Appendix E3

Geotechnical Engineering Report - Tuolumne Facility

Geotechnical Engineering Report

PELLET PROCESSING FACILITY

WKA No. 12774.03

June 21, 2021

Prepared for:

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Geotechnical Engineering Report

PELLET-PROCESSING FACILITY

12001 La Grange Road

Jamestown, California

WKA No. 12774.03

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Geotechnical Engineering Report
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12001 La Grange Road
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INTRODUCTION

We have completed a geotechnical engineering study for a proposed Pellet Processing Facility to be constructed at 12001 La Grange Road near Jamestown, California. The purpose of our study has been to explore the existing site, soil and groundwater conditions, and to provide geotechnical engineering conclusions and recommendations for the design and construction of the proposed improvements. This report presents the results of our study.

Scope of Services

Our scope of services for this project included the following tasks:

1. A site reconnaissance;
2. Review of historic United States Geological Survey (USGS) topographic maps, historical aerial photographs, USDA soil survey maps, various geologic maps, and available groundwater information;
3. Review of our *Phase I Environmental Site Assessment* (ESA) prepared for the subject property and previous geotechnical studies completed by others near the project site;
4. Subsurface exploration, including the excavating and sampling of 12 test pits to depths ranging from about three to eight feet below the existing ground surface (bgs);
5. Laboratory testing of selected soil samples to determine engineering properties of the soil encountered;
6. Engineering analyses; and,
7. Preparation of this report.

Figures and Attachments

This report contains a Vicinity Map as Figure 1; a Site Plan showing the test pit locations as Figure 2; and the Logs of Test Pits as Figures 3 through 14. An explanation of the symbols and classification system used in developing the exploration logs is contained on Figure 15. Appendix A contains general information regarding project concepts, the exploratory methods used during our field investigation, and the laboratory test results that are not included on the logs.

Proposed Development

We understand the 58.56-acre project site will be extensively redeveloped into a wood pellet processing facility. Based on information provided by the owner's (Golden State Natural Resources) engineering firm, the Project will include the following buildings and equipment:

- Log Crane with Log Storage - 50,000 pounds (or 50 kip) point load at center log crane pivot, 2,000 pounds per square foot (psf) maximum bearing pressure with 400-foot working diameter for log storage;
- Drum Debarker - Large, slowly rotating drum with a 200 kip operating weight distributed over multiple saddles;
- Chipper - Highly dynamic equipment with a 75 kip operating weight;
- Stacker Reclaimer w/ Chip Storage - 100 kip point load at center equipment pivot with a 2,000 psf maximum bearing pressure and 350 ft working diameter for log storage;
- Green/Dry Hammer Mills - Equipment stack-ups on standalone structure with surrounding equipment access structure. Highly dynamic equipment, 25 kips per equipment stack-up on standalone structure, and +/- 25 kips maximum column loads (DL, DL+LL) for access structure;
- Pellet Mills - Equipment stack-ups on standalone structure with surrounding equipment access structure. Highly dynamic equipment, 50 kips per equipment stack-up on standalone structure, and about 25 kip maximum column loads for access structure;
- Dryer - Large, dynamic system with ancillary equipment (e.g. fans, cyclones, etc.). Heavy equipment loads will be spread over system footprint. The rotary drum or belt dryer technology is unknown at this time;
- Silos – 60 to 80 foot working diameter silos with a 2,000 to 3,000 psf maximum bearing pressure. It is currently unknown if the silos will be flat-bottom or supported on a leg/skirt system;
- Conveyors – Used for material handling across whole site. Assumed they will be supported by bents and transfer towers with about 25 kip maximum column loads;
- Cogen Facility - Packaged turbine and generator system with ancillary equipment (e.g. fans, vessels, etc.). Heavy equipment loads will be spread over system footprint;
- Miscellaneous Buildings - Pre-Engineered Metal Buildings ranging between 20' x 20' to 100' x 200'. Functionality will range from office space to heavy fork traffic and storage areas. Column loads (DL, DL+LL) will be +/- 40 kip maximum with 400 psf maximum floor live loads; and,
- Rail - Industry standard rail siding with multiple spurs in the west-central portion of the project site.

We anticipate that associated improvements may include underground utilities, various exterior flatwork, low retaining walls and paved parking and drive aisles subjected to heavy wheel loads. Since the project gently slopes to the west, we anticipate that maximum cuts and fills should be on the order of 5 to 10 feet or less to provide level building and equipment pads.

FINDINGS

Site Description

The 58.56-acre project site is located at 12001 La Grange Road near Jamestown, California, and occupies land identified by Tuolumne County Assessor's Parcel Number 063-190-056. A vicinity map is provided as Figure 1. In general, the site is enclosed by a chainlink fence and bound on all sides by gently rolling foothills covered by grass, weeds and scattered mature oak trees. La Grange Road traverses the south and west perimeter of the property. A rail line operated by the Sierra Northern Railway also extends parallel La Grange Road on the west perimeter of the site. A wood chipping facility owned by American Wood fibers is located adjacent and southwest of the site.

A vast majority of the project site appears to have been previously disturbed or altered. At the time of our field investigation on February 17, 2021, a vacant, approximately 380,000 square foot, rectangular shaped bark and mulch facility occupied the east-central portion of the site. Approximately 14 concrete masonry unit (CMU) walled bunkers were located in the southern portion of the facility. A tall, pre-manufactured, metal building surrounded by elevated vessels and various equipment supported by concrete pedestals and slab foundations was located near the central portion of the area. A large cast-in-place concrete ramp was located north of the building. The ramp appears to have been used by vehicles to dump wood product into a large bin structure at the east end. A second large cast-in-place concrete ramp was located in the northern portion of the facility. The past use of this ramp was not apparent. The remainder of the facility was covered by asphalt-concrete pavement. In general, the eastern most pavement appeared to be in relatively good condition with only occasional cracking. Frequent longitudinal and alligator cracking, along with scattered pot holes, was noted in the western portion of the pavement.

Topographically, the facility rises about 10 to 15 feet from the south and north ends of the pavement, respectively, to the center of the facility and slopes down about 8 to 10 feet from the east perimeter. In general, the eastern one half to two thirds of the facility appears to have been cut to existing grade, while the rest was raised with fill. Fill embankments ranging from about 1 to 10 feet in vertical height were observed along the north, south and west perimeters of the pavement. A cut slope ranging from about one to over 10 feet in height was located along the east perimeter of the facility.

The remainder of the site gently sloped down and away for the perimeter of the bark and mulch facility towards La Grange Road and two ponds located to the north. The ground surface within the project area had a hummocky/disturbed appearance and was generally covered by dense weeds, grass and scattered debris. Wood waste or the remnants of stockpiled bark and mulch was noted on the surface in at least five areas adjacent the pavement for the existing facility. Stormwater drainage from the facility was being routed through shallow swales to two detention ponds, one located in the western portion of the site and the second located in the southern portion. The exposed soil in the swales was wet and soft. The detention ponds appear to have been excavated into existing grade with the soil loosely stockpiled on three sides.

Site access was provided by an asphalt-concrete driveway that intersected La Grange Road in the south portion of the site near the southern detention pond. A guard shack and truck scale were located in the southern portion of the driveway. A well within a pump house, remnant concrete foundations, and an elevated water tank were observed in the southeastern, southwestern, south-central portions of the site, respectively. A second, unused, east-west trending driveway from La Grange Road crossed the northern portion of the site. The asphalt surfacing appeared to be in poor condition with extensive cracking and several segments missing. A concrete building pad surrounded by gravel covered driveways was located north of this second driveway towards the east end. A second elevated water tank and another concrete feature were located east and north of the concrete pad.

<u>Parcel History</u>

A historical records review was performed during preparation of the referenced ESA performed by WKA. The records review revealed that the project site was vacant, mostly grass-covered land from at least 1893 to at least 1959. From at least 1976 to at least 1984, the site was then developed as a lumber mill with at least two structures, a teepee burner and lumber storage areas. The existing mulch and bark facility has been operating onsite since at least 1998.

Historical aerial photographs of the project site and general vicinity were compiled for the ESA by Environmental Data Resources, Inc. (EDR®). Photographs covering the years 1959, 1976, 1984, 1998, 2006, 2009, 2012, and 2016 were available for review (EDR®, 2020d). Table 1 notes the changes on the property and in the vicinity.

Table 1

Year	Scale	Observations
June 1959	1" = 500'	Site: Mostly grass-covered land with some trees. North: Mostly grass-covered land with some trees. East: Mostly grass-covered land with some trees. South: A road is visible followed by mostly grass-covered land. West: A road followed by partially wooded and grass-covered land.
July 1976	1" = 500'	Site: At least two structures are visible. One structure is located on the northern portion and the remainder are on the southwestern portion, with some structures extending onto the adjacent southwestern adjacent property. A feature on the south-central portion, near the majority of the lumber mill buildings appears to be a teepee burner. Ponds are visible on the northeastern portions. Large areas used for lumber storage are visible across the rest of the property. North: No significant changes noted. East: No significant changes noted. South: Additional lumber mill structures are visible adjacent to the southwest portion of the site. West: At least one structure is visible adjacent to the site.
June 1984	1" = 500'	Site: Most of the lumber stored on the property appears to have been removed. North: No significant changes noted. East: No significant changes noted. South: No significant changes noted. West: Two additional structures are visible adjacent to the site.
1998	1" = 500'	Site: The previously noted structures and lumber storage areas have been removed. A new processing area is visible on the central portion. A paved area is visible along the eastern property boundary. North: No significant changes noted. East: No significant changes noted. South: Some of the structures associated with the lumber mill have been removed. West: No significant changes noted.
2005	1" = 500'	No significant changes noted for the site or the vicinity.
2006	1" = 500'	No significant changes noted for the site or the vicinity.
2009	1" = 500'	No significant changes noted for the site or the vicinity.
2012	1" = 500'	No significant changes noted for the site or the vicinity.
2016	1" = 500'	No significant changes noted for the site or the vicinity.

General Site Geology

Tuolumne County is located in the western portion of the foothills of the Sierra Nevada geomorphic province of California. The Sierra Nevada geomorphic province is approximately 640 kilometers in length by approximately 65 to 160 kilometers in width and extends from Lassen Peak in the north to the Tehachapi Mountains in the south (Norris and Webb, 1990). The Sierra Nevada is a west-tilted fault block uplifted along the Sierra Nevada fault system. Topographic relief with the Sierra Nevada province varies from approximately 120 meters (400 feet) on the west to approximately 4,421 meters (14,505 feet) on the east at Mount Whitney.

The local geology has been mapped by various authors. The maps reviewed differ in scale and detail but agree that the site is underlain by metavolcanic rock of the Copper Hill Volcanics (commonly referred to as greenstone) and serpentinized ultramafic rock. Based on previous experience, the upper one to three feet of greenstone tends to be highly weathered and fractured, quickly decreasing in weathering with depth while becoming highly resistant and strong.

