GEOTECHNICAL REPORT
PROPOSED NEW VALLEY OF THE SACRED HEART EDUCATION CENTER
SOUTHWEST CORNER OF NORTH 2ND STREET AND EAST A STREET
DIXON, CA

Prepared for Valley of the Sacred Heart

February 21, 2024

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SIEGFRIED

February 21, 2024

Chris Simpson
Valley of the Sacred Heart Academy
210 Peters Avenue
Dixon, CA 95620

RE: GEOTECHNICAL REPORT
PROPOSED NEW VALLEY OF THE SACRED HEART EDUCATION CENTER
SOUTHWEST CORNER OF NORTH 2ND STREET AND EAST A STREET
DIXON, CA

Dear Chris,

We have completed our geotechnical report for the proposed new Valley of the Sacred Heart Education Center located at southwest corner of North 2nd Street and East A Street in Dixon, California. The purpose of our study was to explore the subsurface soil and groundwater conditions at the site to provide geotechnical engineering recommendations related to foundation design and earthwork construction.

Based on our study, the site conditions are suitable for design and construction of the subject project from a geotechnical engineering perspective. We encountered expansive soils and undocumented fill near the surface of the site. The presence of these soils and conditions could cause undesirable distress to foundations and slabs if not mitigated. We make specific design and construction recommendations to address the adverse effects of these conditions in the following report.

We appreciate the opportunity to collaborate with you 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

Date Stamped 02/21/24

Bradford Quon, GE
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PART 1. INTRODUCTION

We have completed our geotechnical report for the proposed new Valley of the Sacred Heart Education Center located at southwest corner of North 2nd Street and East A Street in Dixon, California. The purpose of our study was to explore the subsurface soil and groundwater conditions at the site to provide 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 existing 25,600 sf site located at 219 E. A Street, Dixon, California is proposed for use as a new education center building. The project will entail an approximately 12,000 sf, two story structure. The building type is to be a wood framed structure with steel columns. Structural load conditions are not anticipated to exceed 50 kips and 2 kips per lineal foot for column and wall loads, respectively. Given the building type and projected height of the building the fundamental period of the building is anticipated to be less than 0.5 seconds. No basements or below grade structures are planned. Other improvements will consist of exterior concrete flatwork, asphalt concrete pavements, underground utilities, trash enclosures, and landscaping. A new multipurpose room with basketball half court will occupy the northeast portion of the building.

1.2. SCOPE OF SERVICES

Our authorized scope of services was outlined in our proposal dated July 26, 2023, and authorized by you. The geotechnical engineering scope of services generally included the following:

- Field exploration consists of a series of drilled borings to maximum of 30 to 100 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 recommendations related to foundation design and earthwork construction.

1.3. SITE CONDITIONS

The site to be developed is located at the southwest corner of North 2nd Street and East A Street in Dixon, California. The relatively level and approximate ½ acre site is bounded by residential homes to the north and east, East A Street to the south, and North 2nd Street to the west. Concrete sidewalk and landscaping is located on the frontage along North 2nd Street and East A Street. Large mature trees are located along North 2nd Street. A distressed concrete paved access way enters the site on the northwest corner of the site.

We reviewed historic aerial images provided at https://historicaerials.com from 1957, 1968, 1984, 1993, 2005, 2009, 2010, 2012, 2014, 2016, 2018, and 2020.

- The 1957 image shows the site occupied by trees and a structure on the southeast corner of the site.
- The 1968 image did not show any noticeable change to the site.
- The 1984 image is not clear about the presence of any noted structures other than that previously noted; however, larger trees remain onsite.
- The 1993 image was presented in black and white at a resolution that was not suitable for detecting any noticeable change to the site.
- The 2005 image remains unchanged.
- The 2009 image shows the structure on the southeast corner of the site as being removed and many of the trees from the center of the site removed.

- The 2010, 2012, 2024, 2016, and 2018 images show the site relatively unchanged from the 2009 image.
- The 2020 image shows the site primarily cleared and the trees from the center of the site removed.

PART 2. ENGINEERING GEOLOGY AND SEISMIC HAZARDS (GEOHAZARDS)

2.1. SITE CHARACTERIZATION

2.1.1. Local Geologic Conditions

Sims, Fox, Bartow, and Helley (1973) mapped the near surface deposits as Quaternary Alluvium. The alluvium is described as mainly unconsolidated flood-plain deposits; sand, silt, gravel, and clay irregularly interstratified.

The Soil Survey of Solano County, California maps the site as the Yolo Silty Clay Loam. These soils are characterized to be “well drained”, with a hydrologic soil group classification “B”. The soil survey describes the silty clay loam with a moderately high saturated hydraulic conductivity of 0.20 to 0.60 inches per hour.

2.1.2. 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 fill soils on the site were tested to have a PI of 16 and 25 and an EI of 58 indicating a medium potential for expansion, thus we consider the expansive soils **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. The near surface soils are generally very stiff to hard cohesive soils. 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 (center of site)	6.64	1.15	4.4 (0.00044%)	45.6 (0.00456%)	+ 347	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 06095C0200F effective 8/2/2012 indicates the entire parcel to be developed is mapped as an "Area of Minimal Flood Hazard", 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 Solano 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 a low potential for hydrocollapse and not to be a design consideration.**

2.3. SEISMIC HAZARDS

2.3.1. Historical Seismicity

The site is located within a moderate seismic region with many of the active faults located greater than about 20 miles west of the project site within the San Francisco Bay Area. The California Earthquake Authority also notes there is a 76% likelihood of one or more M7.0+ earthquakes affecting Northern California in a 30 year-period, beginning in 2014. The CEA notes there is a 98% chance of one or more M6.0+ earthquakes within the San Franciso Bay area in the same period.

Toppozada, et. al., (2000) mapped the epicenters of and areas damaged by Magnitude (M) ≥ 5 Earthquakes.

- The mapping showed one significant earthquake located at the west of the site with a M6.6 in 1892. This event showed historic reports of “numerous chimneys and fire walls thrown down.”
- One quake northwest of the site within the county with a M6.4 in 1892. This event showed historic reports where “walls cracked; roofs fell in; and walls thrown out of plumb.”

The Unified States Geological Survey (USGS) Earthquake Hazard Toolbox maintains an interactive online portal at https://earthquake.usgs.gov/nshmp to disaggregate the nearest earthquake faults that contribute the most towards the earthquake hazard. The interactive tool models the National Seismic Hazard Map for the Conterminous US (2018). For this site, the disaggregate earthquake has a mean magnitude M6.57 occurring at a radius of 21.4 km (miles) from the site. Table 2.2 provides the faults modeled that pose a significant contribution to the seismic hazard, distance from the site, and the magnitude it can generate. The targe epsilon which represents the departure of the target ground motion, at a specific hazard level from that predicted by the ground-motion prediction equations is also presented.

Table 2.2: Faults, distance from site, and magnitude				
Fault Name (UCERF[1] Fault Model 3.1)	Distance, km (miles)	Direction from Site	Magnitude (M)	Target epsilon (ε_o)
Great Valley 6 (Midland)	3.47 ()	West	6.73	0.12
Great Valley 4b (Gordon Valley)	20.65 ()	West	6.83	1.20
Hunting Creek – Berryessa	35.39 ()	Northwest	7.18	1.87
Great Valley 4a (Trout Creek)	21.23 ()	West	6.96	1.10
Great Valley 4b (Gordon Valley)	20.79 ()	West	7.03	1.02
Great Valley 5 (Pittsburg-Kirby Hills)	26.51 ()	West	6.38	2.15
Fault Name (UCERF Fault Model 3.2)	Distance, km (miles)	Direction from Site	Magnitude (M)	Target epsilon (ε_o)
Great Valley 4b (Gordon Valley)	20.65 ()	West	7.17	0.91
Hunting Creek – Berryessa	35.39 ()	Northwest	7.18	1.87
Great Valley 4a (Trout Creek)	21.23 ()	West	6.92	1.14

[1]UCERF is the third Uniform California Earthquake Rupture Forecast that provides authoritative estimates of the magnitude, location, and likelihood of earthquake fault rupture throughout the state.

