the life of the project. Building pads should also be designed such that the lowest adjacent grade surrounding the building is at or below the elevation of the building pad surface (at or below the bottom of the capillary break material beneath the floor slab. Landscaping after construction should not promote ponding of water adjacent to the structures.

5.12. INFILTRATION BASIN

Infiltration testing (converted by percolation methods) was performed at IN-1 on the at a depth of about 5 feet on the north side and indicated a design infiltration rate of 0.58 in/hr (8.61 gal/sf/day) based on a safety factor of 5. Side slopes are stable at 3:1 (H:V). For final design of other ponds elsewhere onsite, we recommend testing be performed at each basin to determine specific infiltration rates.

5.13. SOILS SPECIAL INSPECTION

Special inspection and tests of soils should be performed per Table 1705.6 of the 2022 California Building Code at a minimum. Specifically, these requirements include the special inspector to:

1. Periodically verify materials below shallow foundations are adequate to achieve the design bearing capacity.
2. Periodically verify excavations are extended to proper depth and have reached proper material.
3. Periodically perform classification and testing of compacted fill materials.
4. Continuously verify use of proper materials, densities and lift thicknesses during placement and compaction of fill.
5. Periodically inspect subgrade and verify the site has been prepared properly prior to placement of compacted fill.

5.14. FURTHER STUDIES AND CONSIDERATIONS FOR FINAL DESIGN

The preliminary recommendations presented herein are based on the current data and findings from the field exploration recently completed at the site. This field exploration included a series of drilled borings to depth of approximately 51½ feet bgs. We also advanced shallow borings around the perimeter to verify the near subsurface conditions. Laboratory testing was performed on samples collected from the field exploration. To develop specific criteria for preparing the final geotechnical report we recommend the following be performed:

1. Refine foundation design parameters and provide construction considerations for the selected foundation type.
2. Perform confirmatory testing of test pits to verify the extent of the existing buried structures.
3. Excavate test pits to verify extent of undocumented fills, and deleterious matter.
4. Perform additional infiltration testing at the proposed invert locations of the basins if others are identified.

PART 6. ADDITIONAL SERVICES

6.1. MODIFICATIONS TO THE GEOTECHNICAL ENGINEERING REPORT

The building layout, load conditions, and/or design elevations were based on correspondence with the project team during preparation of this report. If the building layout, load conditions, and/or design elevations exceed what was initially assumed and stated in this report, additional services and fee may be required to review the updated information and to perform additional analysis as necessary for the new design concepts. An Addendum to this report may be prepared and submitted to document the findings and provide updated recommendations, if needed.

6.2. PLAN AND SPECIFICATIONS REVIEW

It is essential that we perform a general review of the plans and specifications to evaluate if the recommendations contained in this report were properly interpreted and incorporated into the project documents. We will not be responsible for any misinterpretation of our recommendations if we are not retained to perform this task.

6.3. GEOTECHNICAL ENGINEER OF RECORD DURING CONSTRUCTION PHASE

To provide continuity of service into the construction phase, it is essential that we be retained as Geotechnical Engineer of Record through project closeout. The purpose of this task is to verify the geotechnical aspects of design and construction are implemented as recommended in this report during the construction phase. This is also a recommended practice promoted by the California Geotechnical Engineering Association (CalGeo).

PART 7. LIMITATIONS

We based our conclusions and recommendations based on our understanding of the proposed project development and improvements, data derived from our field explorations and laboratory testing, interpretations of available published data, and our geotechnical engineering analysis. The reported locations of the field explorations were determined by pacing from available landmarks; survey of the field explorations was not included in this scope. It is possible that actual subsurface conditions can vary between points of exploration. Similarly, load conditions may vary from what we have assumed during our analysis. If this is found to be the case, we should be notified and requested to review the changes and provide modifications to our conclusions and recommendations if needed.

We prepared this report in general accordance with the generally accepted geotechnical engineering practice as it exists in the project vicinity at the time the work was performed. No warranty, express or implied, is made. This report may be used by the Client and its design consultants, for the purpose stated for this project site for up to two years from the date of this report. If construction is delayed, or if land use, or other factors modify the site and subsurface conditions, additional field work may be needed (i.e., additional borings and/or laboratory testing) and an updated report issued. We shall be released from any liability resulting from misuse of the report by the authorized party. The Client agrees to defend, indemnify, and hold harmless Siegfried from any claim or liability associated with such unauthorized use or non-compliance with the requirements outlined herein.

PART 8. REFERENCES

American Concrete Pavement Association (ACPA), 2006, *Design of Concrete Pavement for City Streets.*

ASTM American Society for Testing of Materials, annual books of standards, current edition.

ASCE Manual 7-16, *Minimum Design Loads for Buildings and Other Structures, incl. Supplement No. 1 and Errata*; American Society of Civil Engineers, 2016.

California Geological Survey, 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California, 2008.*

California Geological Survey, 2012, *Tsunamis, Note 55*.

Coduto, D.P., Kitch W.A., and Yeung, M.R., 2016. *Foundation Design*, 3rd Edition, New Jersey: Pearson Education, Inc.

Das, B.M., 2007, *Principles of Foundation Engineering*, 6th Edition, Stamford CT: Cengage Learning.

Deep Foundations Institute (DFI), 2016, Guidelines for Soil Mixing, 1st Edition, 1st Printing.

Delattre, M.P., Graymer, R.W., Langenheim, V.E., Knudsen, K.L., et. al., 2023, *Geologic and Geophysical Maps of the Stockton 30' x 60' Quadrangle, California,* California Geological Survey, Regional Geologic Map Series, Map No. 5, Plate 1.

Duncan, J.M., and Buchignani, A. L., *Revised October 1987, An Engineering Manual for Settlement Studies*.

Duncan, J.M, Horz, R.C., and Yang, T.L., 1989, *Shear Strength Correlations for Geotechnical Engineering*, August, Blacksburg: Virginia Tech Department of Civil/Geotechnical Engineering.

Jennings, C.W., and Bryant, W.A., 2010, *2010 Fault Activity Map of California,* California Geological Survey.

NAVFACS Design Manual 7.01 and 7.02.

Reese, L.C., Isenhower, W.L., and Wang, Shin-Tower, 2006, *Analysis and Design of Shallow and Deep Foundations,* Hoboken, New Jersey, John Wiley & Sons, Inc.

Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2010, *Epicenters of and Areas Damaged by M≥5 California Earthquakes, 1800-1999,* Map Sheet 49, California Geological Survey.

Unified Facilities Criteria (UFC), 2022, *Soil Mechanics (DM 7.1),* UFC 3-220-10.

U.S. Environmental Protection Agency, *Radon Zones Map,* https://www.epa.gov/sites/default/files/2018-12/documents/radon-zones-map.pdf, accessed April 7 2023.