The United States Department of Agriculture, Natural Resources Conservation Service website (http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx), maps the project area as being underlain primarily by soils of the Bonanza-Loafercreek complex. These soils are described as low plastic clay residuum weathered from metavolcanics.

Subsurface Soil Conditions

The subsurface soil conditions at the project site were explored on February 17, 2021, by excavating and sampling 12 test pits to depths ranging from about three to eight feet bgs. Practical refusal using a Case 580M backhoe was encountered in moderately weathered bedrock at all the test pit locations. The approximate test pit locations are presented in Figure 2.

These native subsurface conditions encountered appear to be generally consistent with the mapped geology described above. In general, the materials encountered consisted of brown, low plastic, sandy clay and bluish grey, highly plastic, sandy silt residuum that transitioned into reddish-brown, highly to completely weathered bedrock with the consistency of hard sandy silt or dense silty sand with angular gravel and cobble sized bedrock fragments. These materials were followed by moderately weathered, friable to weak bedrock until practical refusal was encountered.

Because of past activities at the site, the upper one to two feet of the residuum at most test pit locations appears to be disturbed. At three test pits (TP1, TP4 and TP5), the native materials were overlain by undocumented fill consisting of gravel or clay with wood fragments to depths ranging from about two to 5½ feet bgs. The pavement encountered in the vicinity of the existing bark and mulch facility consisted of about three to four inches of asphalt concrete overlying eight to nine inches of aggregate base. Only six inches of aggregate base was encountered at one test pit (TP3); however, this test pit was excavated through a large pothole. At two test pits (TP6 and TP9), the pavement was directly underlain by weathered bedrock. The pavement at the two remaining test pits excavated in the area of the existing facility were underlain by residual soils.

The subsurface conditions described above are a generalized interpretation of the conditions encountered. For specific information regarding the soil conditions encountered at each exploration location, refer to the exploration logs presented as Figures 3 through 14.

Groundwater

Static groundwater was not encountered at the time of our field explorations. However, seepage was observed emerging from the sidewalls of several test pits near the transition from the near-surface residual soils to the weathered bedrock.

We reviewed available groundwater information at the California Department of Water Resources website and found that no currently monitored groundwater wells are located near the site. Furthermore, no groundwater reports were found on the State Water Resources Control Board's (SWRCB) GeoTracker website for facilities near the site.

CONCLUSIONS

We believe that the project is feasible from a geotechnical standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations for the project are discussed in the following sections.

Seismic and Geologic Hazards

The evaluation of potential seismic hazards was not within the scope of this study. Based on records currently available from the California Geologic Survey (CGS), the project site is not located within a Fault-Rupture Hazard Zone as delineated by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults are mapped within the project site. Therefore, we do not anticipate the proposed project will need special design or construction requirements to account for faulting.

Due to the absence of near-surface groundwater, the relatively low seismicity of the project area and the relatively shallow depth to bedrock, in our professional opinion the potential for other seismically induced hazards, such as liquefaction and settlement, is considered low. Accordingly, mitigation for these potential hazards should not be necessary for development of this project.

Seismic Design Criteria

The 2019 edition of the *California Building Code* (CBC) references the American Society of Civil Engineers (ASCE) Standard 7-16 for seismic design. Using the latitude and longitude for the approximate center of the project site, Table 2 provides 2019 seismic design parameters developed using a web interface developed by the Structural Engineers Association of California (SEAOC) and the California Office of Statewide Health Planning and Development (OSHPD) (https://seismicmaps.org). Since S_1 is greater than 0.2g, the 2019 *CBC* coefficient values F_v, S_{M1}, and S_{D1} presented are valid for seismic design, provided the requirements in Exception Note No. 2 in Section 11.4.8 of *ASCE 7-16* apply, specifically if $T \leq 1.5T_S$. If not, a site-specific ground motion hazard analysis is required.

Table 2

Latitude: 37.8396° N Longitude: 120.5045° W	ASCE 7-16 Table/Figure	2019 CBC Table/Figure	Factor/ Coefficient	Value
MCE_G Peak Ground Acceleration	Figure 22-7	--	PGA	0.163
Site Amplification Factor	Table 11.8-1	--	F_{PGA}	1.237
Site Modified Peak Ground Acceleration	Equation 11.8-1	--	PGA_M	0.201
Short-Period MCE at 0.2 seconds	Figure 22-1	Figure 1613.2.1(1)	S_S	0.384
1.0 second Period MCE	Figure 22-2	Figure 1613.2.1(2)	S_1	0.192
Soil Class	Table 20.3-1	Section 1613.2.2	Site Class	C
Site Coefficient	Table 11.4-1	Table 1613.2.3(1)	F_a	1.3
Site Coefficient	Table 11.4-2	Table 1613.2.3(2)	F_v	1.5
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16-36	S_{MS}	0.5
	Equation 11.4-2	Equation 16-37	S_{M1}	0.287
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16-38	S_{DS}	0.333
	Equation 11.4-4	Equation 16-39	S_{D1}	0.192

Latitude: 37.8396° N Longitude: 120.5045° W	ASCE 7-16 Table/Figure	2019 CBC Table/Figure	Factor/ Coefficient	Value
Seismic Design Category	Table 11.6-1	Table 1613.2.5(1)	Risk Category I to III	C
	Table 11.6-1	Table 1613.2.5(1)	Risk Category IV	D
	Table 11.6-2	Table 1613.2.5(2)	Risk Category I to III	C
	Table 11.6-2	Table 1613.2.5(2)	Risk Category IV	D

Notes: MCE = Maximum Considered Earthquake; g = gravity

Soil Expansion Potential

Laboratory tests performed on sample of two samples of near surface soils revealed that these materials possessed a low plasticity index when tested in accordance with ASTM D4318 test method (see Figure A1). Based on these test results, the near-surface brown clay and weathered bedrock encountered during our explorations should not exhibit significant expansion (shrink/swell) characteristics. Although highly plastic, the blueish-grey silt encountered at some explorations should also not exhibit significant expansion characteristics. Accordingly, measures to resist or control potential soil expansion pressures are not considered necessary for this project.

Foundation Support

At nine of the twelve test pits excavated for this investigation, the highly to moderately weathered bedrock was overlain by about 2½ to 3½ feet of residual soils consisting of brown, low density clay and blueish-grey silt and/or two to 5½ feet of undocumented clay fill mixed with wood fragments. Both the fill and residuum appeared to be relatively soft/loose and in poor condition overall. Furthermore, most of the exposed surface soils outside of the existing bark and mulch facility appeared to highly disturbed from past operations at the site.

In our professional opinion, the undocumented fill and residuum, where encountered, will not be suitable for support of foundations or other proposed improvements. Instead, foundations should be supported on the underlying weathered bedrock or the overlying fill and residuum should be completely undercut to weathered bedrock and replaced with engineered fill. Foundation design criteria for both subgrade alternatives is presented in the *Recommendations*.

Soil Suitability for Engineered Fill Construction

The brown, sandy clay residuum encountered is considered suitable for use in engineered fill construction provided the material does not contain rubble, rubbish, significant organic concentrations and is at a moisture content appropriate for compaction. The undocumented fill and the native blueish-grey silt encountered was very soft and wet (nearly saturated) and should not be used as engineered fill in structural areas. Imported materials, if necessary, should be granular and approved by our office prior to importing the materials to the site.

Excavation Conditions

Based on our field data and previous experience, it should be possible to excavate the near-surface residual soils, undocumented fill, and the upper few feet of weathered bedrock using conventional earthmoving equipment. Below this material, the bedrock becomes less weathered and more resistant to excavation. Larger earthmoving equipment such as a D9H or D10L Caterpillar tractor fitted with a single tooth ripper may be necessary to complete site grading. Deep cuts may require even heavier equipment such as a Caterpillar D10N or D11N. Heavy tractors or hydraulic shovels with case-hardened steel rippers probably can excavate utility trenches that extend in to this material, although over-widening is inevitable. Pneumatic hammers (hoe-ram) may be required to break-up resistant bedrock areas.

The predicted excavation conditions reported herein are intended for informational purposes only and should not be interpreted to imply that localized resistant bedrock layers, boulders, or outcroppings will not be encountered. The ultimate proof of a materials rippability can only be determined by machine trial during grading. Accordingly, the earthwork contractor should perform his own analysis, prior to grading, to evaluate the rippability of the bedrock and size his equipment for the project. We suggest a unit cost be included in the bid schedule for localized overexcavation and/or blasting to remove resistant materials if excavation(s) are anticipated to extend more than a few feet into the weathered bedrock material.

Temporarily sloped excavations and shored excavations less than 20 feet in depth should be constructed in accordance with federal, local and OSHA standards (29 CFR Part 1926) under the guidance of the Contractors qualified “competent person.” For preliminary evaluation, the silts and clays encountered would classify as Cal-OSHA Type C soil, while the weathered bedrock would classify as Type B soils. In no case should the information provided be interpreted to mean that WKA is assuming responsibility for site safety or the Contractor’s activities.

Excavated materials should not be stockpiled directly adjacent to an open excavation to prevent surcharge loading of the excavation sidewalls. Heavy or frequent truck and equipment traffic should also be avoided near excavations. If material is stored or heavy equipment is stationed and/or operated near an excavation, a shoring system must be designed to resist the additional pressure due to the superimposed loads.

Pavement Subgrade Quality

The results of our laboratory tests indicate the near-surface residual soils should provide fair support characteristics for pavements as represented by Resistance ("R") values (California Test 301) of 30 and 45 (see Figure A2). Furthermore, the weathered bedrock should also provide equal or better support characteristics based on past experience. Given the anticipated grading and mixing of soils and weathered bedrock during earthwork construction, and R-value of 30 was used for pavement evaluations.

Percolation Testing

Two percolation tests were performed at the locations shown in Figure 2 in accordance with test procedures outlined in the United States Environmental Protection Agency (EPA) Design Manual for Onsite Wastewater Treatment and Disposal Systems (1980). A test pit was also excavated to determine the depth of weathered bedrock near the percolation test locations. Resistant weathered bedrock was encountered at a depth of about 3½ feet bgs, overlain by low plastic clay (residual soil). Because of the shallow depth to bedrock, the percolation tests were performed at depths of one and two feet bgs with the results summarized in Table 3 as follows:

Table 3

Test No.	Approximate Depth, ft.	Soil Classification	Percolation Rate, minutes/inch
P1	1	Sandy Clay	150
P2	2	Sandy Clay	300

The purpose of our testing was to evaluate the preliminary on-site septic/leach field potential. Referencing Section 13.08.220 of the Tuolumne County On-Site Sewage Treatment and Disposal Code (Chapter 13.08), “there shall be a minimum of five feet of permeable soil below the bottom of a leach trench or bed” with permeable soil defined as soil with a percolation rate not slower than 120 minutes per inch for standard leach trenches or beds. With the shallow bedrock conditions and slow percolation test results, a conventional absorption trench, bed or pit sewage treatment system will not meet Tuolumne County criteria. Alternative systems, such as a mound system or a system that incorporates pre-treatment prior to evaporation or ground disposal, may be required.