We note that other faults in the region not listed in this table may also generate strong seismic shaking at the project site.

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). The Midland Fault Zone trends northwest/southeast towards Dixon but is not aligned towards the site. This fault is not active and is considered “undifferentiated Quaternary”. We conclude 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. In Boring B-2, the method of drilling was switched from solid stem augers to the rotary wash method of drilling at a depth of approximately 20 feet bgs prior to encountering groundwater. Groundwater was subsequently not encountered in Borings B-3 and B-4 which were only advanced to about 30 feet bgs The borings primarily encountered cohesive lean clay to the maximum depths explored for the borings. Dense to very dense silty and clayey sand was encountered in the borings. Based on the presence of primarily cohesive clay soils and deep groundwater, 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 20 miles west of the Stockton. 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

The near surface soils consisted of undocumented fill underlain by alluvial soils.

3.1.1. Undocumented Fill

The upper 1 foot of soil consisted of undocumented fill that was likely disturbed from prior backfill of trees and existing structures on the site. The undocumented fill was comprised of lean clay with sand with a medium plasticity. We note some vegetation was observed at the surface.

3.1.2. Alluvium

Below the undocumented fill, we encountered native alluvium which was comprised of stiff to hard lean clay with sand, lean clay, and fat clay to the maximum depths explored at about 31½ and 101½ feet bgs. There were isolated layers of medium dense to dense silty sand and clayey sand, and medium dense sandy silt. Pocket penetrometer tests performed on the cohesive soil samples encountered ranged from 0.75 tsf to greater than 4.5 tsf.

3.2. GROUNDWATER CONDITIONS

In Boring B-2, the method of drilling was switched from solid stem augers to the rotary wash method of drilling at a depth of approximately 20 feet bgs prior to encountering groundwater. Groundwater was subsequently not encountered in Borings B-3 and B-4 which were only advanced to about 30 feet bgs as summarized in Table 3.1.

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

Boring	Depth, bgs (ft)	Comments
B-1	---	Not encountered during drilling
B-2	---	Not encountered during drilling
B-3	---	Not encountered during drilling
B-4	---	Not encountered during 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: Groundwater Levels from DWR Wells

Well Site	Distance and Direction	Ground Elevation (ft)	Measured Depth to Water (ft)	Last Measurement Date
384520N1218245W001	~0.5 mile, Northwest	60.0	42.0	3/1/23
384261N1218264W001	~1.2 miles, South	51.0	42.1	9/26/23
384530N1217847W001	2 miles, Northeast	52.6	45.9	10/3/23

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 40 feet of the surface and **not expected to be a design consideration.**

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 design and construction:

4.1. UNDOCUMENTED FILL

The presence of undocumented fill from backfill of prior structures and prior grading activity at the site may cause undesirable settlement unless they are mitigated. We recommend that loose soils (undocumented fill) be overexcavated and recompacted to provide an engineered fill for foundation support.

- Prior to site grading, the upper 18 inches within the building footprint and 5 feet beyond should be moisture conditioned and recompacted in place utilizing the existing site soil. Refer to Section 5.9 for earthwork requirements.
- 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 5.9.

4.2. 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 are considered to have a high plasticity based on the PI of 16 and 25. The soils were tested to have an expansion index (EI) of 58 indicating a medium potential for expansion. During rough and finish grading, we recommend these "expansive soils" not be allowed within the upper 12 inches of building pad subgrades. Where exposed, expansive soils in the upper 12 inches of building pads and pavements should be chemically treated with lime. ***The treatment should extend at least 5 feet beyond the building footprint.*** For flatwork, the grading contractor should 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.**

For specific earthwork recommendations, refer to Section 5.9 through 5.11.

4.3. OTHER CONCLUSIONS

Organics present near the surface should be stripped and disposed offsite. For acceptance purposes, they should have a maximum organic content of 3 percent per ASTM D2974. The contractor should consider the requirements needed to achieve maximum organic content of 3 percent which may necessitate discing or deeper stripping and removal. We anticipate that organics will be addressed during site grading.

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.

PART 5. RECOMMENDATIONS

5.1. SHALLOW SPREAD FOUNDATIONS

5.1.1. Allowable Design Criteria

Shallow spread foundations may be incorporated for structure when designed according to the following parameters presented in Table 5.1.

Table 5.1: Shallow Foundation Design Criteria

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	2,500 psf	Note 2 and 3
Estimated Total Settlement	S_{total}	1 inch	
Estimated Differential Settlement	S_{diff}	¾ inch in 30 feet	Based on Risk Category II
Allowable Passive Pressure	P_p	240 pcf	Note 5
Allowable Friction Factor	μ	0.38	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]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 main structure. 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. Passive pressure and friction 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 1809.13, 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. The site falls under Site Class D criteria so seismic ties are not required.

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. DRILLED PIER FOUNDATIONS

5.2.1. Axial Capacity

Where needed, specifically for light pole design, cast-in drilled hole concrete piers having a minimum diameter of 18 inches can be used. For axial capacity, the piers may be designed using the following values:

Table 5.2: Axial Capacity for Drilled Piers in Clay

Depth from top of pier (ft)	Soil Type	Average Friction Angle, ϕ (sands)	Average Undrained Shear Strength (clay) (psf)	Allowable Unit Skin Friction (psf)
0-20	Clay	---	2,000	333

The upper 24 inches of the shaft should be neglected in the determination of the frictional component of the axial capacity. The frictional capacity of the pier was based on a safety factor of 3.

The allowable uplift capacity may be taken as 125 psf plus the weight of the pier. The uplift capacity of the pier was based on a safety factor of 6.

5.2.2. Estimated Settlement

We estimate that the proposed structure, designed and constructed as recommended herein, will undergo total settlements of up to approximately 1 inch. Differential settlements are typically less than about one-half of the total settlement.

5.2.3. Lateral Resistance

The depth of drilled piers required to resist lateral loads may be determined using the design criteria established in Section 1807A.3.2 of the 2022 California Building Code. For areas where no lateral constraint is provided at the ground surface, such as by rigid floor or pavement, the nonconstrained formula (Equation 18A-1) may be used. For areas where lateral constraint is provided at the ground surface, such as by rigid floor or pavement, the constrained formula (Equation 18A-2 or 18A-3) may be used. The allowable lateral bearing pressure of 100 psf/ft may be used in determining the required depth.

5.2.4. Construction Considerations

Foundation excavations should be firm, neat, plumb, and clean of debris, loose or soft soil, or water prior to placing any reinforcement. All pier excavations should be observed by the project Geotechnical Engineer's representative just prior to placing reinforcing steel or concrete to verify the recommendations presented herein are implemented during construction. The inspections should also verify immediately that excessive sloughing and/or caving has not reduced the required hole depth. This may be accomplished by using a weighted tape measure or similar measuring device. Steel reinforcement should be placed the same day the concrete will be placed. Additionally, drilled pier excavations should be scheduled to allow concrete in each pier to set over night before drilling adjacent holes that are closer than 4 diameters center to center.

Concrete used for drilled pier construction should be discharged vertically into the drilled holes to reduce aggregate segregation. The pier concrete should not be allowed to free fall against the steel reinforcement or sides of the excavation. Sufficient space should be provided in the pier reinforcement cage during fabrication to allow insertion of a pump hose or tremie tube for concrete placement. The pier reinforcement cage should be installed, and the concrete pumped and vibrated during placement immediately after drilling is completed.