PLATES

F:\23PROJECTS\23288 HEALTH PLAN OF SAN JOAQUIN BE WELL\PLANS AND GRAPHICS\EXHIBITS\GEOTECH EXHIBIT\23288 BE WELL - SITE LOCATION MAP.DWG 11/17/2023 6:07 PM

SITE LOCATION MAP

HEALTH PLAN OF SAN JOAQUIN BE WELL

DATE	11/17/23
DESIGN	BLQ
DRAWN	AA
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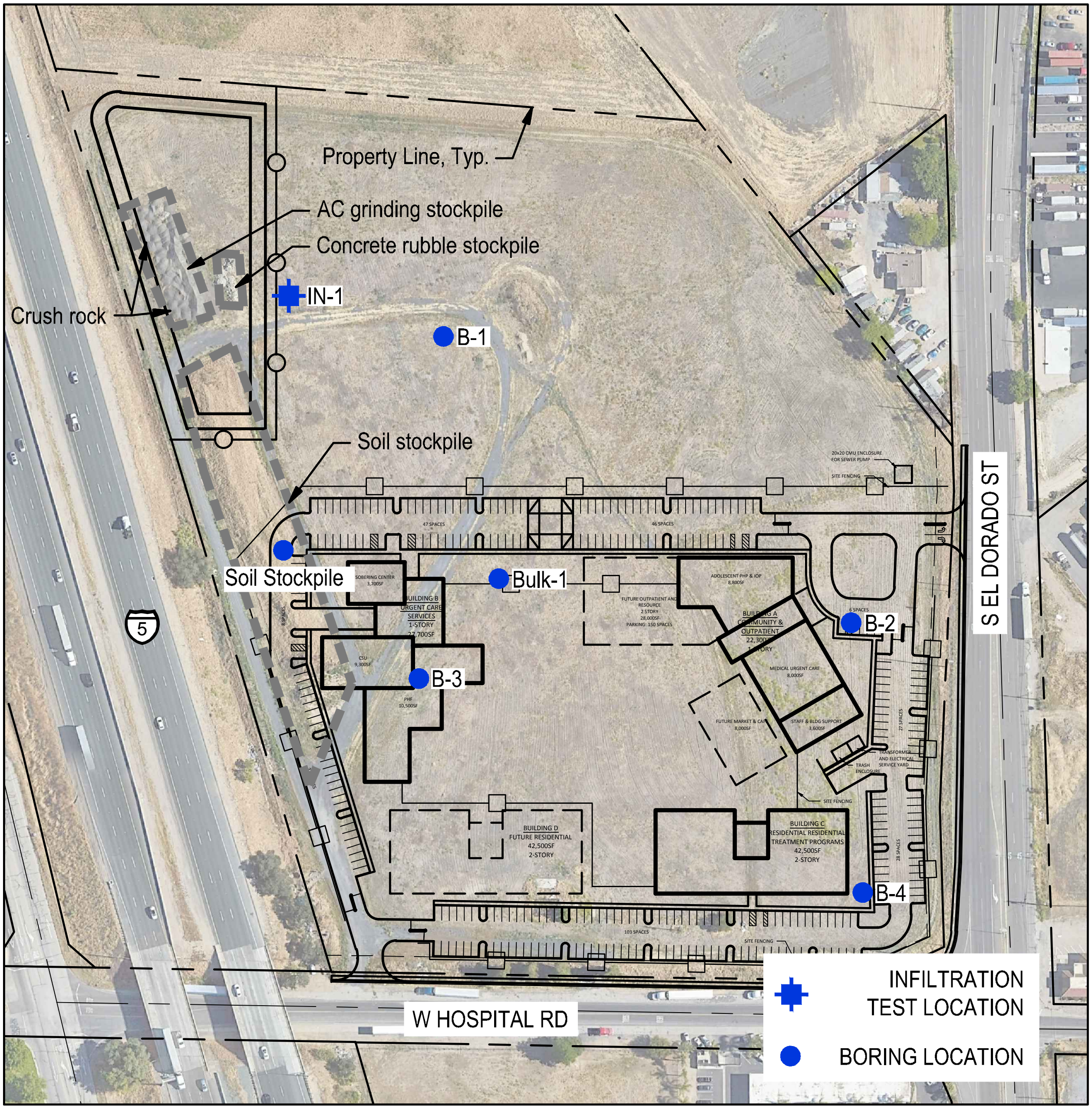
SCALE:	NO SCALE
PLATE	1

F:\23PROJECTS\23288 HEALTH PLAN OF SAN JOAQUIN BE WELL\PLANS AND GRAPHICS\EXHIBITS\GEOTECH EXHIBIT\23288 BE WELL - EXPLORATION LOCATION MAP.DWG 11/17/2023 4:27 PM



BORING NOTES

B-2 AND B-3: 15' BORING
B-1: 20' BORING
B-4: 50' BORING
IN-1: 5' BORING WITH 8" DIAMETER ROD

Aerial from Google Earth

Survey from Siegfried (2023)

EXPLORATION LOCATION MAP

HEALTH PLAN OF SAN JOAQUIN BE WELL

DATE	11/17/23
DESIGN	BLQ
DRAWN	AA
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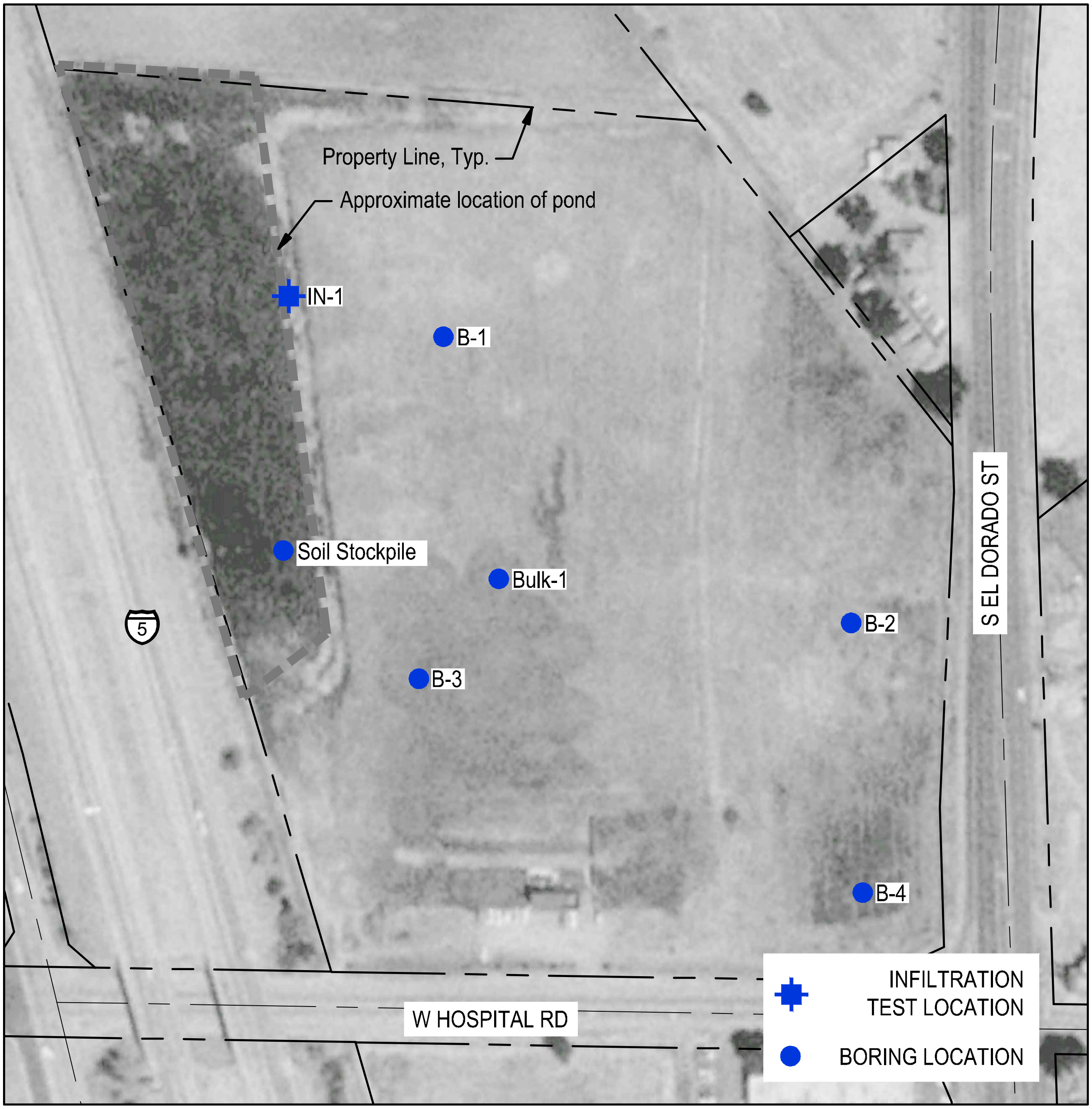
SCALE: 1" = 200'