Soil Corrosion Potential

Two samples of near-surface soil were submitted to Sunland Analytical Lab of Rancho Cordova, California, for testing to determine pH, chloride and sulfate concentrations, and minimum resistivity to help evaluate the potential for corrosive attack upon buried concrete. The results of the corrosivity testing are summarized in Table 4. Copies of the test reports are presented in Figures A4 through A7.

Table 4

Analyte	Test Method	Sample Identification	
		TP7 (0'-3')	TP11 (0'-3')
pH	CA DOT 643 Modified*	5.75	5.95
Minimum Resistivity	CA DOT 643 Modified*	1960 Ω-cm	590 Ω-cm
Chloride	CA DOT 422	19.3 ppm	176.6 ppm
Sulfate	CA DOT 417	48.1 ppm	464 ppm
Sulfate – SO4	ASTM D-516	47.7 mg/kg	433.4 mg/kg

Notes: * = Small cell method, Ω-cm = Ohm-centimeters, ppm = Parts per million, mg/kg= milligrams per kilogram

The California Department of Transportation (Caltrans) 2018 Corrosion Guidelines (Version 3.0), considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil sample taken: the soil has a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 2,000 ppm, or the pH is 5.5 or less. Based on this criterion, the on-site soils tested are not considered significantly corrosive to concrete or steel reinforcement properly embedded within Portland cement concrete (PCC).

The California Amendments to Section 10.7.5 of the American Association of State Highway and Transportation Officials (AASHTO) bridge design specifications, 6th Edition (AASHTO 2012) considers soils to be corrosive to buried metals if the minimum resistivity is 1,000 ohm-cm or less. Based on this criterion, the on-site residuum tested may be highly corrosive to buried metal.

Table 19.3.1.1 – Exposure Categories and Classes, of American Concrete Institute (ACI) 318-14, Section 19.3 – Concrete Design and Durability Requirements, as referenced in Section 1904.1 of the 2016 CBC, indicates the severity of sulfate exposure for the sample tested is

Exposure Class S1. The project structural engineer should evaluate the requirements of ACI 318-14 and determine their applicability to the site.

Wallace-Kuhl & Associates are not corrosion engineers. Therefore, if it is desired to further define the soil corrosion potential at the site, a corrosion engineer should be consulted.

Naturally Occurring Asbestos

Asbestos is classified by the EPA as a known human carcinogen and naturally occurring asbestos (NOA) has been identified as a potential health hazard. The California Conservation Service has published a web based map using digitized data from the CGS's Open File Report 2000-19 and other sources to identify ultramafic rock in outcrop layers that may contain asbestos or asbestos-like materials. The project site is located about ½ mile west of the closest mapped ultramafic rock outcrops.

A residual soil and highly weathered bedrock sample obtained from test pit TP12 was tested for NOA. The sample was transported under chain-of-custody documentation to a California-certified laboratory in accordance with California Air Resources Board 435 test method. The result is included in Appendix A as Figure A8. The result indicates that no asbestos was detected in the sample tested.

Groundwater and Seasonal Moisture

A permanent groundwater table was not encountered during our explorations and is expected to be relatively deep with no impact to the development of the site. However, due to the shallow depth and low permeability of the underlying weathered bedrock, it is common for isolated or perched water to be encountered during grading operations. Perched groundwater seepage was observed at test pits TP3, TP7, and TP12.

The presence or absence of perched water can very due to several factors such as, the proximity to bedrock, topographic relief across the site, and the proximity to surface water. Based on our previous experience, areas of perched water above the weathered bedrock should be anticipated and could vary through the year with higher concentrations during or following precipitation. Furthermore, if site grading is performed during or following extended periods of rainfall (winter and spring months), the moisture content of the near-surface soils may be significantly above optimum and unstable.

Typical remedial measures include discing and aerating the soils during dry weather, mixing the soils with dryer materials, removing and replacing the soils with an approved fill material, stabilization with a geotextile fabric or grid, or mixing the soils with an approved hydrating agent such as a lime or cement product. Our firm should be consulted prior to implementing any remedial measure to observe the unstable subgrade condition and provide site-specific recommendations.

RECOMMENDATIONS

General

The recommendations presented below are appropriate for typical construction in the late spring through fall months. The on-site soils typically become very moist and wet following rainfall in the winter and early spring months, and often are not be suitable for earthwork without drying by aeration, chemical treatment, or geogrid stabilization. Should the construction schedule require work to start or continue during the wet months, additional recommendations can be provided, as conditions warrant.

A representative of the Geotechnical Engineer should be present during all earthwork and ground improvement construction operations to evaluate compliance with the recommendations presented in this report and the project plans and specifications. The Geotechnical Engineer of Record referenced herein should be considered the Geotechnical Engineer that is retained to provide geotechnical engineering observation and testing services during construction.

Site Clearing

Prior to site grading, construction areas should be cleared of any existing surface and subsurface structures to expose firm and stable soils, as determined by the Geotechnical Engineer's representative. Construction areas to be cleared should extend at least five feet beyond the edge of all exterior foundations and at least five feet beyond any exterior flatwork or pavements, where practical. Demolition debris should be removed from the site, or used as engineered fill, provided it is processed per the recommendations included in this report.

Any existing underground utilities designated to be removed or relocated should include all trench backfill and bedding materials. The resulting excavations should be restored with engineered fill placed and compacted in accordance with the recommendations included in this report. On-site wells, septic systems, or below-grade tanks should be properly abandoned in accordance with State and local requirements.

We understand the existing asphalt-concrete pavement in the area of the vacant bark and mulch facility will be completely demolished. The existing pavements designated for removal may be broken up, pulverized and reused as engineered fill or removed from the site. If pavement rubble is to be reused as engineered fill, the material should be pulverized to fragments less than three inches in largest dimension, mixed with soil to form a compactable mixture, and must be approved by the owner.

Existing surface vegetation/organics and organically laden soil within construction areas should be stripped from the site. Debris from the stripping should not be used as general fill within structure, concrete slab or pavement areas. With prior approval from the Geotechnical Engineer, strippings may be used in proposed non-structural areas, provided they are kept at least five feet from building footprints, pavements, concrete slabs and other surface improvements.

Any trees, bushes and other vegetation designated for removal should include the entire root-ball and roots larger than ½ inch in diameter. Adequate removal of debris and roots may require laborers and handpicking to clear the subgrade soils to the satisfaction of the Geotechnical Engineer's representative.

Any on-site ditches, swales, or detention ponds should be fully drained of water and cleaned of organics. Saturated and unstable soils exposed should be removed to expose firm, native materials, as determined by our representative. These soils will likely be saturated and will require aeration and a period of drying to allow proper compaction. Organically contaminated soils will not be suitable for use in engineered fill construction.

Depressions resulting from site clearing operations, as well as any loose, soft, disturbed, wet, or organically contaminated soils, as identified by the Geotechnical Engineer's representative, should be cleaned out to firm, undisturbed soils and backfilled with engineered fill placed and compacted in accordance with the recommendations in this report. It is important that the Geotechnical Engineer's representative be present during site clearing operations to verify adequate removal of the surface and subsurface items, as well as the proper backfilling of resulting excavations.

Subgrade Preparation

In general, the residual soils and undocumented fill overlying the weathered bedrock is loose, low density material that is highly disturbed in areas and therefore not suitable in its current condition for support of foundations, concrete slabs or pavements. Undocumented fill contaminated with wood fragments or other deleterious materials should be removed during site clearing and disposed in non-structural areas. The native residual soils and uncontaminated fill should be overexcavated to weathered bedrock, moisture conditioned or dried as necessary, and compacted as engineered fill. The bluish grey residual soil encountered at several test pits consists of highly plastic, wet silt that may be difficult to compact. This material should either be disposed away from structural areas or be thoroughly mixed with suitable fill prior to compaction. The zone of overexcavation and compaction should extend at least five feet beyond any structural foundations or concrete slabs. In proposed exterior slab-on-grade and pavement areas, the lateral zone of overexcavation and compaction can be reduce to two feet beyond the proposed improvements.

We anticipate that some foundations or structures could be constructed over transitions between compacted fill and undisturbed weathered bedrock. Because of the different physical properties and thus support characteristics of these two materials, there is a possibility that unpredictable and sometimes adverse differential settlement could occur if the differential thickness of fill exceeds five feet. In these situations, the weathered bedrock should be undercut and compacted as engineered fill to maintain a maximum differential fill thickness of five feet.

All completely weathered bedrock areas that will support concrete slabs, engineered fill or pavement, should be thoroughly scarified to a depth of at least 12 inches, brought to a uniform moisture content above the optimum moisture content, and compacted to not less than 90 percent of the maximum dry density per ASTM D1557 specifications. In pavement areas, the relative compaction of the upper six inches of final soil subgrade should be increased to 95 percent of the maximum dry density. Where moderately to unweathered (rocky) bedrock is exposed, no scarification should be necessary; however, these surfaces should be moisture-conditioned and uniformly compacted to an unyielding condition with at least five passes of a heavy, self-propelled sheepsfoot compactor. Any localized zones of soft or pumping materials observed should be scarified and compacted or be overexcavated and replaced with engineered fill.

The performance of pavement is critically dependent upon uniform and adequate compaction of the soil subgrade, as well as all engineered fill and utility trench backfill within the limits of the pavements. Final pavement subgrade preparation (i.e. scarification, moisture conditioning and compaction) should be performed after underground utility construction is completed and just prior to aggregate base placement.

The pavement subgrades should be stable and unyielding under heavy wheel loads of construction equipment. To help identify unstable subgrades within the pavement limits, a proof-roll should be performed with a fully-loaded, water truck on the exposed subgrades prior to placement of aggregate base. The proof-roll should be observed by the Geotechnical Engineer’s representative.

The prepared subgrade soils should be protected from disturbance until covered by capillary break material or aggregate base. Disturbed subgrade soils may require additional processing and recompaction just prior to construction of these improvements, depending on the level of disturbance.

Compaction of the existing grade must be performed in the presence of the Geotechnical Engineer's representative who will evaluate the performance of the subgrade under compaction loads and identify any loose or unstable soil conditions that could require remediation. Construction bid documents should contain a unit price (price per cubic yard) for additional excavation due to unsuitable materials and replacement with engineered fill.

Engineered Fill

From a geotechnical standpoint, the on-site residual soils and existing on-site fill material are considered suitable for use as engineered fill provided that they do not contain significant quantities of organics, rubble and deleterious debris, and are at a proper moisture content to achieve the desired degree of compaction. The bluish grey residual soil encountered at several test pits consists of highly plastic, wet silt that will be difficult to compact. This material should either be disposed away from structural areas or be thoroughly mixed with suitable fill prior to compaction.