To develop the skin friction values, concrete used for drilled pier construction should have a slump of 4 to 6 inches for dry placement methods or at least 8 inches if slurry drilling is used. The concrete mix should be designed by a registered design professional to include admixtures and/or water cement ratios to achieve the recommended slumps. We do not recommend adding water to achieve slump.

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, footings should be extended below a 2 horizontal to 1 vertical, 2(h) to 1(v), plane projected upward from the closest bottom corner of the trench. Foundation excavations within clay soils that are left exposed for extended periods of time may shrink and result in cracking at the surface. They should be kept moist to seal the cracks prior to placing reinforcing steel and concrete.

5.3. 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 5.3.

5.3.1. Active and At-Rest Pressure

Table 5.3: Lateral Earth Pressures			
Condition	Lateral Earth Pressure	Drained Case[1,3]	Undrained Case[2,3]
Active Case	P_a	40	80
At – Rest Case	P_o	60	90
Seismic Increment	P_{AE}	Not needed for walls retaining less than 6 feet in height.	

[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.

5.3.2. 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.

5.4. 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 the data from the soil borings from the interpreted blow counts, we determined the site may be classified as Site Class D.

Table 5.4 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 5.4: Seismic Design Criteria per 2022 California Building Code and ASCE 7 16		
Reference	**Seismic Parameter**	**Value**
Google Earth	Latitude	38.445746
Google Earth	Longitude	-121.82097
Table 20.3-1	Site Class	D
Table 1.5-1	Risk Category	II
Table 11.4-1	Site Coefficient for Short Period, F_A	1.105
Table 11.4-2	Site Coefficient for Long Period, F_v	1.931*
Figure 22-7	Peak Ground Acceleration, PGA	0.413g
Table 11.8-1	Site Amplification Factor, F_{PGA}	1.187
Equation 11.8-1	Peak Ground Acceleration, PGA_M	0.490g
Figure 22-1	Mapped MCE_R Spectral Response Acceleration at 0.2-second period, S_s	0.987g
Figure 22-2	Mapped MCE_R Spectral Response Acceleration at 1.0-second period, S_1	0.369g
Equation 11.4-1	Site-Adjusted MCE_R Spectral Acceleration at 0.2-second period, S_{MS}	1.091g
Equation 11.4-2	Site-Adjusted MCE_R Spectral Acceleration at 1.0-second period, S_{M1}	0.713g**
Equation 11.4-3	Design Spectral Response Acceleration at 0.2-second period, S_{DS}	0.727g
Equation 11.4-4	Design Spectral Response Acceleration at 1.0-second period, S_{D1}	0.731g
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	8 sec
	Short-period transition, $T_S = S_{D1}/S_{DS}$	0.981 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.

5.5. CORROSIVITY

The American Concrete Institute (ACI) 318 code, Table 19.3.2.1 is reproduced in Table 5.5 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 5.5: 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 5.6: 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 5.6.

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.

5.6. 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 thickness may need to be increased depending on anchorage requirements. Moisture barriers should be considered if moisture sensitive floor coverings, coatings, underlayments, adhesives, moisture sensitive goods, humidity-controlled environments, or climate-cooled environments are initially anticipated, or intended in the future. If a moisture barrier is to be laid to protect floor finishes, we recommend the permeance as tested before and after mandatory conditioning be less than 0.01 perms, 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 be 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.

5.7. EXTERIOR FLATWORK

Exterior flatwork for pedestrian traffic should be at least 4 inches thick. The flatwork should be underlain by aggregate base with thickness by at least 6 inches or whichever thickness is required in the local jurisdiction. The base materials should be placed 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. **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.** 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 designer engineer should determine the final slab thickness, reinforcing, and joint spacing based upon the anticipated loads.

5.8. FLEXIBLE AND RIGID PAVEMENTS

5.8.1. 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 6. 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 and include lime treated subgrade to address the expansive soils present at the site. To account for potential variability in the subgrade soils, pavement sections are based on a reduced subgrade R-value equal to 5.

Table 5.7: Recommended Flexible (Asphalt Concrete) Pavement Sections

Traffic Index[1]	Asphalt Concrete (in)	Class 2 Aggregate Base (in)	Geogrid[2]	Lime Treated Subgrade[3] (in)	Total Section (in)
5	3	10	---	---	13
5	3	6	InterAx 750	---	9
5	3	5	---	9	8
6	3 1/2	13	---	---	16 1/2
6	3 1/2	8	InterAx 750	---	11 1/2
6	3 1/2	6	---	9	9 1/2
7	4	16	---	---	20
7	4	9	InterAx 750	---	13
7	4	7	---	12	11
8	5	18	---	---	23
8	5	11	InterAx 750	---	16
8	5	8	---	12	13
9	5 1/2	21	---	---	26 1/2
9	5 1/2	12	InterAx 750	---	17 1/2
9	5 1/2	10	---	12	15 1/2

[1] Traffic Index provided is a range based on assumed traffic volume. The project civil engineer should verify the appropriate TI to be used in design.
[2] Geogrid based on the performance criteria of Tensar ™ InterAx 750 placed per manufacturer's recommendations at the bottom layer.
[3] Lime treated subgrade based on details presented in Section 5.8.2.

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.

5.8.2. Lime-Treated Subgrade for Pavements

The subgrade soils can be stabilized by mixing in a chemical additive such as lime. The lime is mixed with the soil at a prescribed rate using specialized contractors equipped with pulverizer/mixing equipment. There are several specialty contractors in the area providing such services. The lime treatment process is typically a two -day process, wherein the lime is blended with the soil on the first day, then allowed to season (or mellow), then re-mixed, moisture conditioned and compacted on a subsequent day. Lime is typically delivered in granular form and creates a violent reaction upon contact with the water in the clay soils.

The benefits of chemical treatment with lime can be realized with reduced expansion potential, lowered moisture content, and increased subgrade strength. For estimating purposes, the lime should be applied with a spread rate of 4 percent by dry weight based on a dry unit weight of 110 pounds per cubic feet (pcf). For a 12-inch-thick layer to be treated, the lime should be spread at rate of 4.4 pounds per square foot (psf). The spread rate should be confirmed by pan weights during placement. We note that a lime – mix design was not performed for this study. If it is desired to utilize lime for stabilization or providing a workable pad, we recommend a lime-mix design be performed prior to earthwork. This may include the standard test for using pH to estimate the soil-lime requirement for soil stabilization as outlined in ASTM D6276 and other index tests to verify acceptability. Treatment should be performed only by a specialty contractor experienced in chemical treatment using a rotary mixing machine. All the local specialty contractors can mix to 12- or 18-inches depth in a single pass. Treatment by windrowing with a motor grader or similar is not acceptable.

Lime treatment should comply with the provisions outlined in California Department of Transportation (Caltrans) Standard Specifications, latest edition. During placement, the treated material should be mixed on the same day applied and allowed to mellow for at least 36 hours prior to final mixing and compaction. Following thorough blending, seasoning, and remixing, the treated soils should be uniformly compacted to at least 95 percent relative compaction at a moisture content of at least 3 percent over optimum moisture per the California Test 216 method. Following the implementation of the lime treatment and prior to placement of AB, the treated areas should be proof rolled to detect for any soft zones or unstable areas. If any zones are detected, additional stabilization may be required.

It is essential a representative of the Geotechnical Engineer be requested onsite to monitor the treatment process and provide testing services as required. The treatment process should be performed to confirm the minimum spread rate and depth recommended above are achieved. Bulk samples should be collected on the completed mix to confirm the resistance value assumed will meet the design requirements. We also suggest that any materials proposed be submitted for testing to confirm the quality of the material meets regulatory and standard specifications for the specific material being used (i.e., gradation, sand equivalent, etc.)