PLATE 2



Google Earth Historic Aerial from 1993

HISTORIC IMAGE 1993

HEALTH PLAN OF SAN JOAQUIN BE WELL

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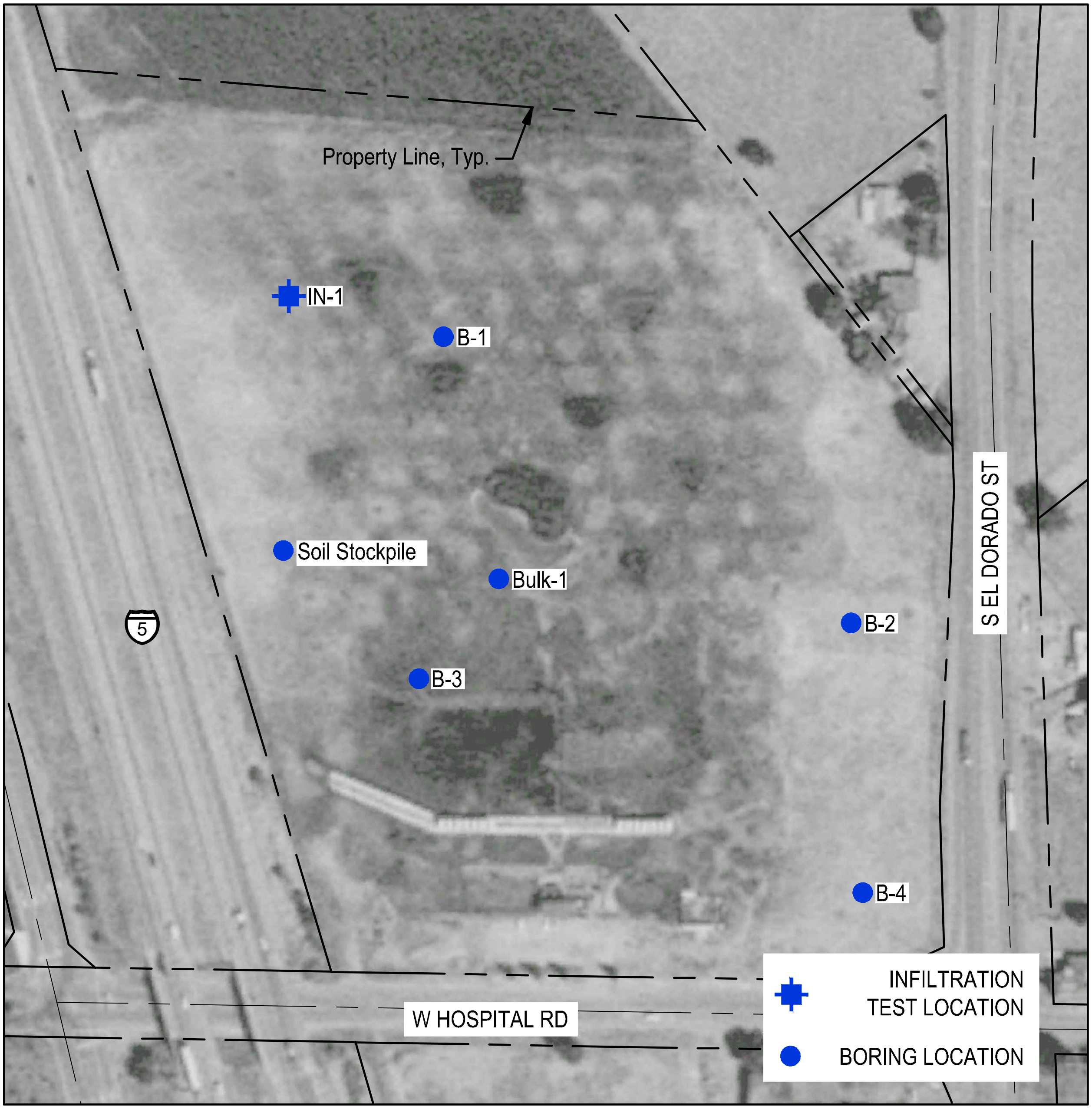
SCALE: 1" = 200'

PLATE 3

F:\23PROJECTS\23288 HEALTH PLAN OF SAN JOAQUIN BE WELL\PLANS AND GRAPHICS\EXHIBITS\GEOTECH EXHIBIT\23288 BE WELL - HISTORIC IMAGE 1998.DWG 11/16/2023 4:46 PM



Google Earth Historic Aerial from 1998

HISTORIC IMAGE 1998

HEALTH PLAN OF SAN JOAQUIN BE WELL

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SCALE: 1" = 200'

PLATE 4

Google Earth Historic Aerial from 2007

HISTORIC IMAGE 2007

HEALTH PLAN OF SAN JOAQUIN BE WELL

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DRAWN	AA
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SCALE:	1" = 200'
PLATE	5