Weathered bedrock, boulders, or approved inert debris, i.e., concrete or asphalt-concrete pavement, that breaks into fragments less than six inches in maximum dimension can be used in engineered fills within the upper three and five feet of subgrade beneath proposed floor/pavement subgrade and building foundations, respectively. The weathered rock and debris fragments should be thoroughly mixed with soil to avoid concentrating or nesting the material. The fragments should not make up more than 30 percent of the fill volume. Weathered rock or debris fragments ranging from 6- to 18-inches in maximum dimension may be placed below these depths provided they are also thoroughly mixed with soil. If the rock and/or debris does not break down to a gradation compatible with in-place density testing, then compactive effort should be applied until there is no perceptible increase in fragmentation of the particles or observable consolidation of the fill during repeated passes of heavy compaction equipment.

In pavement areas, weathered rock or debris fragments greater than 18 inches in maximum size may be included in engineered fills below a depth of five feet, but only at the foundation level for the fill. The boulders or fragments should be staggered and selectively spaced so that soil or crushed rock fill can be machine placed and compacted between them to form an interstitial fill. As an alternative, flooding and jetting can be used to sluice cohesionless soil, i.e., sand, into voids between the boulders and fragments. Following sluicing, this fill course should be proofrolled with heavy track equipment until there is no observable consolidation of the fill beneath the equipment. Our representative should witness all filling and proof-rolling operations to determine the adequacy of each course. Fragments greater than 36 inches in maximum size should not be included in any fill.

Engineered fill consisting of on-site residual soils, highly to completely weathered bedrock, existing on-site fill material, or import materials should be placed in lifts not exceeding six inches in compacted thickness, with each lift being thoroughly moisture conditioned to at least the optimum moisture content and uniformly compacted to at least 90 percent relative compaction. The upper six inches of engineered fill placed in pavement areas should be uniformly compacted to at least 95 percent relative compaction at a moisture content of at least the optimum moisture content.

Imported fill materials should be compactable, well-graded, granular soils with a Plasticity Index not exceeding 15 when tested in accordance with ASTM D4318; an Expansion Index of 20 or less when tested in accordance with ASTM D4829; and, should not contain particles greater than three inches in maximum dimension. Imported fill material to be used within pavement areas should possess a Resistance value of 40 or higher, when tested in accordance with California Test 301. In addition, with the exception of imported aggregate base and bedding/initial fill materials for underground utility construction, the contractor should provide appropriate documentation for all imported fill materials that designates the import materials do not contain known contaminants per Department of Toxic Substances Control's guidelines for clean imported fill material (DTSC, 2001), and have corrosion characteristics within acceptable limits. Imported soils should be approved by the Geotechnical Engineer prior to being transported to the site.

The Geotechnical Engineer's representative be present on a regular basis during all earthwork operations to observe and test the engineered fill and to verify compliance with the recommendations of this report and the project plans and specifications.

<u>Cut and Fill Slopes</u>

We anticipate that slopes ranging from about 5 to 10 feet in vertical height may be planned. In our professional opinion, permanent cut and fill slopes should be inclined no steeper than two horizontal feet to one vertical foot (2H:1V). This slope recommendation is based on our experience with similar conditions since no detailed slope stability analysis was performed to justify steeper slopes. Given this inclination, however, there is a modest risk that displacement and/or movement could occur in the event of strong seismic ground shaking. For the native soils, highly weathered bedrock and compacted fill conditions anticipated, we expect this movement to be relatively shallow, requiring limited cleanup and dressing to restore the slopes to their original condition. If this risk is unacceptable, the slopes should be flattened to three horizontal feet to one vertical foot (3H:1V).

Where fills will be constructed on ground that slopes at an inclination of six horizontal feet to one vertical foot (6H:1V) or steeper, a two-foot-deep toe key should be excavated into firm, competent soil/weathered rock. The keyway should be at least four-feet-wide at the bottom or a width equal to ½ the vertical slope height, whichever is greater, with the bottom inclined down and back into the slope at two percent. As filling progresses, benches should also be cut into firm, competent soil/weathered rock. Each bench should consist of a level terrace at least four feet wide with the rise to the next bench held to three feet or less.

It is difficult to construct fill on the specified slopes without leaving a loose, poorly-compacted soil zone on the slope face. To reduce sloughing and erosion, the fill slopes should be slightly over-built, then cut back to firm, well-compacted soils prior to applying vegetative cover. If slopes cannot be over-built and cut back, the finished soil slopes should be compacted to reduce, as much as practical, the thickness of the loose surficial veneer. The compaction may be done by making several coverages from top to bottom of the slopes with a track-mounted bulldozer, front-end loader, or sheeps foot compactor.

Paved interceptor drains should be provided along the tops of slopes where the tributary area flowing toward the slope has a drainage path greater than 40 feet, measured horizontally. The interceptor drains should be sloped to a suitable drainage device and disposed off-site well below the toe of the slope. Drop inlets and drainage pipes should not be installed near the crests of slopes because leakage can result in maintenance problems or possible slope failure. The slopes should be inspected periodically for erosion, and if detected, repaired immediately. Interceptor drains should be cleaned before the start of each rainy season, and if necessary, after each rainstorm. To reduce erosion and gulling, all disturbed areas should be planted with erosion-resistant vegetation suited to the area. As an alternative, jute netting or geotextile erosion control mats can be installed per the manufacturer's recommendations.

Utility Trench Backfill

Utility trench backfill should be mechanically compacted as engineered fill in accordance with the following recommendations. Bedding of utilities and initial backfill around and over the pipe should conform to the manufacturer's recommendations for the pipe materials selected and applicable sections of the governing agency standards. If open-graded, crushed rock is used as bedding or initial backfill, an approved geotextile filter fabric should be used to separate the crushed rock from finer-grained soils. The intent of geotextile filter fabric is to prevent soil from migrating into the crushed rock (piping), which could result in trench settlement.

The on-site residual soil (in lieu of select sand/gravel/crushed rock backfill) should be used as backfill for utility trenches located within the building footprints and extending at least five feet horizontally beyond perimeter foundations to reduce water transmission beneath the buildings. Utility trench backfill should be placed in thin lifts, thoroughly moisture conditioned to at least the optimum moisture content, and mechanically compacted to at least 90 percent relative compaction. The lift thickness will be dependent of the type of compaction equipment used. Within the upper six inches of pavement subgrade soils, compaction should be increased to at least 95 percent relative compaction at no less than the optimum moisture content.

Underground utility trenches that are aligned nearly parallel with shallow foundations should be at least three feet from the outer edge of foundations, wherever possible. As a general rule, trenches should not encroach into the zone extending outward at one horizontal foot to one vertical foot (1H:1V) inclination below the bottom of shallow foundations. Additionally, trenches parallel to shallow foundations should not remain open longer than 72 hours. The intent of these recommendations is to prevent loss of both lateral and vertical support of shallow foundations, resulting in possible settlement.

Foundation Design

Structures and Equipment

The proposed structures and equipment may be supported upon continuous and/or isolated spread foundations bearing on engineered fill or on weathered bedrock. To decrease the potential for differential settlement, the foundations should not bear on a combination of fill and bedrock. Additional overexcavation of existing grades may be required to provide a differential fill depth less than five feet across structures, or foundations can be deepened so that all foundations bear on weathered bedrock.

Foundations bearing on weathered bedrock or engineered fill should extend at least 12 and 18 inches below lowest adjacent soil grade, respectively. Lowest adjacent soil grade is defined as the grade upon which the capillary break material is placed or exterior soil grade, whichever is lower. Continuous foundations should maintain a minimum width of 12 inches; while isolated spread foundations should be at least 24 inches in plan dimension. Foundations or thickened slab edges should be continuous around the perimeter of the building to reduce moisture intrusion beneath the structures.

Foundations bearing on engineered fill may be sized for maximum allowable “net” soil bearing pressure of 3,500 pounds per square foot (psf) for dead plus live load. Foundations bearing on sound weathered bedrock may be sized for a maximum allowable “net” soil bearing pressure of 6,000 psf for dead plus live load. A one-third increase in the allowable bearing pressures may be applied when considering short-term loading due to wind or seismic forces. The weight of the foundation concrete extending below lowest adjacent soil grade may be disregarded in sizing computations.

Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Based on the foundation criteria discussed above and the anticipated foundation and equipment loads, foundations are anticipated to experience a maximum total <u>static</u> settlement on the order of about ¾ inch or less, and differential settlement on the order of about ½ inch for 50 lineal feet or the shortest distance of the structure, whichever is less. Post-construction settlement due to dynamic equipment loads should be limited to ¼ inch or less.

All foundations should be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The structural engineer should determine final foundation reinforcing requirements.

Resistance to lateral foundation displacement may be computed using an allowable friction factor of 0.40, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure equivalent to a fluid pressure of 300 psf per foot of depth, acting against the vertical projection of the foundation. These two modes of resistance should not be added together unless the frictional component is reduced by 50 percent since full mobilization of the passive resistance requires some horizontal movement, effectively reducing the frictional resistance. We recommend that all foundation excavations be observed by the Geotechnical Engineer's representative prior to placement of reinforcement and concrete to verify firm bearing materials are exposed.

Silos

We understand the proposed 60- to 80-foot-diameter silos will be supported on a leg/skirt system or on a rigid concrete slab foundation. Spread foundations supporting a leg/skirt system may be designed using the criteria presented above in the <u>Structures and Equipment</u> section.

If the proposed silos are supported by individual rigid structural slab or mat foundations, an allowable "net' bearing pressure of 4,000 psf for dead plus live loading may be used for design. A one-third increase in the allowable bearing pressure may be applied when considering short-term loading due to wind or seismic forces. Following placement of the silos, the estimated foundation settlement should be nominal. Once filled, the total settlement will vary according to the final height, diameter and spacing between the silos. Assuming the proposed silos are spaced adequately apart to prevent overlapping stress influences, total settlement at the center of the silo should be limited to 1 inch or less. The maximum differential foundation settlement should be on the order of about ½- to ¾-inch between the edge and center of the silo. If the silos are closely spaced, the estimated settlement may increase and should be evaluated once the silo layout and details are known.

If the proposed structural slab foundations are designed based on approximate flexible methods, a k-value or soil coefficient of subgrade reaction at the center of the foundation may be estimated by dividing the foundation contact or bearing pressure (dead-plus-live) by the estimated settlement (in units of feet). The k-value at the outside edge of foundations can be reasonably assumed to be double the k-value determined at the center of the foundation. A Young's modulus (E_s) of 5,000 kips per square foot (ksf) and a Poisson's ratio (μ) of 0.35 may be used to determine the subgrade reaction modulus for the unsaturated subgrade soil conditions encountered at the site. The Young's Moduli was determined based on empirical correlations (Mitchell & Gardner, 1975, and Duncan & Buchignani, 1976) and the Poisson's ratio was selected from published values. If the project would benefit from greater design values or if the structural slab foundations are critically sensitive to loading and deflection, field and/or laboratory tests should be performed to better define the subgrade parameters.