5.8.3. 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 5.8 is based on a design composite subgrade modulus of 225 pci for rigid pavement over aggregate baserock or 271 pci, for rigid pavement over aggregate baserock over a lime treated subgrade. 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 5.8: Recommended Rigid (Portland Cement Concrete) Pavement Sections					
Traffic Classification[1] assuming ACI 330 Category B	Rigid Concrete (in)	Class 2 Aggregate Base (in)	Lime Treated Subgrade (in)	Total Section (in)	Max Joint Spacing (ft)
Light – ADTT[2] = 3	5	12	---	17	10
Light – ADTT[2] = 3	4½	4	12	8½	8
Moderate – ADTT = 10	5½	12	---	17½	11
Moderate – ADTT = 10	5	4	12	9	9

[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 8 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 or they are chemically treated with lime.

5.8.4. 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, or a minimum of 95 percent based on the California Test Method 216 method when stabilized with lime.
- 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.

5.9. EARTHWORK

5.9.1. 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. 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.

5.9.2. 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.

5.9.3. 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 should not be allowed within the upper 12 inches of building pads. Specific requirements for engineered fill including the applicable test procedures to verify suitability are presented in Table 5.9.

Table 5.9: 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.

5.9.4. 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.

5.9.5. Lime Stabilization

An alternative to removing unstable subgrades due to wet conditions and replacing with engineered fill within in the areas is to chemically treat the upper 12 inches of building pads and pavements with high calcium quicklime. When lime is mixed with soil in a hydroxide state, it creates a chemical (pozzolanic) reaction where the moisture film surrounding the clay particles is released and allowed to evaporate. The reaction creates a gel structure and acts as cement after the clay particles agglomerate into coarse friable particles that harden with time after compaction. Thus, the benefit of chemical treatment is threefold in that it reduces the plasticity of the soil, decreases the moisture content to manageable levels, and increases the strength of pavement bearing soils.

For estimating purposes, the lime should be applied with a spread rate of 4 percent by dry weight based on a dry unit weight of 110 pounds per cubic feet (pcf). For a 12-inch-thick layer to be treated, the lime should be spread at rate of 4.4 pounds per square foot (psf). The spread rate should be confirmed by pan weights during placement. We note that a lime – mix design was not performed for this study. If it is desired to utilize lime for stabilization or providing a workable pad, we recommend a lime-mix design be performed prior to earthwork. This may include the standard test for using pH to estimate the soil-lime requirement for soil stabilization as outlined in ASTM D6276 and other index tests to verify acceptability.

Lime treatment should comply with the provisions outlined in California Department of Transportation (Caltrans) Standard Specifications, latest edition. During placement, the treated material should be mixed on the same day applied and allowed to mellow for at least 36 hours prior to final mixing and compaction. The moisture content should be at least 3 percent above optimum moisture content and compacted to at least 95 percent relative compaction based on the Caltrans Test Methods.

5.9.6. 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.

5.10. EXCAVATIONS

5.10.1. Temporary Excavations and Excitability

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.

5.10.2. 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.

5.10.3. 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.

5.10.4. 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.

5.10.5. 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.

5.10.6. 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.

5.11. 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 5.10. Jetting should not be allowed.

Table 5.10: Compaction and Moisture Conditioning Summary

Area to be Compacted	Minimum Relative Compaction (RC) [1, 3]	Moisture Content[2] Required
Non-Expansive Engineered fill (Import)	≥90%	0 to 3% > optimum moisture
Subgrade prior to placing fill	88%<RC<92%	Min 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 upper 12 inches of Building Pad	≥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 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 nuclear methods (ASTM D6938).

[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.

5.12. 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.

5.13. 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.

As a guide, the areas noted should be tested at the minimum frequencies below or modified accordingly to the project geotechnical engineer during construction. It is essential the project geotechnical engineer be engaged early in the project and to attend any pre-construction (preparatory) meetings related to earthwork and/or foundation construction.

Table 5.11: Minimum Testing Summary

Area to be Compacted	Min. Frequency of In Place Density / Moisture Content
Non-Expansive Engineered fill (Import)	1 per 600 cy or lift
Subgrade prior to placing fill	1 per 5,000 sf
Expansive soils (in place compaction, if encountered)	1 per 600 cy or lift
Trench backfill[6]	1 per 50 to 100 lf of trench per lift
Upper 12 inches of Trench backfill in paved areas	1 per 50 to 100 lf of trench per lift
Lime Treatment as Engineered Fill (if used)	1 per 10,000 sf, or additional as needed
Lime Treatment as Pavement Subgrade (if used)	1 per 10,000 sf, or additional as needed
Upper 12 inches of pavement subgrade	1 per 5,000 sf
Aggregate Baserock for pavement[5] section	1 per 5,000 sf
Hot Mix Asphalt (HMA) compaction	1 per 50 tons placed

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

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PLATES

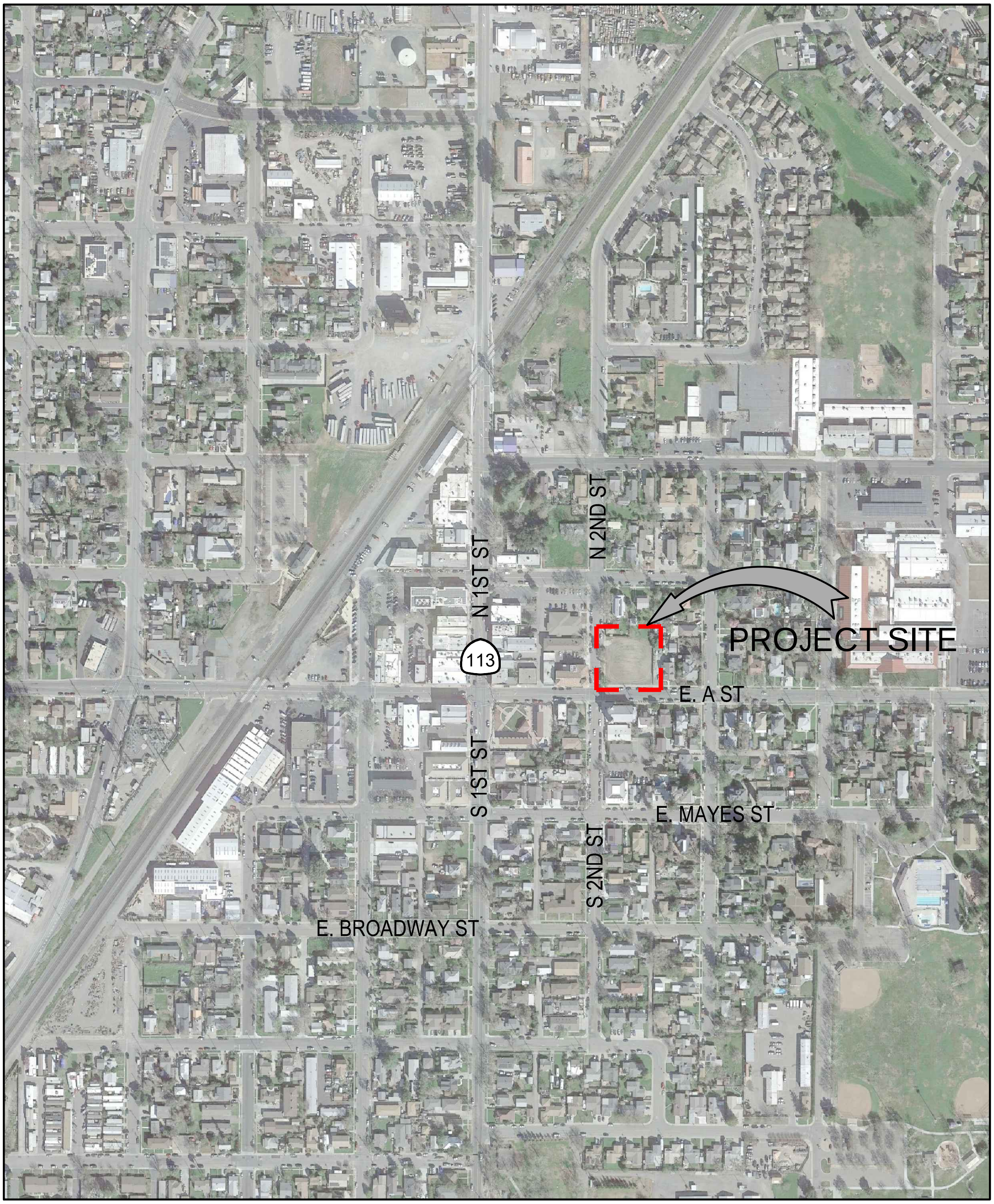
STOCKTON
3428 Brookside Rd.
Stockton, CA 95219
t: 209.943.2021