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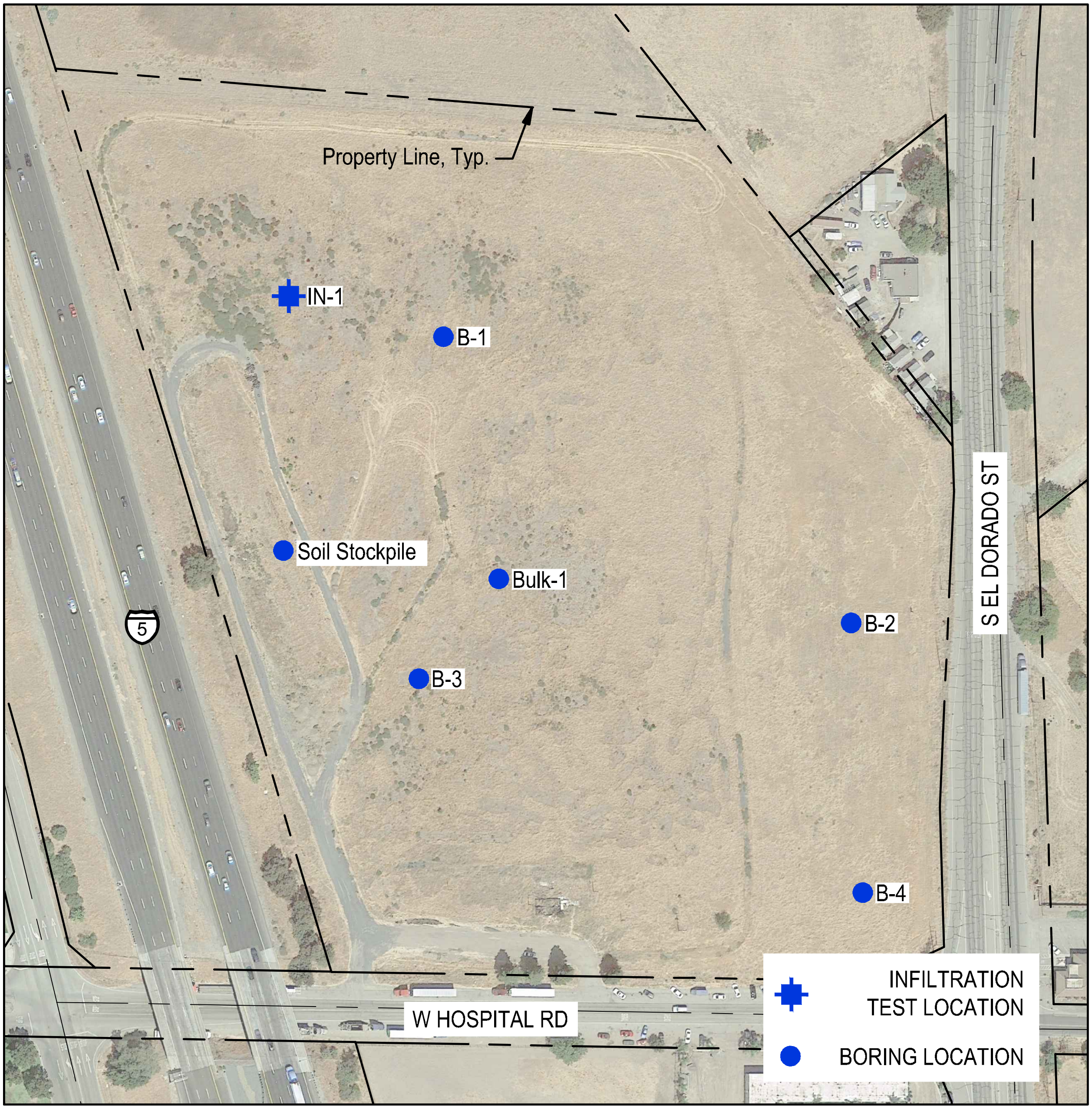
F:\23PROJECTS\23288 HEALTH PLAN OF SAN JOAQUIN BE WELL\PLANS AND GRAPHICS\EXHIBITS\GEOTECH EXHIBIT\23288 BE WELL - HISTORIC IMAGE 2018.DWG 11/16/2023 4:46 PM



Google Earth Historic Aerial from 2018

HISTORIC IMAGE 2018

HEALTH PLAN OF SAN JOAQUIN BE WELL

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DESIGN	BLQ
DRAWN	AA
JOB NO.	23288

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SCALE:	1" = 200'
PLATE	6

APPENDIX A
FIELD EXPLORATION

Prior to initiating our field exploration, the planned exploration locations were checked for underground utilities by contacting Underground Service Alert (USA) which located underground and aboveground utilities within the vicinity of our proposed explorations. Based on the planned depths of the explorations and review of the available data regarding depth to groundwater, it was determined drilling permits with the San Joaquin County Environmental Health Department would be required for both the borings and CPTs.

The soil borings were advanced on November 17, 2023. The locations of the soil borings are shown on Plate 2.

Drilled Borings

Four (4) soil borings identified as Borings B-1, B-2, B-3, and B-4 were drilled to depths of between approximately 16½ and 51½ feet around the site. A shallow boring, IN-1, was advanced near the proposed infiltration area to a depth of about 5 feet bgs. The borings were advanced using a truck-mounted CME 75 drill rig equipped with 8-inch outer diameter hollow-stem augers.

Samples were collected from the borings using split barrel soil samplers having nominal outer dimensions of 3.0 inches or standard penetration test sampler (i.e., SPT) without liners which were advanced automatically with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the samplers for the 18-inch sample interval was recorded on the Boring logs. The sum of the blow counts for the final 12 inches of driving is recorded as the “N Value”. The N Values reported are raw values obtained in the field and are not corrected for overburden, rod length, bore diameter, and hammer energy effects. Relatively undisturbed and bulk samples were collected at select depths from the bores and transported to our laboratory for further analysis and geotechnical testing. The boring logs are presented in this Appendix.

BOREHOLE NUMBER B-1

SIEGFRIED

CLIENT Boulder Associates, Inc.
PROJECT NAME Health Plan of San Joaquin Be Well
PROJECT NUMBER 23288-5001
PROJECT LOCATION El Dorado Street and Hospital Road, French Camp, CA
DATE STARTED 11-07-2023 **COMPLETED** 11-07-2023
POSITION
DRILLING CONTRACTOR Baja Exploration
GROUND ELEVATION **FINAL DEPTH** 21.50 ft
DRILLING METHOD Hollow Stem Auger
GROUNDWATER LEVELS:
EQUIPMENT CME 75
AT TIME OF DRILLING Not Encountered
HOLE SIZE 8.0 in.
AT END OF DRILLING
LOGGED BY Alejandro Aguilera, EIT **CHECKED BY** Charley Scott, PE
AFTER DRILLING

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	Sandy Lean CLAY (CL): brown; dry; low plasticity; fine; dead grass and tumbleweeds on surface, undocumented fill.								
2.50	Sandy SILT (ML): hard; brown; dry; nonplastic; fine.	1B 1C	6-19-28 (47)	102	9				
		2B 2C	17-22-22 (44)	104	4				
7.50	: gray with rust mottling; moist.	3B 3C	19-16-22 (38)	102	22	NP	NP	NP	8
8.75	Poorly-graded SAND with silt (SP-SM): light brown; moist; nonplastic; fine to medium.								
10.00	Poorly-graded SAND (SP): medium dense; light brown; dry to moist; nonplastic; fine to medium.	4B 4C	9-15-17 (32)	100	2	NP	NP	NP	3
15.00	: light gray.	5B 5C	7-9-10 (19)	96	29				
15.75	Lean CLAY (CL): very stiff; brown; moist; medium plasticity; fine.								
20.00	: hard.	6B 6C	8-14-17 (31)	104	22				
21.50	Terminated at 21.50 ft.								