Structural slab foundations should be underlain by at least six inches of Class 2 aggregate base uniformly compacted to at least 95 percent of the maximum dry density in accordance with ASTM D1557 at the optimum moisture content.

The proposed silos should be initially loaded in ⅓ increments, allowing for the initial settlement to occur between the incremental loads. A monitoring program should be established to evaluate the total and differential settlement during the initial loading. The monitoring program should include at least six survey points for each proposed tank established along the perimeter of the tanks. The Geotechnical Engineer should evaluate the results after each survey reading.

If pipes or other conduits entering or leaving the tanks are sensitive to the anticipated total or differential movement, flexible connections should be used until the tanks have been filled and settlement is complete. Thereafter, rigid connections may be installed.

<u>Uplift Resistance</u>

We anticipate that additional uplift resistance may be necessary to prevent overturning of the proposed stackers, log cranes, and the like. Uplift resistance may be provided by the weight of the structure, the weight of soil directly above the foundation, and the weight of soil within an envelope defined by a three-quarters horizontal feet to one vertical foot (¾H:1V) projection up and away from the perimeter of the foundation. The upper two feet of soil should be neglected in determining uplift capacity. All foundation backfill should be compacted to at least 90 percent

compaction in accordance with ASTM D1557. A soil unit weight of 120 pounds per cubic foot may be assumed for compacted backfill, native residual soil, and weathered bedrock.
If foundations are supported on firm, competent bedrock, rock tiedown anchors (such as grouted dowels) can be used to provide additional uplift resistance. There are several approaches and anchor products available that would be suitable for this project. If dowels are used, a common approach would be to drill two- to four-inch-diameter holes using air percussion to a depth of 10 to 20 feet; blowing out the hole to remove as much rock dust as possible; filling the hole with a non-shrink grout (such as Embeco 636) or an approved high strength epoxy; and then installing the dowel (such as a No. 8, grade 60 reinforcing bar).

The uplift capacity of the anchor is typically assumed to be equivalent to the effective weight of bedrock within a cone or wedge defined by a one horizontal foot to one vertical (1H:1V) projection up from the outside edge and mid-depth of the grouted dowel. A bedrock effective unit weight of 130 pounds per cubic foot and a minimum factor of safety of 2.0 may be used for estimating uplift. For anchors with overlapping cones, the effective weight of bedrock within the overall area of the overlapping cones should be used for determining uplift. The overlapping of the zones of influence between adjacent anchors results in anchor uplift capacity less than that for a single anchor.

The actual anchor design and approach should be determined by the Contractor in coordination with the Structural Engineer. Additional rock cores or geophysical testing may be required to determine the final depth of the anchors and the design criteria. An uplift load test should be performed on at least 10 percent of the completed anchors to verify the design capacity. The Geotechnical Engineer should review the final anchor design and a representative should observe the load test and anchor installation.

Interior Floor Slabs

Interior concrete slab-on-grade floors should be supported by the soil subgrade prepared in accordance with the recommendations contained in the Subgrade Preparation and Engineered Fill sections.

The interior concrete slabs should be at least four-inches-thick; however, the project structural or civil engineer should determine final floor slab thickness, reinforcement and joint spacing. Temporary loads exerted during construction from vehicle traffic, cranes, forklifts, other construction equipment, storage of palletized construction materials, etc. should be considered in the design of the thickness and reinforcement of the interior concrete slabs-on-grade.

Moisture Penetration Resistance

It is likely that floor slab subgrade soils will become very moist or wet at some time during the life of the structures. This is a certainty when slabs are constructed during the wet season or when constantly wet ground or poor drainage conditions exist adjacent to structures. For this reason, it should be assumed that interior slabs with moisture-sensitive floor coverings or coatings will require protection against moisture or moisture vapor penetration through the slabs.

Interior floor slabs for the planned buildings should, as a minimum, be underlain by a layer of free-draining crushed rock/gravel, serving as a deterrent to migration of capillary moisture. The crushed rock/gravel layer should be between four- and six-inches-thick and graded such that 100 percent passes a one-inch sieve and less than five percent passes a No. 4 sieve. Additional moisture protection may be provided by placing a vapor retarder membrane (at least 10-mils thick) directly over the crushed rock/gravel. The water vapor retarder membrane should meet or exceed the minimum specifications as outlined in ASTM E1745 and be installed in strict conformance with the manufacturer's recommendations. For portions of the interior floor slabs that are designated to support vehicular traffic, we recommend placing the vapor retarder membrane directly over compacted aggregate base.

Floor slab construction practice over the past 30 years or more has included placement of a thin layer of dry sand or pea gravel over the vapor retarder membrane. The intent of the sand/pea gravel is to aid in the proper curing of the slab concrete. However, during the wet seasons moisture can become trapped in the sand or pea gravel, which can lead to excessive moisture vapor emissions from floor slabs. As a consequence, we consider use of the sand/pea gravel layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

It is emphasized that the crushed rock/grave and the vapor retarder membrane suggested above provides only a limited, first line of defense against soil-related moisture issues and will not "moisture proof" the slab. Nor do these measures provide an assurance that slab moisture transmission levels will tolerable levels to prevent damage to floor coverings or other building components. If increased protection against moisture vapor penetration is desired, a concrete moisture protection specialist should be consulted. The design team should consider all available measures for slab moisture protection. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slabs.

Exterior Flatwork

The final subgrade for exterior concrete flatwork (i.e., sidewalks, patios, etc.) should be prepared and constructed in accordance with recommendation provided in the Subgrade Preparation and Engineered Fill sections. Exterior flatwork should be underlain by at least four inches of aggregate base compacted to at least 95 percent relative compaction to provide stability during slab construction and to protect the soils from disturbance during construction.

Exterior flatwork concrete should be at least four inches thick. Consideration should be given to thickening the edges of the slabs at least twice the slab thickness where wheel traffic is expected over the slabs. Expansion joints should be provided to allow for minor vertical movement of the flatwork. Exterior flatwork should be constructed independent of other structural elements by the placement of a layer of felt material between the flatwork and the structural element. The slab designer should determine the final thickness, strength and joint spacing of exterior slab-on-grade concrete. The slab designer should also determine if slab reinforcement for crack control is required and determine final slab reinforcing requirements.

Practices recommended by the Portland Cement Association (PCA) for proper placement, curing, joint depth and spacing, construction, and placement of concrete should be followed during exterior concrete flatwork construction.

Pavement Design

The subgrade soils and weathered bedrock in pavement areas should be prepared in accordance with the recommendations contained in the *Subgrade Preparation* and *Engineered Fill* sections.

Based on laboratory testing, an R-value of 30 was used for design of pavements supported on the near-surface materials encountered. The pavement sections presented in Table 5 have been calculated using traffic indices assumed to be appropriate for the project. The procedures used for pavement design are in general conformance with Chapters 600 to 670 of the *California Highway Design Manual* (Caltrans, 2019). The project civil engineer should determine the appropriate traffic index and pavement section based on anticipated traffic conditions. If needed, we can provide alternative pavement sections for different traffic indices.

Table 5

Traffic Index (TI)	Typical Pavement Use	Maximum Equivalent 18-kip Axle Load (EAL)	Number of Loaded 5-Axle Trucks per Week (20-year Design)	Untreated Subgrade R-value = 30		
				Type A Asphalt Concrete (inch)	Class 2 Aggregate Base (inch)	Portland Cement Concrete (inch)
4.5	Automobile Parking Only	4,710	1	2½	6	--
				3*	5	--
				--	4	4
5.0	Automobile Drive Lanes	10,900	3	2½	7	--
				3*	6	--
				--	4	4
6.0	Automobile, Light Delivery Truck and Fire Lanes	47,300	13	2½	10	--
				3½*	8	--
				--	4	4
7.0	Trash Enclosures and Moderate Delivery Truck Traffic	164,000	47	3	12	--
				4*	10	--
				--	4	5
8.0	Heavy Truck Traffic Lanes	487,000	140	3½	14	--
				5*	11	--
				--	6	6

Note: * = *Asphalt concrete thickness contains the Caltrans safety factor.*

In the summer heat, high axle loads coupled with shear stresses induced by sharply turning tire movements can lead to failure in asphalt concrete pavements. Therefore, consideration should be given to using Portland cement concrete (PCC) pavements in areas subjected to concentrated heavy wheel loading, such as entry driveways, in front of trash enclosures, and/or in storage/unloading areas. Alternate PCC pavement sections have been provided above in Table 5.

Concrete slabs be constructed with thickened edges in accordance with American Concrete Institute design standards (ACI, 2016), latest edition. Reinforcing for crack control, if desired, should be provided in accordance with ACI guidelines. Reinforcement must be located at mid-slab depth to be effective. Joint spacing and details should conform to the current PCA or ACI guidelines. PCC should achieve a minimum compressive strength of 3,500 pounds per square inch at 28 days.

All pavement materials and construction methods of structural pavement sections should conform to the applicable provisions of the *Caltrans Standard Specifications*, latest edition.

Efficient drainage of all surface water to avoid infiltration and saturation of the supporting aggregate base and subgrade soils is important to pavement performance. Weep holes could be provided at drainage inlets, located at the subgrade-aggregate base interface, to allow accumulated water to drain from beneath the pavements.

<u>Retaining Walls</u>

All retaining walls or below grade walls for the buildings should be designed to resist the lateral soil pressures of the retained soils. Retaining walls that are fixed/restrained at the top should be capable of resisting an “at-rest” lateral soil pressure equal to an equivalent fluid pressure of 60 psf per foot of the wall height (fully drained conditions). Retaining walls that will be allowed to slightly rotate about their base (unrestrained at the top or sides) should be capable of resisting an "active" lateral soil pressure equal to an equivalent fluid pressure of 40 psf per foot of wall height (fully drained conditions).

For retaining walls with backfill sloped at a gradient of up to 2H:1V, add 20 and 15 psf per foot of the wall height to the at-rest or active equivalent fluid pressures provided above, respectively. Based on recent research (Lew, et al. 2010), the seismic increment of earth pressure may be neglected if the maximum peak ground acceleration at the site is 0.4g or less. Our analysis indicates the maximum peak ground acceleration at the site will be about 0.2g; therefore, the seismic increment of lateral earth pressure may be neglected and retaining walls may be designed using the lateral earth pressures presented above.

If structural elements, i.e., foundations, roadways, etc., encroach the 1H:1V projection from the bottom of retaining walls, the retaining walls should account for surcharge loads resulting from those structural elements. Additionally, any below-grade retaining walls should also account for surcharge loads resulting from construction equipment, vehicles, palletized materials, etc. that encroach the 1h:1v projection from the bottom of the below-grade retaining walls. Surcharge loading under the circumstances described above should be evaluated by the retaining wall designer on a case-by-case basis and be included in their design of the walls. The retaining wall designer should evaluate the surcharge load distribution, magnitude of the surcharge resultant force to be applied on the walls, and the location of where the resultant force should be applied on the walls. Surcharge loading on the retaining walls will depend on the specific surcharge load type (e.g. point load, distributed load, etc.) and distance away from the retaining walls.