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

F:\23PROJECTS\23217 V-LLEY OF THE S-CRED HE-RT EDUC-TION CENTER - DIXON\PL-NS -ND GR-PHICS\EXHIBITS\GEOTECH EXHIBITS\23217 - SITE LOC-TION M-P.DWG 2/15/2024 8:22 ,



Google Earth 2023

SITE LOCATION MAP

VALLEY OF THE SACRED HEART EDUCATION CENTER

219 E. A ST, DIXON, CA 95620

DATE	02/15/24
DESIGN	BLQ
DRAWN	AA
JOB NO.	23217

SIEGFRIED

- CIVIL
- STRUCTURAL
- LANDSCAPE ARCHITECTURE
- SURVEYING
- PLANNING
- ATHLETIC FACILITY DESIGN
- GEOTECHNICAL

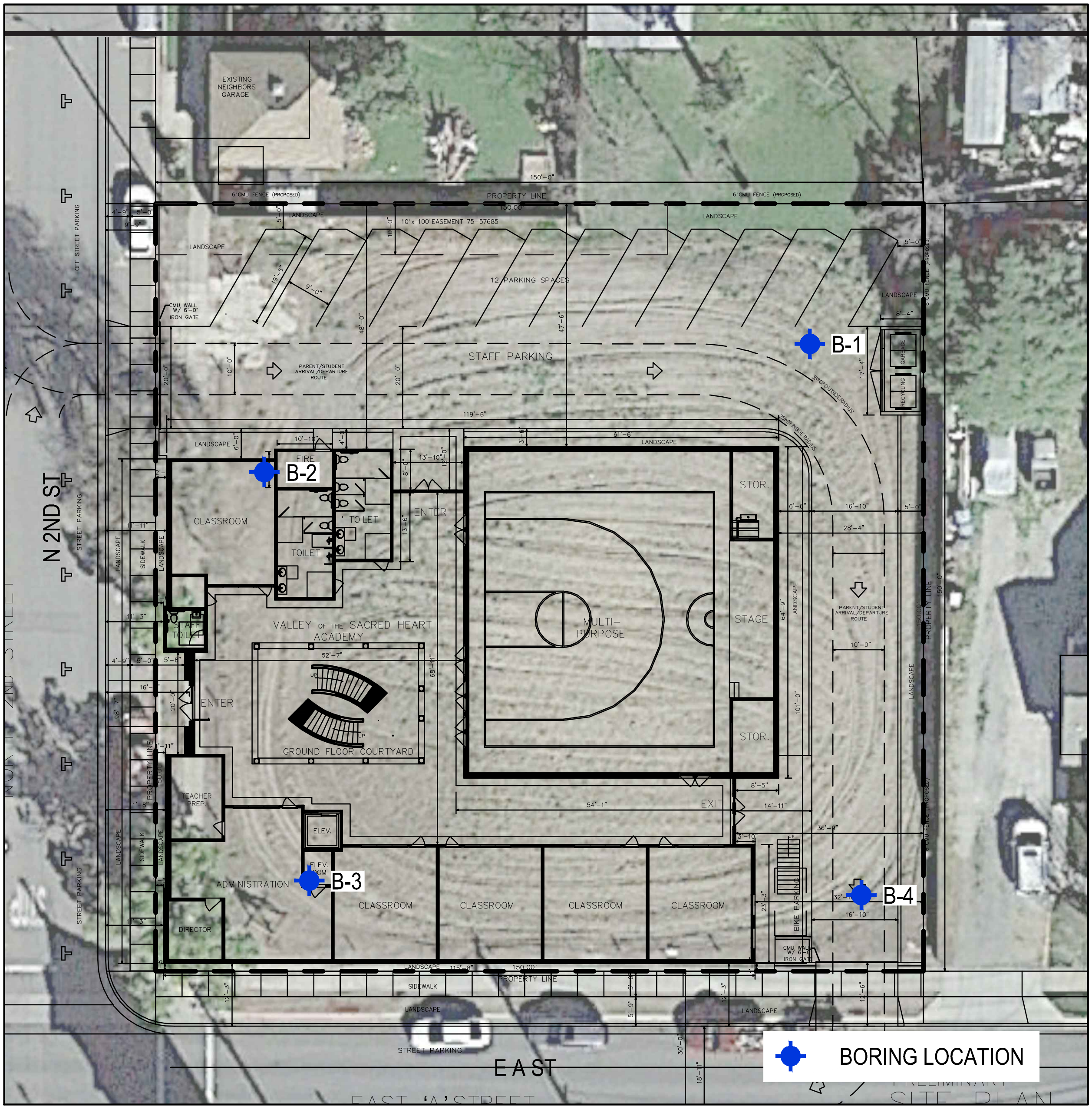
3428 Brookside Road, Stockton, California 95219
209-943-2021 www.siegfriedeng.com

SCALE: 1" = 500'

PLATE

1

F:\23PROJECTS\23217 VALLEY OF THE SACRED HEART EDUCATION CENTER – DIXON\PLANS AND GRAPHICS\EXHIBITS\GEOTECH EXHIBITS\23217 – EXPLORATION LOCATION MAP.DWG 2/15/202



BORING NOTES

B-1: 5' BORING WITH BULK
B-2 AND B-4: 30' BORING
B-3: 100' BORING WITH BULK

SOURCE: GOOGLE EARTH 2023

BASE MAP FROM PROJECT ARCHITECT

EXPLORATION LOCATION MAP

VALLEY OF THE SACRED HEART EDUCATION CENTER

219 E. A ST, DIXON, CA 95620

DATE	02/15/24
DESIGN	BLQ
DRAWN	AA
JOB NO.	23217

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SCALE: 1" = 30'

PLATE 2

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 Solano County Environmental Health Department would be required for the borings.

The soil borings were advanced on January 18, 2024 by Geo-Ex Subsurface Exploration. 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 4 and 100 feet bgs around within the site. The shallow Boring B-1 was advanced with a hand auger and the deeper borings B-2, B-3, and B-4 were advanced with a truck -mounted CME 75 drill rig equipped with 4-inch outer diameter solid-stem augers. Boring B-2 was converted to the rotary wash method of drilling between 25 and 30 feet bgs.

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 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.

The borings were backfilled with cement grout upon completion.