NOTES

BOREHOLE NUMBER B-2

SIEGFRIED

CLIENT Boulder Associates, Inc.
PROJECT NAME Health Plan of San Joaquin Be Well
PROJECT NUMBER 23288-5001
PROJECT LOCATION El Dorado Street and Hospital Road, French Camp, CA
DATE STARTED 11-07-2023 **COMPLETED** 11-07-2023
POSITION
DRILLING CONTRACTOR Baja Exploration
GROUND ELEVATION
FINAL DEPTH 16.50 ft
DRILLING METHOD Hollow Stem Auger
GROUNDWATER LEVELS:
EQUIPMENT CME 75
▽ **AT TIME OF DRILLING** Not Encountered
HOLE SIZE 8.0 in.
▼ **AT END OF DRILLING**
LOGGED BY Alejandro Aguilera, EIT **CHECKED BY** Charley Scott, PE
▽ **AFTER DRILLING**

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)
0–2.25	Sandy Lean CLAY (CL): gray; dry; low plasticity; fine; dead grass and tumbleweeds on surface, undocumented fill.	1B 1C	19-15-19 (34)	113	9
2.25–4.25	Silty SAND (SM): dense; dark gray; dry to moist; nonplastic; fine.				
4.25–6.00	: very dense; gray with rust mottling; moist; strong reaction to HCl.	2B 2C	38-33-19 (52)	99	20
6.00–8.50	SILT with sand (ML): hard; gray with rust mottling; moist; nonplastic; fine.	3B 3C	12-13-24 (37)	98	20
8.50–15.00	Sandy Lean CLAY (CL): very stiff; gray with rust mottling; moist; low plasticity; fine.	4B 4C	8-14-16 (30)	97	28
15.00–16.50	Lean CLAY with sand (CL): hard; brown with white mottling; moist; medium plasticity; medium reaction to HCl.	5B 5C	8-15-18 (33)	97	28

Terminated at 16.50 ft.

NOTES

SIEGFRIED

BOREHOLE NUMBER B-3

CLIENT Boulder Associates, Inc.
PROJECT NAME Health Plan of San Joaquin Be Well
PROJECT NUMBER 23288-5001
PROJECT LOCATION El Dorado Street and Hospital Road, French Camp, CA
DATE STARTED 11-07-2023 **COMPLETED** 11-07-2023
POSITION
DRILLING CONTRACTOR Baja Exploration
GROUND ELEVATION **FINAL DEPTH** 16.50 ft
DRILLING METHOD Hollow Stem Auger
GROUNDWATER LEVELS:
EQUIPMENT CME 75
AT TIME OF DRILLING Not Encountered
HOLE SIZE 8.0 in.
AT END OF DRILLING
LOGGED BY Alejandro Aguilera, EIT **CHECKED BY** Charley Scott, PE
AFTER DRILLING

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)
0.00–2.00	Sandy SILT (ML): gray; dry; nonplastic; fine; dead grass and tumbleweeds on surface, undocumented fill.	1B 1C	12-12-12 (24)		4
2.00–3.50	Silty SAND (SM): medium dense; gray; dry; nonplastic; fine.				
3.50–6.00	Sandy SILT (ML): hard; dry to moist.	2B 2C	13-23-18 (41)	108	4
6.00–8.50	SILT (ML): hard; gray with rust mottling; dry to moist; nonplastic; fine.	3B 3C	7-15-17 (32)	92	14
8.50–15.00	: moist.	4B 4C	13-20-22 (42)	104	21
15.00–16.50	Sandy Lean CLAY (CL): hard; brown; moist; medium plasticity; fine.	5B 5C	18-27-33 (60)	100	23

Terminated at 16.50 ft.

NOTES

BOREHOLE NUMBER B-4

SIEGFRIED

CLIENT Boulder Associates, Inc.
PROJECT NUMBER 23288-5001
DATE STARTED 11-07-2023 **COMPLETED** 11-07-2023
DRILLING CONTRACTOR Baja Exploration
DRILLING METHOD Hollow Stem Auger
EQUIPMENT CME 75
HOLE SIZE 8.0 in.
LOGGED BY Alejandro Aguilera, EIT **CHECKED BY** Charley Scott, PE

PROJECT NAME Health Plan of San Joaquin Be Well
PROJECT LOCATION El Dorado Street and Hospital Road, French Camp, CA
POSITION
GROUND ELEVATION **FINAL DEPTH** 51.50 ft
GROUNDWATER LEVELS:
▽ **AT TIME OF DRILLING** 24.00 ft
▼ **AT END OF DRILLING**
▼ **AFTER DRILLING**

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		Silty SAND with gravel (SM): gray; dry; nonplastic; dead grass and tumbleweeds on surface, undocumented fill.								
	2.50	SILT (ML): hard; gray with rust mottling; moist; nonplastic; fine.	1B 1C	26-23-27 (50)	109	8	NP	NP	NP	97
5	5.00	: very stiff.	2B 2C	10-13-13 (26)	98	9				
	7.50	: hard.	3B 3C	7-18-21 (39)	100	19				
10			4B 4C	8-10-15 (25)	98	25				
	11.00	Lean CLAY (CL): very stiff; gray with rust mottling; moist; low to medium plasticity; fine.								
15	15.00	Silty SAND (SM): very stiff; reddish brown; moist; nonplastic; fine.	5B 5C	5-15-15 (30)		17	NP	NP	NP	14
	16.25	Poorly-graded SAND with silt (SP-SM): medium dense; reddish brown; moist; nonplastic; fine to medium.								
20	20.00	Sandy Lean CLAY (CL): very stiff; brown with red and black mottling; moist; medium plasticity; fine.	SPT 6	9-9-8 (17)		19				
	▽									
25	25.00	: brown with rust mottling; low plasticity.	7B 7C	9-17-20 (37)	99	29				
	26.33	Silty SAND (SM): dense; brown with rust mottling; moist; nonplastic; fine.								

NOTES

CLIENT Boulder Associates, Inc.

PROJECT NAME Health Plan of San Joaquin Be Well

PROJECT NUMBER 23288-5001

PROJECT LOCATION El Dorado Street and Hospital Road, French Camp, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
		Silty SAND (SM): dense; brown with rust mottling; moist; nonplastic; fine.								
30	30.00	Clayey SAND (SC): very stiff; brown; moist; low plasticity; fine to medium.	SPT 8	16-10-13 (23)		26				
35	35.00	Lean CLAY with sand (CL): very stiff; brown; moist; medium plasticity; fine.								
	35.75	: dark gray.	9B 9C	4-8-11 (19)		29				
40	40.00	: stiff; light gray; medium to high plasticity.	SPT 10	3-6-5 (11)		33				
45	45.75	Silty SAND (SM): medium dense; brown; moist; nonplastic; fine.	11B 11C	3-6-9 (15)	104	26				
50	50.00	Sandy Lean CLAY (CL): very stiff; light gray; moist to wet; medium plasticity; fine.	SPT 12	6-7-9 (16)		35				
	51.50	Terminated at 51.50 ft.								
55										

NOTES

BOREHOLE NUMBER IN-1

SIEGFRIED

CLIENT	Boulder Associates, Inc.	**PROJECT NAME**	Health Plan of San Joaquin Be Well
PROJECT NUMBER	23288-5001	**PROJECT LOCATION**	El Dorado Street and Hospital Road, French Camp, CA
DATE STARTED	11-07-2023 **COMPLETED** 11-07-2023	**POSITION**	
DRILLING CONTRACTOR	Baja Exploration	**GROUND ELEVATION**	**FINAL DEPTH** 5.00 ft
DRILLING METHOD	Hollow Stem Auger	**GROUNDWATER LEVELS:**	
EQUIPMENT	CME 75	∇ **AT TIME OF DRILLING**	Not Encountered
HOLE SIZE	8.0 in.	▼ **AT END OF DRILLING**	
LOGGED BY	Alejandro Aguilera, EIT **CHECKED BY** Charley Scott, PE	∇ **AFTER DRILLING**	