Retaining wall or below grade walls should be fully drained to prevent the build-up of hydrostatic pressures behind the wall. Retaining walls should be provided with a drainage blanket of Class 2 permeable material, Caltrans Standard Specification, Section 68-2.02F(3), at least one foot wide extending from the base of wall to within one foot of the top of the wall. The top foot above the drainage layer should consist of compacted on-site or imported engineered fill materials, unless covered by a concrete slab or pavement. Weep holes or perforated rigid pipe, as appropriate, should be provided at the base of the wall to collect accumulated water. Drainpipes, if used, should slope to discharge at no less than a one percent fall to suitable drainage facilities. Open-graded ½- to ¾-inch crushed rock may be used in lieu of the Class 2 permeable material provided the rock and drain pipe are completely enveloped in an approved non-woven, geotextile filter fabric. Alternatively, approved geotextile drainage composites, such as MiraDRAIN®, may be used in lieu of the drain rock layer. If used, geocomposite drain panels should be installed in accordance with the manufacturer's recommendations.

If efflorescence (discoloration of the wall face) or moisture/water penetration of the retaining walls is not acceptable, moisture/water-proofing measures should be applied to the back face of the walls. A moisture/water-proofing specialist should be consulted to determine specific protection measures against moisture/water penetration through the walls.

Structural backfill materials for retaining walls within a one horizontal to one vertical (1H:1V) projection from the bottom of the walls (other than the drainage layer) should consist of on-site or imported, compactable granular material that does not contain significant quantities of rubbish, rubble, organics and rock over four inches in size. Clay, pea gravel and/or crushed rock should not be used for structural wall backfill. Structural wall backfill should be placed in lifts not exceeding 12 inches in compacted thickness, moisture conditioned to at least the optimum moisture content, and should be mechanically compacted to at least 90 percent relative compaction.

Foundations for support of retaining or below grade walls should be designed using the appropriate foundation design parameters provided in the *Spread Foundations* section included in this report.

<u>Railroad Siding and Spurs</u>

We understand that a railroad siding and several spurs will be constructed from the Sierra Northern Railway tracks located along the west perimeter of the project site. It does not appear that Sierra Northern Railway has specific track design and construction criteria; therefore we've assume that the siding and spurs would need to meet American Railway Engineering and Maintenance-of-Way Association (AREMA) design criteria and specifications.

The subgrade soils and weathered bedrock in pavement areas should be prepared in accordance with the recommendations contained in the Subgrade Preparation and Engineered Fill sections.

The proposed tracks should be underlain by at least 12 inches of No. 5 (3/8- to 1-inch) ballast that is vibrated into place to provide a dense base. The ballast should conform to AREMA specifications and meet ASTM criteria for durability and strength. The ballast should be underlain by at least six inches of Class 2 aggregate base (subballast) compacted to at least 95 percent relative compaction in accordance with ASTM D1557. A representative of the Geotechnical Engineer should be on-site to document the compaction of the ballast and subballast.

The track substructure profile, side slopes and drainage should meet the minimum criteria presented in the AREMA Manual for Railway Engineering.

Site Drainage

Final site grading should be accomplished to provide positive drainage of surface water away from the buildings and prevent ponding of water adjacent to foundations, slabs or pavements. The subgrade adjacent to the buildings should be sloped away from the building at a minimum two percent gradient for at least five feet, where possible. All roof drains should be connected to non-perforated rigid pipes, which in-turn are connected to available drainage features that convey water away from the buildings and foundations or discharging the drainage onto paved or hard surfaces that slope away from the buildings. Landscape berms, if planned, should not be constructed in such a manner as to promote drainage toward the buildings.

Drought Considerations

The State of California can experience extended periods of severe drought conditions. The ability for landowners to use irrigation as a means for maintaining landscape vegetation and soil moisture can be inhibited for unpredictable periods of time. For this reason, landscape and hardscape systems for this development should be carefully planned to prevent the desiccation of soils under and near foundations and slabs. Trees with invasive shallow root systems should be avoided. No trees or large shrubs that could remove soil moisture during dry periods should be planted within five feet of any foundation or slab. Fallow ground adjacent to foundations must be avoided.

Geotechnical Engineering Construction Observation Services

Wallace-Kuhl & Associates be retained to review the final plans and specifications to verify that the intent of our recommendations has been implemented in those documents.

Site preparation should be accomplished in accordance with the recommendations of this report. Geotechnical testing and observation during construction is considered a continuation of our geotechnical engineering investigation. Wallace-Kuhl & Associates should be retained to provide testing and observation services during site clearing, preparation, earthwork, and foundation construction at the project site to verify compliance with this geotechnical report and the project plans and specifications, and to provide consultation as required during construction. These services are beyond the scope of work authorized for this study; however, we can submit a proposal to provide these services upon request.

In the event that Wallace-Kuhl & Associates is not retained to provide geotechnical engineering observation and testing services during construction, the Geotechnical Engineer retained to provide these services should indicate in writing that they agree with the recommendations of this report, or prepare supplemental recommendations as necessary. A final report by the Geotechnical Engineer providing construction testing services should be prepared upon completion of the project.

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed project, combined with our analysis of site conditions revealed by the previous field explorations and associated laboratory testing programs. We have used engineering judgment based upon the information provided and the data generated from our study. This report has been prepared in substantial compliance with generally accepted geotechnical engineering practices that exist in the area of the project at the time the report was prepared. No warranty, either express or implied, is provided.

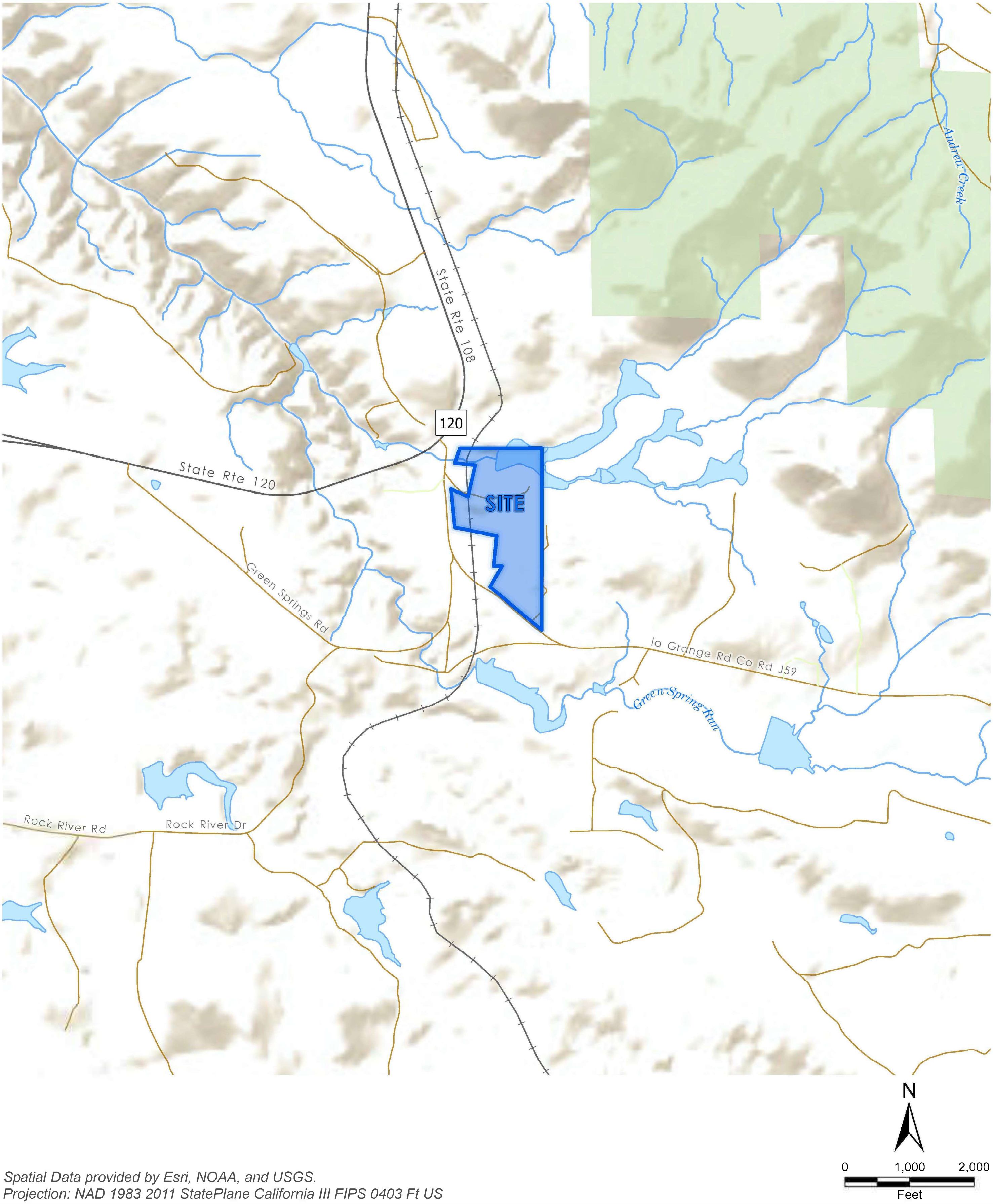
If the proposed construction is modified or re-sited; or, if it is found during construction that subsurface conditions differ from those we encountered at the exploration locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.

We emphasize that this report is applicable only to the proposed construction and the investigated site and should not be utilized for construction on any other site. The conclusions and recommendations of this report are considered valid for a period of two years. If design is not completed and construction has not started within two years of the date of this report, the report must be reviewed and updated, if necessary.