Boring Log Legend

Sample Type: **Split Barrel Sampler with 3” outer diameter barrel** and 2.4” inner diameter brass or stainless liners

6B — Sample ID 6B

6C — Sample ID 6C

SPT 7 — Sample ID SPT 7

Sample Type: **Standard Penetration Test (SPT) Sampler**

9-12-20 — # of blows with 140-pound hammer free falling 30 inches for (3) consecutive 6-inch increments

(32) — N Value: Sum of the final # of blows for the final 12 inches of driving

BOREHOLE NUMBER B-1

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CLIENT	Valley of the Sacred Heart Academy	**PROJECT NAME**	Valley of the Sacred Heart Education Center - Dixon
PROJECT NUMBER	23217-5001	**PROJECT LOCATION**	219 E. A Street, Dixon, California, 95620
DATE STARTED	01-19-2024 **COMPLETED** 01-19-2024	**POSITION**	
DRILLING CONTRACTOR	Hanlon Drilling	**GROUND ELEVATION**	**FINAL DEPTH** 4.00 ft
DRILLING METHOD	Hand Auger	**GROUNDWATER LEVELS:**	
EQUIPMENT	Hand Auger	▽ **AT TIME OF DRILLING**	
HOLE SIZE	4.0 in.	▼ **AT END OF DRILLING**	Not Encountered
LOGGED BY	RJ Teel **CHECKED BY**	▽ **AFTER DRILLING**	

DEPTH (ft)	GRAPHIC LOG		MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER
		1.00	Lean CLAY (CL): dark brown; moist; medium plasticity; disced surface with vegetation, soft, undocumented fill.	
		2.00	Lean CLAY (CL): dark brown; moist; medium plasticity.	
		3.50	Sandy Lean CLAY (CL): brown; moist; medium plasticity.	
		4.00	Lean CLAY with sand (CL): brown; dry to moist; medium plasticity.	
5			Terminated at 4.00 ft.	
10				
15				
20				
25				

NOTES

SIEGFRIED

BOREHOLE NUMBER B-2

CLIENT Valley of the Sacred Heart Academy
PROJECT NAME Valley of the Sacred Heart Education Center - Dixon
PROJECT NUMBER 23217-5001
PROJECT LOCATION 219 E. A Street, Dixon, California, 95620
DATE STARTED 01-18-2024 **COMPLETED** 01-18-2024
POSITION
DRILLING CONTRACTOR Geo-Ex Subsurface Exploration
GROUND ELEVATION **FINAL DEPTH** 101.00 ft
DRILLING METHOD Solid Stem Auger / Mud Rotary
GROUNDWATER LEVELS:
EQUIPMENT CME 75
AT TIME OF DRILLING
HOLE SIZE 4.0 in.
AT END OF DRILLING Not Measured
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)	POCKET PEN. (tsf)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
		Lean CLAY with sand (CL): brown; moist; medium plasticity; disced surface with vegetation, soft, undocumented fill.									
	1.00	Lean CLAY (CL): hard; dark brown with gray mottling; moist; medium plasticity.	1B 1C	4-12-19 (31)	2.50 4.50		20				
	2.50	: very stiff; brown.	SPT 2	9-8-9 (17)			17				
5	5.00	Lean CLAY with sand (CL): stiff; brown; moist; medium plasticity.	3B 3C	6-6-6 (12)	4.50 4.50	100	16 20	45	20	25	80
	7.50	Sandy Lean CLAY (CL): very stiff; brown; moist; medium plasticity.	4B 4C	5-8-12 (20)	2.50	103	21				
10	10.00	Lean CLAY (CL): very stiff; light brown with white mottling; moist; medium plasticity.	5B 5C	8-10-15 (25)	3.25	107	22				
15	15.00	Lean CLAY with sand (CL): hard; light brown with black mottling; moist; medium plasticity.	6B 6C	9-12-20 (32)	4.25	111	20				
20	20.00	Sandy Lean CLAY (CL): very stiff; light brown with black and white mottling; moist; medium plasticity; driller switched to mud rotary.	SPT 7	7-9-13 (22)			23				
25	25.00	Lean CLAY with sand (CL): hard; light brown with white mottling; moist; medium plasticity.	8B 8C	15-18-28 (46)	4.50	107	20				

NOTES

BOREHOLE NUMBER B-2

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CLIENT Valley of the Sacred Heart Academy
PROJECT NAME Valley of the Sacred Heart Education Center - Dixon
PROJECT NUMBER 23217-5001
PROJECT LOCATION 219 E. A Street, Dixon, California, 95620

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
		Lean CLAY with sand (CL): hard; light brown with white mottling; moist; medium plasticity.									
30	30.00	Sandy Lean CLAY (CL): very stiff; light brown with white mottling; moist; medium plasticity.	SPT 9	8-10-13 (23)			25				
35	35.00	: hard; light brown with black mottling.	10B 10C	10-15-16 (31)	4.50	103	24				
40	40.00	Lean CLAY (CL): stiff; light brown with black and rust mottling; moist; medium plasticity.	SPT 11	5-7-8 (15)			30				
45	45.00	Sandy Lean CLAY (CL): hard; light brown with white mottling; moist; medium plasticity.	12B 12C	12-17-21 (38)	3.75	104	24				
50	50.00	Sandy SILT (ML): medium dense; brown; moist; nonplastic.	SPT 13	7-7-11 (18)			32				58
55											

NOTES

CLIENT Valley of the Sacred Heart Academy
PROJECT NAME Valley of the Sacred Heart Education Center - Dixon
PROJECT NUMBER 23217-5001
PROJECT LOCATION 219 E. A Street, Dixon, California, 95620

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
		Sandy SILT (ML): medium dense; brown; moist; nonplastic.									
60	60.00	Lean CLAY with sand (CL): hard; brown with white, black and rust mottling; moist; medium plasticity.	SPT 14	10-13-19 (32)			27				
65											
70	70.00	Sandy Lean CLAY (CL): very stiff; brown with rust mottling; moist; low plasticity.	SPT 15	8-11-13 (24)			23				68
75											
80	80.00	Poorly-graded SAND with silt (SP-SM): medium dense; brown with white, black and rust mottling; moist; nonplastic.	SPT 16	10-14-16 (30)			25				46
	80.33	Silty SAND (SM): medium dense; brown with white, black and rust mottling; moist; nonplastic.									
85											

NOTES

SIEGFRIED

BOREHOLE NUMBER B-2

CLIENT Valley of the Sacred Heart Academy
PROJECT NAME Valley of the Sacred Heart Education Center - Dixon
PROJECT NUMBER 23217-5001
PROJECT LOCATION 219 E. A Street, Dixon, California, 95620

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
		Silty SAND (SM): medium dense; brown with white, black and rust mottling; moist; nonplastic.									
90	90.00	Sandy SILT (ML): medium dense; brown; moist; nonplastic.	SPT 17	10-14-15 (29)			32				39
95											
100	100.00	Clayey SAND with gravel (SC): hard; dark brown; moist; nonplastic.	SPT 18	42-50/5"			19				23
	101.00	Terminated at 101.00 ft.									
105											
110											
115											

NOTES

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BOREHOLE NUMBER B-3

CLIENT Valley of the Sacred Heart Academy
PROJECT NAME Valley of the Sacred Heart Education Center - Dixon
PROJECT NUMBER 23217-5001
PROJECT LOCATION 219 E. A Street, Dixon, California, 95620
DATE STARTED 01-19-2024 **COMPLETED** 01-19-2024
POSITION
DRILLING CONTRACTOR Geo-Ex Subsurface Exploration
GROUND ELEVATION
FINAL DEPTH 31.50 ft
DRILLING METHOD Solid Stem Auger
GROUNDWATER LEVELS:
EQUIPMENT CME 75
AT TIME OF DRILLING
HOLE SIZE 4.0 in.
AT END OF DRILLING Not Encountered
LOGGED BY Alejandro Aguilera, EIT **CHECKED BY** Charley Scott, PE
AFTER DRILLING

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0.00–1.00	Lean CLAY with sand (CL): brown; moist; medium plasticity; disced surface with vegetation, soft, undocumented fill.									
1.00	Lean CLAY (CL): firm; dark brown; moist; medium plasticity.	1B 1C	5-4-3 (7)	0.75		21				
2.50	Lean CLAY with sand (CL): stiff; dark brown; moist; medium plasticity.	2B 2C	9-6-7 (13)	1.50	102	22				
5.00	: brown.	3B 3C	6-8-7 (15)	2.00	97	23				
7.50	: very stiff.	4B 4C	4-7-9 (16)	1.00 1.25	99 103	23 24	33	17	16	77 83
10.00	Lean CLAY (CL): very stiff; brown with white and black mottling; moist; medium plasticity.	5B 5C	7-10-14 (24)	3.25	105	22				
15.00	Lean CLAY with sand (CL): very stiff; light brown with white and black mottling; moist to wet; medium plasticity.	6B 6C	6-10-16 (26)	4.50	108	21				
20.00	Sandy Lean CLAY (CL): very stiff; light brown with white and black mottling; moist; low to medium plasticity.	SPT 7	8-10-16 (26)			22				
25.00	: hard; light brown with white mottling.	8B 8C	9-20-32 (52)	4.50	111	18				