DEPTH (ft)	GRAPHIC LOG		MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)
		0.50	Silty SAND (SM): brown; dry; nonplastic; fine; dead grass and tumbleweeds on surface, native.				
			: medium dense.	SPT 1	2-3-9 (12)		
		2.00	: dense.	2B 2C	15-18-21 (39)	107	4
				3B 3C	17-21-24 (45)	107	4
5		5.00	Terminated at 5.00 ft.				
10							
15							
20							
25							

NOTES

APPENDIX B
LABORATORY TESTING

Laboratory testing was performed to quantify and evaluate the geotechnical characteristics of the soil samples obtained at the site. The following laboratory tests were performed on selected samples from the borings:

- Moisture Content (ASTM D 2216)
- Dry Density (ASTM D 2937)
- Atterberg Limits (ASTM D 4318)
- Particle Size Distribution (ASTM D 6913)
- R-Value (ASTM D 2844/CT301)
- Expansion Index (ASTM D 4829)
- pH and Electrical Resistivity (CT643)
- Sulfate and Chloride Content (CT417 and CT422)
- Redox Potential (ASTM G 200m)
- Sulfides (AWWA C105/A25.5)

Tests were performed by Siegfried, Blackburn Consulting, and Sunland Analytical.

The results of the tests performed above are discussed in the Subsurface Conditions section of the report (Section 3.1). They are also presented on the boring logs provided in Appendix A, and as summaries and reports provided in Appendix B.

SIEGFRIED

CIVIL STRUCTURAL GEOTECHNICAL SURVEY LANDSCAPE PARTNERING

Geotechnical Materials Testing Summary

Tested in General Accordance with ASTM D1140, D2487, D4318, D6913, and D7263

Project Name: Health Plan of San Joaquin Be Well

Project Number: 23288-5001

Project Location: French Camp, CA

					ASTM D2216	ASTM D7263		ASTM D4318		ASTM D1140/D6913			ASTM D2487	
Sample Date	Location ID	Depth Top (ft)	Depth Base (ft)	Color	Moisture (%)	Wet Density (pcf)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	USCS Group Symbol	USCS Description
11/7/2023	B4-1C	3.5	4.0	Gray with rust mottling	8.4	118.2	109.1	NP	NP	0	3	97	ML	Silt
11/7/2023	B4-2C	6.0	6.5	Gray with rust mottling	9.4	106.7	97.5	---	---	---	---	---	---	---
11/7/2023	B4-3C	8.5	9.0	Gray with rust mottling	19.4	120.0	100.5	---	---	---	---	---	---	---
11/7/2023	B4-4B	10.5	11.0	Gray with rust mottling	25.3	123.5	98.5	---	---	---	---	---	---	---
11/7/2023	B4-5C	16.3	16.5	Reddish Brown	17.3	---	---	NP	NP	0	86	14	SM	Silty Sand
11/7/2023	B4-SPT6	20.0	21.5	Brown with black and red mottling	19.0	---	---	---	---	---	---	---	---	---
11/7/2023	B4-7C	26.0	26.5	Brown	29.2	128.0	99.1	---	---	---	---	---	---	---
11/7/2023	B4-SPT8	30.0	31.5	Brown	26.3	---	---	---	---	---	---	---	---	---
11/7/2023	B4-9C	36.0	36.5	Dark gray	29.2	---	---	---	---	---	---	---	---	---
11/7/2023	B4-SPT10	40.0	41.5	Light gray	33.2	---	---	---	---	---	---	---	---	---
11/7/2023	B4-11C	46.0	46.5	Brown	25.5	130.1	103.7	---	---	---	---	---	---	---
11/7/2023	B4-SPT12	50.0	51.5	Light gray	35.2	---	---	---	---	---	---	---	---	---
11/7/2023	Bulk 1	EL 16.0	EL 16.5	Dark Brown	---	---	---	28	11	1	36	63	CL	Sandy Lean Clay
11/7/2023	Bulk 2	EL 21.0	EL 21.5	Light Brown	---	---	---	27	9	1	39	60	CL	Sandy Lean Clay

STOCKTON
3428 Brookside Rd.
Stockton, CA 95219
t: 209.943.2021

SAN JOSE
111 N. Market St., #300
San Jose, CA 95113
t: 408.754.2021

SACRAMENTO
1164 National Drive #20
Sacramento, CA 95834
t: 916.520.2777

MODESTO
101 Sycamore Ave, #100
Modesto, CA 95354
t: 209.762.3580

SIEGFRIED

Geotechnical Materials Testing Summary

Tested in General Accordance with ASTM D1140, D2487, D4318, D6913, and D7263

Project Name: Health Plan of San Joaquin Be Well

Project Number: 23288-5001

Project Location: French Camp, CA

					ASTM D2216	ASTM D7263		ASTM D4318		ASTM D1140/D6913			ASTM D2487	
Sample Date	Location ID	Depth Top (ft)	Depth Base (ft)	Color	Moisture (%)	Wet Density (pcf)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	USCS Group Symbol	USCS Description
11/7/2023	B1-1C	3.5	4.0	Brown	9.1	110.9	101.6	---	---	---	---	---	---	---
11/7/2023	B1-2C	6.0	6.5	Brown	4.2	108.8	104.4	---	---	---	---	---	---	---
11/7/2023	B1-3C (TOP)	8.5	8.8	Gray with rust mottling	22.1	124.8	102.2	---	---	---	---	---	---	---
11/7/2023	B1-3C (BOTTOM)	8.8	9.0	Light brown	3.5	---	---	NP	NP	0	92	7.7	SP-SM	Poorly-Graded Sand with Silt
11/7/2023	B1-4C	11.0	11.5	Light brown	2.1	102.5	100.5	NP	NP	0	97	2.8	SP	Poorly-Graded Sand
11/7/2023	B1-5C	16.0	16.5	Brown	28.8	123.5	95.9	---	---	---	---	---	---	---
11/7/2023	B1-6C	21.0	21.5	Brown	22.4	127.9	104.5	---	---	---	---	---	---	---
11/7/2023	B2-1C	2.0	2.5	Dark gray	8.8	123.0	113.0	---	---	---	---	---	---	---
11/7/2023	B2-2C	4.5	5.0	Gray with rust mottling	19.8	118.6	98.9	---	---	---	---	---	---	---
11/7/2023	B2-3C	7.0	7.5	Gray with rust mottling	20.0	117.7	98.1	---	---	---	---	---	---	---
11/7/2023	B2-4C	9.5	10.0	Gray with rust mottling	28.4	124.6	97.1	---	---	---	---	---	---	---
11/7/2023	B2-5C	16.0	16.5	Brown with white mottling	28.0	124.2	97.0	---	---	---	---	---	---	---
11/7/2023	B3-1C	2.0	2.5	Gray	4.1	---	---	---	---	---	---	---	---	---
11/7/2023	B3-2C	4.5	5.0	Gray	4.5	112.8	107.9	---	---	---	---	---	---	---
11/7/2023	B3-3C	7.0	7.5	Gray with rust mottling	14.0	104.6	91.7	---	---	---	---	---	---	---
11/7/2023	B3-4C	9.5	10.0	Gray with rust mottling	20.7	125.9	104.3	---	---	---	---	---	---	---
11/7/2023	B3-5C	16.0	16.5	Brown	23.4	123.4	100.0	---	---	---	---	---	---	---
11/7/2023	IN1-2C	3.0	3.5	Brown	3.7	111.1	107.1	---	---	---	---	---	---	---
11/7/2023	IN1-3C	4.5	5.0	Brown	4.0	111.1	106.8	---	---	---	---	---	---	---