Wallace - Kuhl & Associates

Michael M. Watari
Principal Engineer

GHG:MMW:/mmw



Spatial Data provided by Esri, NOAA, and USGS.
Projection: NAD 1983 2011 StatePlane California III FIPS 0403 Ft US

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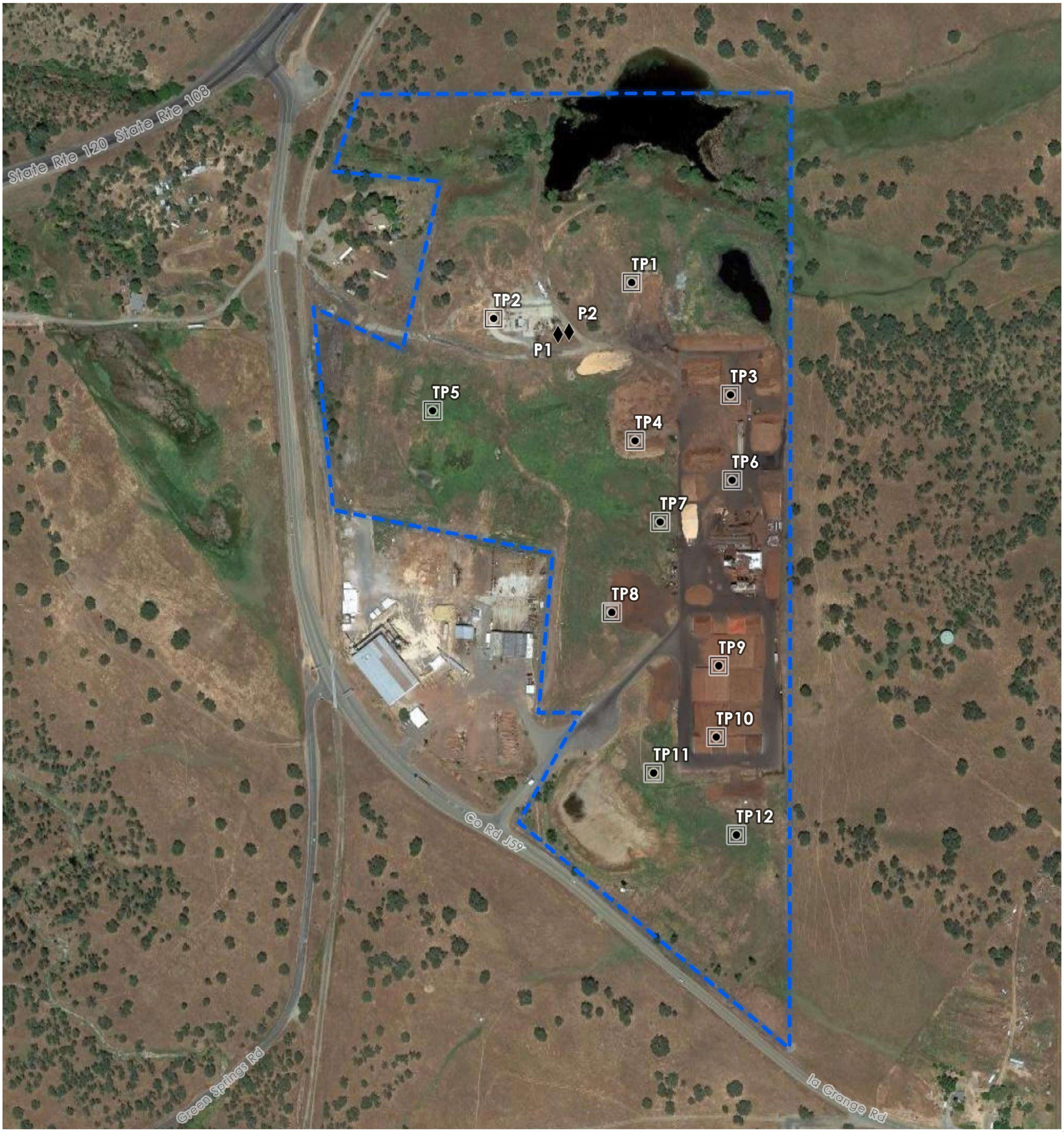
VICINITY MAP

PELLET PROCESSING FACILITY

Jamestown, California

FIGURE	1
DRAWN BY	RWO
CHECKED BY	GHZ
PROJECT MGR	MMW
DATE	06/2021
WKA NO.	12774.03P



Approximate Test Pit Location

Approximate Percolation Test Location

Approximate Site Boundary

N

0 200 400

Feet

Aerial imagery provided by Esri.
Projection: NAD 1983 2011 StatePlane California III FIPS 0403 Ft US

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SITE PLAN

PELLET PROCESSING FACILITY

Jamestown, California

FIGURE	2
DRAWN BY	RWO
CHECKED BY	GHZ
PROJECT MGR	MMW
DATE	06/2021
WKA NO.	12774.03P

Project: Pellet Processing Facility
Project Location: Jamestown, California
WKA Number: 12774.03P

LOG OF TEST PIT TP1

Date(s) Drilled	**2/17/21**	Logged By	**GHZ**	Checked By	**MMW**
Drilling Method	**Backhoe**	Drilling Contractor	**All Septic Service Co.**	Total Depth of Drill Hole	**5.5 feet**
Drill Rig Type	**Case 580M**	Diameter(s) of Hole, inches	**24**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks	**Bulk (0-3')**			Driving Method and Drop	**Open Drive Sampler with 8-inch Sleeve**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Fill: Brown, moist, lean CLAY (CL) mixed with wood chips		TP1-1I TP1(0-3')		15.0	100	
			Dark blueish gray, wet, high plastic, sandy SILT (MH)		TP1-2I		52.0	60	LL = 53 PI = 13
	5		**Highly to moderately weathered bedrock:** Reddish brown, closely fractured, friable to weak		TP1-3I				

Practical Refusal was encountered at approximately 5½ feet below existing ground surface. Groundwater or seepage not observed.

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FIGURE 3

Project: Pellet Processing Facility
Project Location: Jamestown, California
WKA Number: 12774.03P

LOG OF TEST PIT TP2

Date(s) Drilled	**2/17/21**	Logged By	**GHZ**	Checked By	**MMW**
Drilling Method	**Backhoe**	Drilling Contractor	**All Septic Service Co.**	Total Depth of Drill Hole	**3.5 feet**
Drill Rig Type	**Case 580M**	Diameter(s) of Hole, inches	**24**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**Open Drive Sampler with 8-inch Sleeve**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Highly weathered bedrock: Reddish brown, moist, closely fractured, friable to weak, sandy silt consistency, with gravel and cobble						
					TP2-1I		15.0	88	
			becoming moderately weathered						
					TP2-2I				
			Practical Refusal was encountered at approximately 3½ feet below existing ground surface. Groundwater or seepage not observed.						

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FIGURE 4

Project: Pellet Processing Facility
Project Location: Jamestown, California
WKA Number: 12774.03P

LOG OF TEST PIT TP3

Date(s) Drilled	2/17/21	Logged By	GHZ	Checked By	MMW
Drilling Method	Backhoe	Drilling Contractor	All Septic Service Co.	Total Depth of Drill Hole	5.0 feet
Drill Rig Type	Case 580M	Diameter(s) of Hole, inches	24	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Observed	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Soil Cuttings
Remarks				Driving Method and Drop	Open Drive Sampler with 8-inch Sleeve

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Approximately 6 inches of Aggregate Base						
			Blueish gray, moist, high plasticity, sandy SILT (MH) with gravel and cobble						
					TP3-1I		13.0	70	PP = 4.0
			seepage (perched groundwater)						
			Highly to moderately weathered bedrock: Reddish brown, moist, strongly cemented, silty sand consistency						
					TP3-2I		16.0	70	
			becoming moderately weathered						
	5								
			Practical Refusal was encountered at approximately 5 feet below existing ground surface.						

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FIGURE 5

Project: Pellet Processing Facility
Project Location: Jamestown, California
WKA Number: 12774.03P

LOG OF TEST PIT TP4

Date(s) Drilled	2/17/21	Logged By	GHZ	Checked By	MMW
Drilling Method	Backhoe	Drilling Contractor	All Septic Service Co.	Total Depth of Drill Hole	6.5 feet
Drill Rig Type	Case 580M	Diameter(s) of Hole, inches	24	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Observed	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Soil Cuttings
Remarks				Driving Method and Drop	Open Drive Sampler with 8-inch Sleeve

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Fill: Dark brown, moist, lean CLAY (CL) with wood chips						
					TP4-1I		47.0	55	PP = 4.5
					TP4-2I		34.0	61	PP = 4.5
	5								
			Highly to moderately weathered bedrock: Reddish brown, closely fractured, friable to weak						
					TP4-3I				

Practical Refusal was encountered at approximately 6½ feet below existing ground surface.
Groundwater not observed.

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FIGURE 6

Project: Pellet Processing Facility **Project Location: Jamestown, California** **WKA Number: 12774.03P**	**LOG OF TEST PIT TP5** Sheet 1 of 1

Date(s) Drilled	**2/17/21**	Logged By	**GHZ**	Checked By	**MMW**
Drilling Method	**Backhoe**	Drilling Contractor	**All Septic Service Co.**	Total Depth of Drill Hole	**6.0 feet**
Drill Rig Type	**Case 580M**	Diameter(s) of Hole, inches	**24**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**Open Drive Sampler with 8-inch Sleeve**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Fill: Gray, moist, sandy gravel; fine to coarse gravel		TP5-1I				
	5		**Highly weathered bedrock:** Reddish brown, moist, strongly cemented, silty sand consistency becoming moderately weathered						
			Practical Refusal was encountered at approximately 6 feet below existing ground surface. Groundwater or seepage not observed.						

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FIGURE 7

Project: Pellet Processing Facility **Project Location: Jamestown, California** **WKA Number: 12774.03P**	**LOG OF TEST PIT TP6** Sheet 1 of 1

Date(s) Drilled	**2/17/21**	Logged By	**GHZ**	Checked By	**MMW**
Drilling Method	**Backhoe**	Drilling Contractor	**All Septic Service Co.**	Total Depth of Drill Hole	**3.0 feet**
Drill Rig Type	**Case 580M**	Diameter(s) of Hole, inches	**24**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**Open Drive Sampler with 8-inch Sleeve**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Approximately 3 inches of Asphalt Concrete						
			Approximately 9 inches of Aggregate Base						
			Highly weathered bedrock: Reddish brown, moist, closely fractured, strongly cemented,silty sand with gravel consistency		TP6-1I				
			becoming moderately weathered		TP6-2I				
			Practical Refusal was encountered at approximately 3 feet below existing ground surface. Groundwater not observed.						

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FIGURE 8

Project: Pellet Processing Facility **Project Location: Jamestown, California** **WKA Number: 12774.03P**	**LOG OF TEST PIT TP7** Sheet 1 of 1

Date(s) Drilled	**2/17/21**	Logged By	**GHZ**	Checked By	**MMW**
Drilling Method	**Backhoe**	Drilling Contractor	**All Septic Service Co.**	Total Depth of Drill Hole	**7.0 feet**
Drill Rig Type	**Case 580M**	Diameter(s) of Hole, inches	**24**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks	**Bulk (0-3')**			Driving Method and Drop	**Open Drive Sampler with 8-inch Sleeve**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			SAMPLE DATA				**TEST DATA**		
			Dark brown, moist, stiff, low plasticity, sandy CLAY (CL)		TP7-1I TP7(0-3')				PP = 1.0
			seepage (perched groundwater) near transition into weather bedrock						
			Highly weathered bedrock: Reddish brown, moist, moderately cemented, sandy silt consistency		TP7-2I				
	5								
			becoming moderately weathered		TP7-3I				
			Practical Refusal was encountered at approximately 7 feet below existing ground surface.						