NOTES

SIEGFRIED

BOREHOLE NUMBER B-3

CLIENT Valley of the Sacred Heart Academy

PROJECT NAME Valley of the Sacred Heart Education Center - Dixon

PROJECT NUMBER 23217-5001

PROJECT LOCATION 219 E. A Street, Dixon, California, 95620

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
		: hard; light brown with white mottling.									
30		30.00 : very stiff.	SPT 9	7-10-13 (23)			22				
		31.50									
		Terminated at 31.50 ft.									
35											
40											
45											
50											
55											

NOTES

BOREHOLE NUMBER B-4

SIEGFRIED

CLIENT Valley of the Sacred Heart Academy
PROJECT NAME Valley of the Sacred Heart Education Center - Dixon
PROJECT NUMBER 23217-5001
PROJECT LOCATION 219 E. A Street, Dixon, California, 95620
DATE STARTED 01-19-2024 **COMPLETED** 01-19-2024
POSITION
DRILLING CONTRACTOR Geo-Ex Subsurface Exploration
GROUND ELEVATION **FINAL DEPTH** 31.50 ft
DRILLING METHOD Solid Stem Auger
GROUNDWATER LEVELS:
EQUIPMENT CME 75
∇ **AT TIME OF DRILLING**
HOLE SIZE 4.0 in.
▼ **AT END OF DRILLING** Not Encountered
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)	POCKET PEN. (tsf)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	FINES CONTENT (%)
		Lean CLAY with sand (CL): dark brown; moist; medium plasticity; some vegetation on surface, material was disked and is very loose, undocumented fill.						
1.00		Lean CLAY (CL): very stiff; dark brown; moist; medium plasticity.	1B 1C	6-9-11 (20)	2.75		26	
2.50		Lean CLAY with sand (CL): stiff; brown; moist; medium plasticity.	2C	5-6-8 (14)			26	
5 — 5.00		: stiff.	3B 3C	3-6-9 (15)		99	23	
7.50		: very stiff.	4B 4C	5-12-16 (28)	2.50	105	22	
10 — 10.00		Fat CLAY (CH): very stiff; brown with white mottling; moist; high plasticity.	5B 5C	7-10-12 (22)	4.50	103	23	
15 — 15.00		Lean CLAY with sand (CL): very stiff; light brown with white and black mottling; moist; medium plasticity.	SPT 6	5-7-9 (16)			22	
20 — 20.00		Clayey SAND (SC): dense; brown with black mottling and gray seams; low plasticity.	7B 7C	10-13-18 (31)	4.50 4.50	114	16	49
25 — 25.00		Sandy Lean CLAY (CL): very stiff; light brown with white and black mottling; moist; medium plasticity.	SPT 8	8-11-14 (25)			18	

NOTES

CLIENT Valley of the Sacred Heart Academy

PROJECT NAME Valley of the Sacred Heart Education Center - Dixon

PROJECT NUMBER 23217-5001

PROJECT LOCATION 219 E. A Street, Dixon, California, 95620

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT (pcf)	MOISTURE CONTENT (%)	FINES CONTENT (%)
		Sandy Lean CLAY (CL): very stiff; light brown with white and black mottling; moist; medium plasticity.						
30	30.00	: hard; brown with white mottling and gray seams.	9B 9C	11-17-22 (39)	4.00	101	25	
	31.50	Terminated at 31.50 ft.						
35								
40								
45								
50								
55								

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.

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Geotechnical Materials Testing Summary

Tested in General Accordance with ASTM D1140, D2487, D2974, D4318, D6913, and D7263.

Project Name: Valley of the Sacred Heart Education Center

Project Number: 23217-5001

Project Location: Dixon, 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
1/19/2024	B-1 Bulk	1.0	2.0	Dark Brown	---	---	---	43	21	---	---		CL	Lean Clay
1/18/2024	B2-1C	2.0	2.5	Dark Brown with Gray Mottling	19.5	---	---	---	---	---	---		---	---
1/18/2024	B2 - SPT 2	2.5	4.0	Brown	16.6	---	---	---	---	---	---		---	---
1/18/2024	B2-3B	5.5	6.0	Dark Brown	16.4	---	---	45	25	0	20	80	CL	Lean Clay with Sand
1/18/2024	B2-3C	6.0	6.5	Brown	20.1	119.6	99.6	---	---	---	---		---	---
1/18/2024	B2-4C	8.5	9.0	Brown	21.3	124.6	102.7	---	---	---	---		---	---
1/18/2024	B2-5C	11.0	11.5	Light Brown with White Mottling	21.5	129.5	106.6	---	---	---	---		---	---
1/18/2024	B2-6C	16.0	16.5	Light Brown with Black Mottling	20.3	133.5	111.0	---	---	---	---		---	---
1/18/2024	B2- SPT 7	20.0	21.5	Light Brown with Black and White Mottling	23.4	---	---	---	---	---	---		---	---
1/18/2024	B2-8C	26.0	26.5	Light Brown with White Mottling	20.4	129.1	107.2	---	---	---	---		---	---
1/18/2024	B2- SPT 9	30.0	31.5	Light Brown with White Mottling	25.0	---	---	---	---	---	---		---	---
1/18/2024	B2-10C	35.5	36	Light Brown with Black Mottling	24.4	127.8	102.7	---	---	---	---		---	---
1/18/2024	B2 - SPT 11	40.0	41.5	Light Brown with Black and Rust Mottling	29.8	---	---	---	---	---	---		---	---
1/18/2024	B2-12C	46.0	46.5	Light Brown with White Mottling	24.3	129.4	104.1	---	---	---	---		---	---
1/18/2024	B2 - SPT 13	50.0	51.5	Brown	31.6	---	---	---	---	0	42	58	---	---
1/18/2024	B2 - SPT 14	60.0	61.5	Brown with White, Black and Rust Mottling	26.6	---	---	---	---	---	---		---	---
1/18/2024	B2 - SPT 15	70.0	71.5	Brown with Rust Mottling	23.3	---	---	---	---	0	32	68	---	---
1/18/2024	B2 - SPT 16	80.0	81.5	Brown with White, Black and Rust Mottling	24.9	---	---	---	---	0	54	46	---	---
1/18/2024	B2 - SPT 17	90.0	91.5	Brown	31.5	---	---	---	---	0	61	39	---	---
1/18/2024	B2 - SPT 18	100.0	100.9	Dark Brown	18.9	---	---	---	---	30	47	23	---	---

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

SAN JOSE
111 N. Market St., #300
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SACRAMENTO
1164 National Drive #20
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t: 916.520.2777

MODESTO
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Modesto, CA 95354
t: 209.762.3580

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Project Name: Valley of the Sacred Heart Education Center