STOCKTON
3428 Brookside Rd.
Stockton, CA 95219
t: 209.943.2021

SAN JOSE
111 N. Market St., #300
San Jose, CA 95113
t: 408.754.2021

SACRAMENTO
1164 National Drive #20
Sacramento, CA 95834
t: 916.520.2777

MODESTO
101 Sycamore Ave, #100
Modesto, CA 95354
t: 209.762.3580

Unconfined Compression
ASTM D 2166

Project Name: Siegfried - 23288-5001
Project Number: 4437.X012
Sample ID: B4-4C
Type of Sample: CalMod
Sample Description: SILT, grayish brown
Depth: 11-11.5'

Sample Data

Sample Length:	5.01	in
Diameter:	2.39	in
Height-to-Diameter Ratio:	2.10	
Sample Area:	4.48	in^2
Sample Volume:	22.5	in^3
Specific Gravity:	2.65	(assumed)

Sample + Tube:	772	g
Tube:	0.00	g
Sample Weight:	772	g
Wet Density:	130.9	pcf
Moisture:	18	%
Dry Density:	110.5	pcf
Saturation:	98.6	%

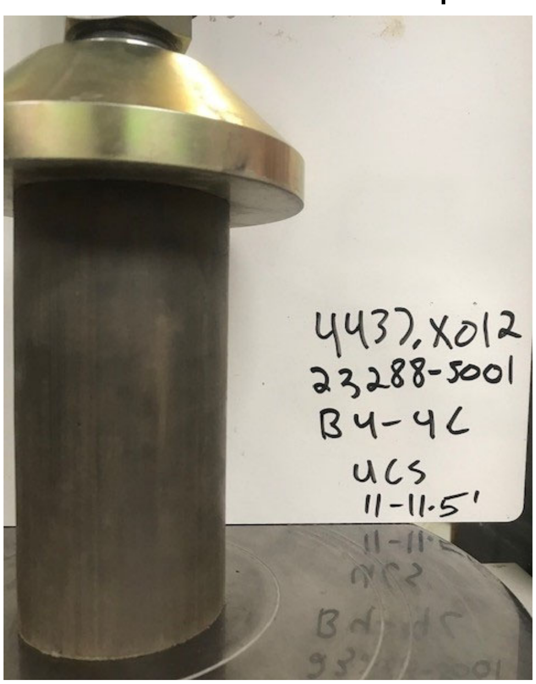
**Moisture content taken after test*

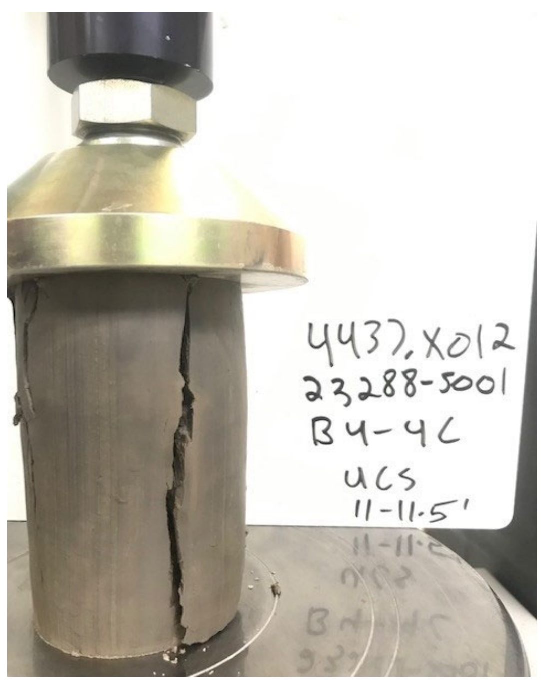
Test Results

Rate of Strain:	0.0300	in/min
Deflection at Max. Load:	0.473	in
Maximum Load:	264	lbs
Strain at Failure:	9.44	%
Average cross-sectional area at failure:	4.95	in^2
Compressive Strength:	**3.84**	**tsf**
	53.4	**psi**

Strain Information

Rate of Strain ½%:	0.025	in/min
Rate of Strain 2%:	0.100	in/min
Strain Rate:	0.030	in/min
15% Strain:	0.752	in