Project Number: 23217-5001

Project Location: Dixon, 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
1/19/2024	B3-1C	2.0	2.5	Dark Brown	20.6	---	---	---	---	---	---		---	---
1/19/2024	B3-2C	3.5	4.0	Dark Brown	21.7	123.7	101.7	---	---	---	---		---	---
1/19/2024	B3-3C	6.0	6.5	Brown	23.3	119.7	97.0	---	---	---	---		---	---
1/19/2024	B3-4B	8.0	8.5	Brown	23.4	122.7	99.4	33	16	0	23	77	CL	Lean Clay with Sand
1/19/2024	B3-4C	8.5	9.0	Brown	23.7	127.7	103.3	---	---	0	17	83	---	---
1/19/2024	B3-5C	11.0	11.5	Brown with White and Black Mottling	22.2	127.8	104.6	---	---	---	---		---	---
1/19/2024	B3-6C	16.0	16.5	Light Brown with White and Black Mottling	20.6	129.9	107.7	---	---	---	---		---	---
1/19/2024	B3 - SPT 7	20.0	21.5	Light Brown with White and Black Mottling	22.0	---	---	---	---	---	---		---	---
1/19/2024	B3-8C	26.0	26.5	Light Brown with White Mottling	18.5	131.6	111.0	---	---	---	---		---	---
1/19/2024	B3 - SPT 9	30.0	31.5	Light Brown with White Mottling	22.1	---	---	---	---	---	---		---	---
1/19/2024	B4-1C	2.0	2.5	Dark Brown	25.8	---	---	---	---	---	---		---	---
1/19/2024	B4-2C	3.5	4.0	Brown	25.7	---	---	---	---	---	---		---	---
1/19/2024	B4-3C	6.0	6.5	Brown	22.8	121.9	99.2	---	---	---	---		---	---
1/19/2024	B4-4C	8.5	9.0	Brown	21.5	128.0	105.4	---	---	---	---		---	---
1/19/2024	B4-5C	11.0	11.5	Brown with White Mottling	22.8	126.9	103.3	---	---	---	---		---	---
1/19/2024	B4 - SPT 6	15.0	16.5	Light Brown with White and Black Mottling	22.5	---	---	---	---	---	---		---	---
1/19/2024	B4 -7C	21.0	21.5	Brown with Black Mottling and Gray Seams	16.1	132.4	114.1	---	---	0	51	49	---	---
1/19/2024	B4 - SPT 8	25.0	26.5	Light Brown with White and Black Mottling	17.8	---	---	---	---	---	---		---	---
1/19/2024	B4-9C	31.0	31.5	Brown with White Mottling and Gray Seams	25.4	126.4	100.8	---	---	---	---		---	---
1/19/2024	Bulk - Center of Site	1.8	3.0	Dark Brown	---	---	---	46	24	1	17	82	CL	Lean Clay with Sand

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3428 Brookside Rd.
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Expansion Index

Test Performed in General Accordance with ASTM D4829

Project Name:	Valley of the Sacred Heart Education Center	**Client:**	Valley of the Sacred Heart Academy
Project Number	23217-5001	**Sample Location:**	Bulk - Center of Site
Date Sampled:	1/19/2024	**Sampled by:**	Alejandro Aguilera
Date Tested:	2/5/2024	**Description:**	Lean Clay with Sand

Expansion Index (EI)	**58**
Molding Water Content (%)	15.6
Final Water Content (%)	22.4

Expansion Index determined by adjusting water content to achieve a degree of saturation of 48-52%.

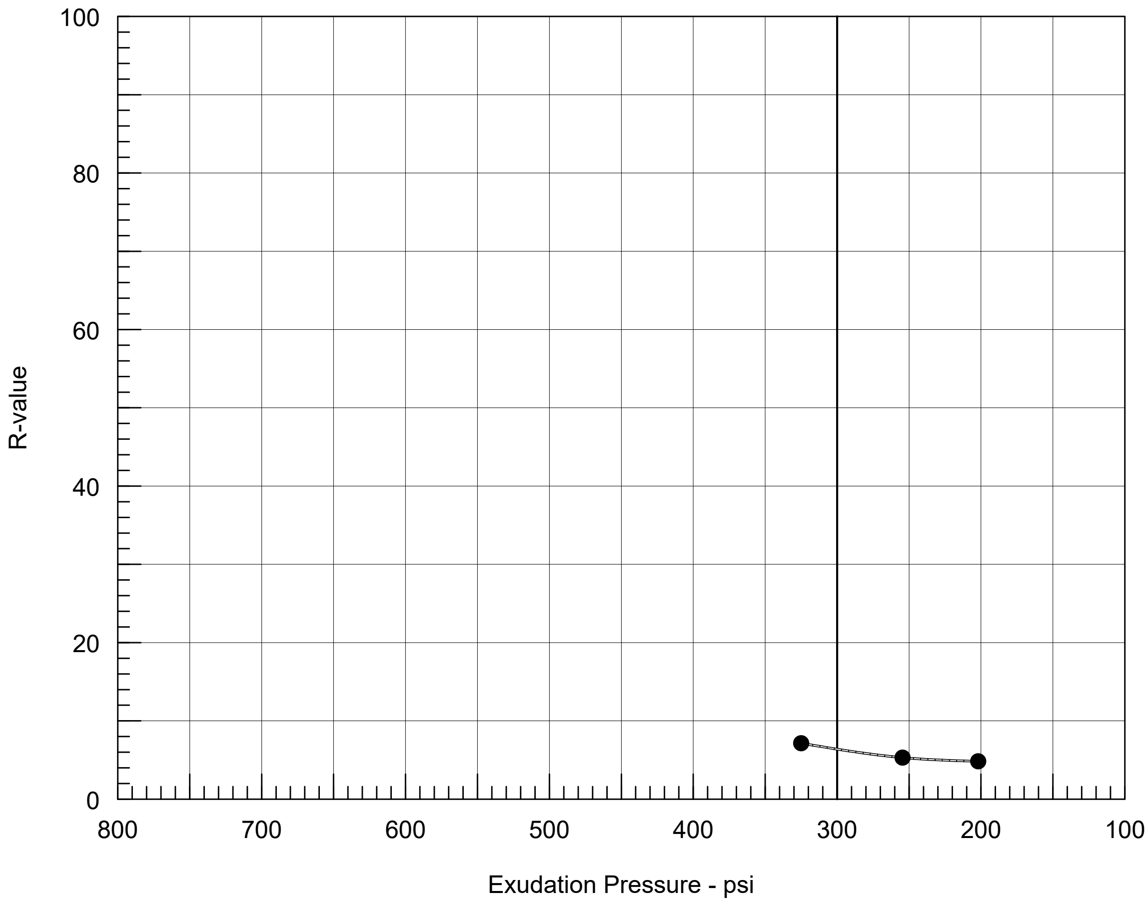
Classification of Potentially Expansive Soil

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

Tested by:	Raymond Teel	Reviewed by:	Charley Scott, PE
Title:	Engineering Technician	Title:	Senior Associate Engineer
Date:	2/13/2024	Date:	2/13/2024

Limitations: Testing results presented are for samples collected by Siegfried Engineering staff at the times and location(s) shown. Pursuant to applicable building codes or specifications, the results presented in this report are for the items listed herein and for exclusive use of the Client and the registered design professional in responsible charge. The results apply only to the samples tested and are not to be considered as a guarantee or warranty, express or implied.

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	47	98.7	24.1	39	138	2.36	325	8	7
2	42	98.0	24.7	0	144	2.43	255	6	5
3	36	97.1	25.3	0	148	2.58	202	5	5

Test Results	Material Description
R-value at 300 psi exudation pressure = 6	Fat CLAY, dark brown

Project No.: 4437.X016

Project: 23217-5001 Valley of the Sacred Heart Education Center

Source of Sample: B-1 **Depth:** 1-2'

Sample Number: Bulk

Date: 2/2/2024

Tested by: BRL
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 02/02/2024
Date Submitted 01/30/2024

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

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

The reported analysis was requested for the following location:
Location : 23217 Site ID : CENTER OF SITE.
Thank you for your business.

* For future reference to this analysis please use SUN # 91487-189640.

EVALUATION FOR SOIL CORROSION

Soil pH	6.64		
Moisture	19.7 %		
Minimum Resistivity	1.15 ohm-cm (x1000)		
Chloride	4.4 ppm	00.00044 %	
Sulfate	45.6 ppm	00.00456 %	
Redox Potential	(+) 347 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