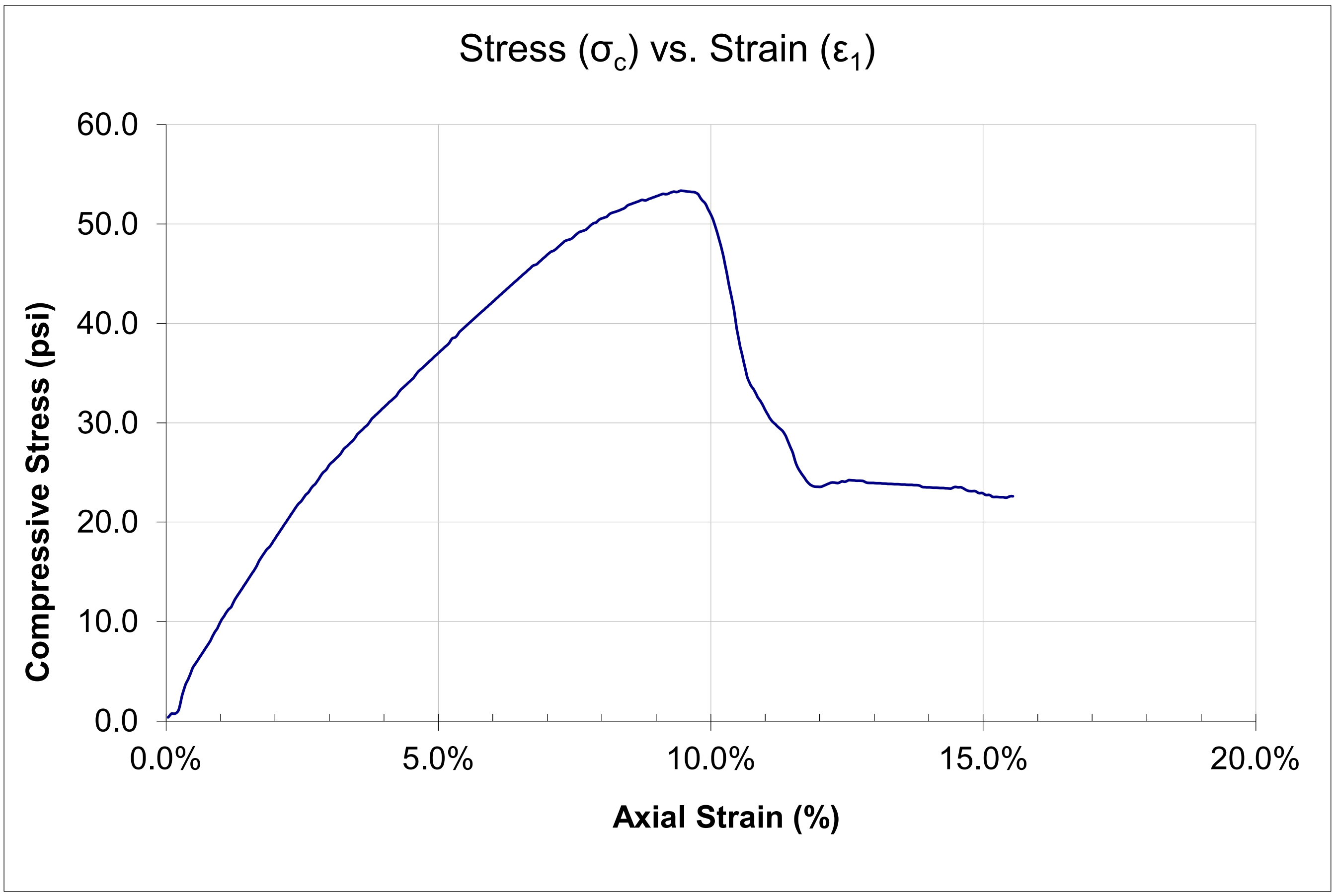




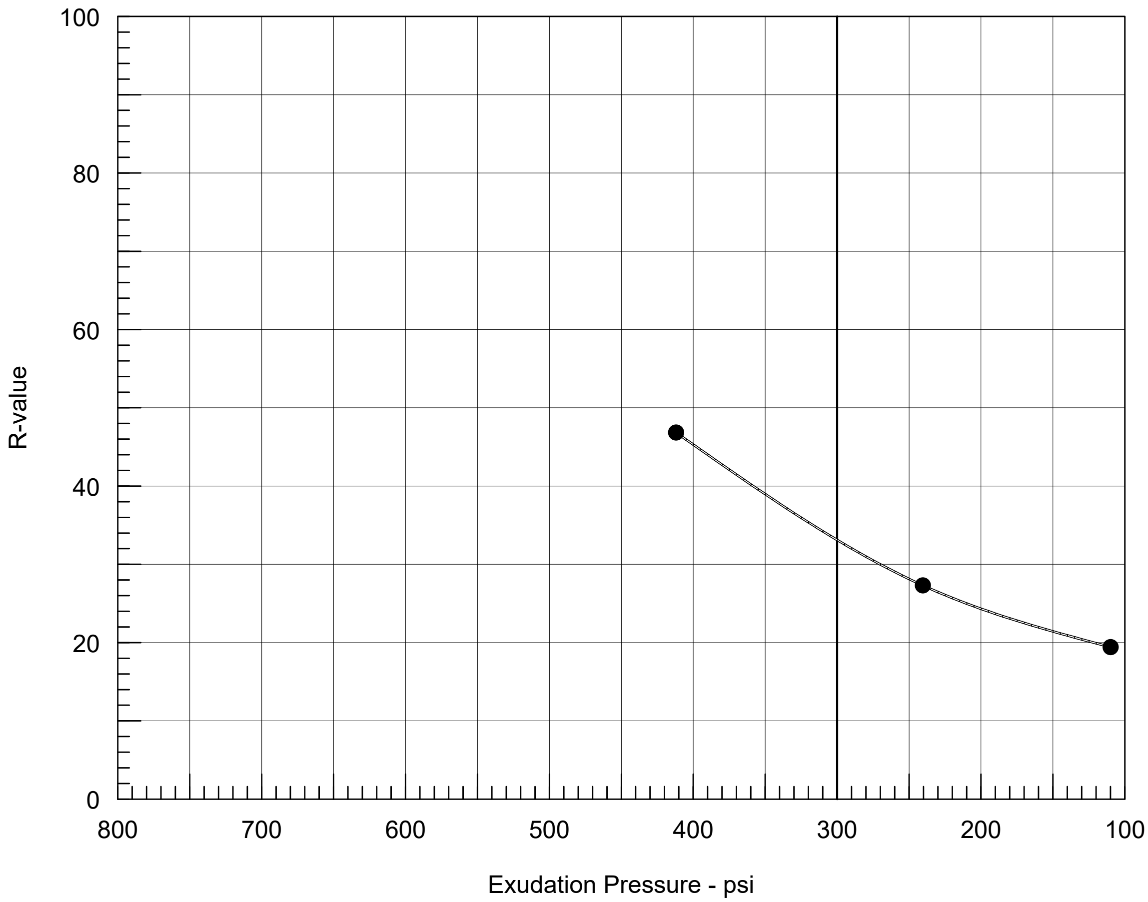
Unconfined Compression
ASTM D 2166

Project Name:	Siegfried - 23288-5001
Project Number:	4437.X012
Sample ID:	B4-4C
Type of Sample:	CalMod
Sample Description:	SILT, grayish brown
Depth:	11-11.5'

Compressive Strength:	**3.84**	**tsf**
	53.4	**psi**



R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	125	113.1	14.3	109	54	2.55	412	47	47
2	83	110.5	15.4	65	87	2.60	240	26	27
3	63	109.5	16.5	70	100	2.55	110	19	19

Test Results	Material Description
R-value at 300 psi exudation pressure = 33	SANDY SILT, dark brown

Project No.: 4437.X012

Project: Health Plan of San Joaquin Be Well 23288

Source of Sample: Center of Site **Depth:** Surface

Sample Number: Bulk 1

Date: 11/13/2023

Tested by: DSB

Checked by: RBL

Remarks:

R-VALUE TEST REPORT

Blackburn Consulting

Figure ______

Sunland Analytical

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

Date Reported 11/15/2023
Date Submitted 11/08/2023

To: Charley Scott
Siegfried-Stockton
3428 Brookside Rd.
Stockton, CA 95219

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

The reported analysis was requested for the following location:
Location : 23288-5001 CNTR SITE Site ID : BULK 1.
Thank you for your business.

* For future reference to this analysis please use SUN # 90921-188516.

EVALUATION FOR SOIL CORROSION

Soil pH	7.01		
Moisture	5.0 %		
Minimum Resistivity	1.82 ohm-cm (x1000)		
Chloride	5.0 ppm	00.00050 %	
Sulfate	6.1 ppm	00.00061 %	
Redox Potential	(+) 268 mv		
Sulfides	Presence - NEGATIVE		

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m
Redox Potential ASTM G-200m, Sulfides AWWA C105/A25.5

Sunland Analytical

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

Date Reported 11/15/2023
Date Submitted 11/08/2023

To: Charley Scott
Siegfried-Stockton
3428 Brookside Rd.
Stockton, CA 95219

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

The reported analysis was requested for the following location:
Location : 23288-5001 STOCKPILE Site ID : BULK 2.
Thank you for your business.

* For future reference to this analysis please use SUN # 90921-188517.

EVALUATION FOR SOIL CORROSION

Soil pH	7.63		
Moisture	7.3 %		
Minimum Resistivity	1.28 ohm-cm (x1000)		
Chloride	3.5 ppm	00.00035 %	
Sulfate	0.2 ppm	00.00002 %	
Redox Potential	(+) 255 mv		
Sulfides	Presence - NEGATIVE		

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m
Redox Potential ASTM G-200m, Sulfides AWWA C105/A25.5

End of Report