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FIGURE 9

Project: Pellet Processing Facility **Project Location: Jamestown, California** **WKA Number: 12774.03P**	**LOG OF TEST PIT TP8** Sheet 1 of 1

Date(s) Drilled	**2/17/21**	Logged By	**GHZ**	Checked By	**MMW**
Drilling Method	**Backhoe**	Drilling Contractor	**All Septic Service Co.**	Total Depth of Drill Hole	**8.0 feet**
Drill Rig Type	**Case 580M**	Diameter(s) of Hole, inches	**24**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**Open Drive Sampler with 8-inch Sleeve**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark brown, moist, low plasticity, strongly cemented, silty CLAY (CL)						
					TP8-1I				
			Reddish brown, moist, strongly cemented, sandy SILT (ML)						
	5								
			Highly to moderately weathered bedrock: Reddish brown, closely fracture, friable to weak						
			Practical Refusal was encountered at approximately 8 feet below existing ground surface. Groundwater not observed.						

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FIGURE 10

Project: Pellet Processing Facility **Project Location: Jamestown, California** **WKA Number: 12774.03P**	**LOG OF TEST PIT TP9** Sheet 1 of 1

Date(s) Drilled	**2/17/21**	Logged By	**GHZ**	Checked By	**MMW**
Drilling Method	**Backhoe**	Drilling Contractor	**All Septic Service Co.**	Total Depth of Drill Hole	**3.5 feet**
Drill Rig Type	**Case 580M**	Diameter(s) of Hole, inches	**24**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**Open Drive Sampler with 8-inch Sleeve**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Approximately 3½ inches of Asphalt Concrete						
			Approximately 8 inches of Aggregate Base						
			Highly weathered bedrock: Reddish brown, moist, strongly cemented, sandy silt consistency		TP9-1I		22.0	82	
			becoming moderately weathered		TP9-2I				

Practical Refusal was encountered at approximately 3½ feet below existing ground surface.
Groundwater or seepage not observed.

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FIGURE 11

Project: Pellet Processing Facility Project Location: Jamestown, California WKA Number: 12774.03P	**LOG OF TEST PIT TP10** Sheet 1 of 1

Date(s) Drilled	2/17/21	Logged By	GHZ	Checked By	MMW
Drilling Method	Backhoe	Drilling Contractor	All Septic Service Co.	Total Depth of Drill Hole	6.0 feet
Drill Rig Type	Case 580M	Diameter(s) of Hole, inches	24	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Observed	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Soil Cuttings
Remarks				Driving Method and Drop	Open Drive Sampler with 8-inch Sleeve

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Approximately 4 inches of Asphalt Concrete						
			Approximately 8 inches of Aggregate Base						
			Reddish brown, moist, strongly cemented, low plastic, sandy SILT (ML) with cobbles						
	5		**Highly to moderately weathered bedrock:** Reddish brown with gray motling, closely fractured, friable to weak	X	TP10-3I				
			Practical Refusal was encountered at approximately 6 feet below existing ground surface. Groundwater not observed.						

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FIGURE 12

Project: Pellet Processing Facility
Project Location: Jamestown, California
WKA Number: 12774.03P

LOG OF TEST PIT TP11

Sheet 1 of 1

Date(s) Drilled	**2/17/21**	Logged By	**GHZ**	Checked By	**MMW**
Drilling Method	**Backhoe**	Drilling Contractor	**All Septic Service Co.**	Total Depth of Drill Hole	**5.5 feet**
Drill Rig Type	**Case 580M**	Diameter(s) of Hole, inches	**24**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks	**Bulk (0-3')**			Driving Method and Drop	**Open Drive Sampler with 8-inch Sleeve**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Brown, very moist, low plasticity, sandy lean CLAY (CL)		TP11-1I TP11(0-3')				
			Highly weathered bedrock: Reddish brown, moist, strongly cemented, closely fractured silty clay inclusions		TP11-2I				
	5		becoming moderately weathered						
			Practical Refusal was encountered at approximately 5.5 feet below existing ground surface. Groundwater not observed.						

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FIGURE 13

Project: Pellet Processing Facility **Project Location: Jamestown, California** **WKA Number: 12774.03P**	**LOG OF TEST PIT TP12** Sheet 1 of 1

Date(s) Drilled	**2/17/21**	Logged By	**GHZ**	Checked By	**MMW**
Drilling Method	**Backhoe**	Drilling Contractor	**All Septic Service Co.**	Total Depth of Drill Hole	**5.5 feet**
Drill Rig Type	**Case 580M**	Diameter(s) of Hole, inches	**24**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks	**Bulk (0-3')**			Driving Method and Drop	**Open Drive Sampler with 8-inch Sleeve**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, low plastic, sandy SILT (ML) with fine gravel						
					TP12-1I TP12(0-3')		21.0	58	
			seepage (perched groundwater) encountered at transition with weathered bedrock		TP12-2I		32.0	84	
	5		**Highly weathered bedrock:** Reddish brown, moist, strongly cemented, sandy silt consistency						
			becoming moderately weathered						

Practical Refusal was encountered at approximately 5½ feet below existing ground surface. Groundwater not observed.

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FIGURE 14

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

MAJOR DIVISIONS		USCS[4]	CODE	CHARACTERISTICS
COARSE GRAINED SOILS (More than 50% of soil > no. 200 sieve size)	GRAVELS[1] (More than 50% of coarse fraction > no. 4 sieve size)	GW		Well-graded gravels or gravel - sand mixtures, trace or no fines
		GP		Poorly graded gravels or gravel - sand mixtures, trace or no fines
		GM		Silty gravels, gravel - sand - silt mixtures, containing little to some fines[2]
		GC		Clayey gravels, gravel - sand - clay mixtures, containing little to some fines[2]
	SANDS[1] (50% or more of coarse fraction < no. 4 sieve size)	SW		Well-graded sands or sand - gravel mixtures, trace or no fines
		SP		Poorly graded sands or sand - gravel mixtures, trace or no fines
		SM		Silty sands, sand - gravel - silt mixtures, containing little to some fines[2]
		SC		Clayey sands, sand - gravel - clay mixtures, containing little to some fines[2]
FINE GRAINED SOILS (50% or more of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML		Inorganic silts, gravely silts, and sandy silts that are non-plastic or with low plasticity
		CL		Inorganic lean clays, gravelly lean clays, sandy lean clays of low to medium plasticity[3]
		OL		Organic silts, organic lean clays, and organic silty clays
	SILTS & CLAYS LL ≥ 50	MH		Inorganic elastic silts, gravelly elastic silts, and sandy elastic silts
		CH		Inorganic fat clays, gravelly fat clays, sandy fat clays of medium to high plasticity
		OH		Organic fat clays, gravelly fat clays, sandy fat clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT		Peat
ROCK		RX		Rocks, weathered to fresh
FILL		FILL		Artificially placed fill material

OTHER SYMBOLS

= Drive Sample: 2-1/2" O.D. Modified California sampler
= Drive Sampler: no recovery
= SPT Sampler
= Initial Water Level
= Final Water Level
— — — = Estimated or gradational material change line
———— = Observed material change line

Laboratory Tests

CR = Corrosion
PI = Plasticity Index
EI = Expansion Index
UCC = Unconfined Compression Test (TSF)
TR = Triaxial Compression Test
GR = Gradational Analysis (Sieve/Hydro)
FC = Wash (Fines Content)
PP = Pocket Penetrometer Test (TSF)
PID = Photo Ionization Detector Test (PPM)
RV = Resistance ("R") Value

REF = Refusal (>50 blows in 6 inches)

GRAIN SIZE CLASSIFICATION

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS (b)	Above 12"	Above 300
COBBLES (c)	12" to 3"	300 to 75
GRAVEL (g) coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	75 to 4.75 75 to 19 19 to 4.75
SAND coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.75 to 0.075 4.75 to 2.00 2.00 to 0.425 0.425 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

Trace - Less than 5 percent
Few - 5 to 10 percent
Little - 15 to 25 percent
Some - 35 to 45 percent
Mostly - 50 to 100 percent

* Percents as given in ASTM D2488

NOTES:

1. Coarse grained soils containing 5% to 12% fines, use dual classification symbol (ex. SP-SM).
2. If fines classify as CL-ML (4<PI<7), use dual symbol (ex. SC-SM).
3. Silty Clays, use dual symbol (CL-ML).
4. Borderline soils with uncertain classification list both classifications (ex. CL/ML).

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UNIFIED SOIL CLASSIFICATION SYSTEM

PELLET PROCESSING FACILITY

Jamestown, California

FIGURE	15
DRAWN BY	RWO
CHECKED BY	GHZ
PROJECT MGR	MMW
DATE	06/2021
WKA NO. 12774.03P	

APPENDIX A
General Information, Field Exploration and Laboratory Testing

APPENDIX A

A. GENERAL INFORMATION

The geotechnical engineering study for the Pellet Processing Facility to be located at 12001 La Grange Road near Jamestown, California, was authorized by Mr. Greg Norton representing the Golden State Finance Authority (GSFA) on February 2, 2021. Authorization was for a study as described in our proposal dated January 22, 2021, sent to our client Ms. Barbara Hayes with the GSFA at 1215 K Street, Suite 1650 in Sacramento, California 95814; telephone (916) 447-4806.

B. FIELD EXPLORATION

Twelve exploratory test pits were excavated at the site on February 17, 2021, utilizing a Case 580M rubber-tire backhoe equipped with a 24-inch-wide bucket provided by All Septic Service Company of Soulsbyville, California. The test pits were excavated to a maximum depth of about eight feet below the existing ground surface, or to refusal to excavation, at the approximate locations shown in Figure 2. Relatively undisturbed soil samples were obtained using a drive tube and slide hammer and disturbed bulk samples were collected during the field exploration and taken to our laboratory for additional soil classification and selection of samples for testing. Finally, two percolation tests were performed at the locations shown on Figure 2 in accordance with test procedures outlined in the United States Environmental Protection Agency (EPA) Design Manual for Onsite Wastewater Treatment and Disposal Systems (1980).

The Logs of Test Pits containing descriptions of the soil and weathered bedrock encountered in each test pit are presented in Figures 3 through 14. A Legend explaining the Unified Soil Classification System (ASTM D2487) and the symbols used on the logs is contained in Figure 15.

C. LABORATORY TESTING

Selected undisturbed samples of the soils were tested to determine dry unit weight (ASTM D2937) and natural moisture content (ASTM D2216). The results of these tests are included in the test pit logs at the depth each sample was obtained.

Two soil samples was collected from Test Pits TP1 and TP4 were tested to determine the Atterberg Limits (ASTM D4318). The results of the tests are presented in Figure A1 and included in the test pit logs.

Two bulk samples of anticipated pavement subgrade soil were collected and for Resistance-value ("R-value") testing in accordance with California Test 301. The results of the R-value tests, along with the test pit number and sample depth, are presented in Figure A2.

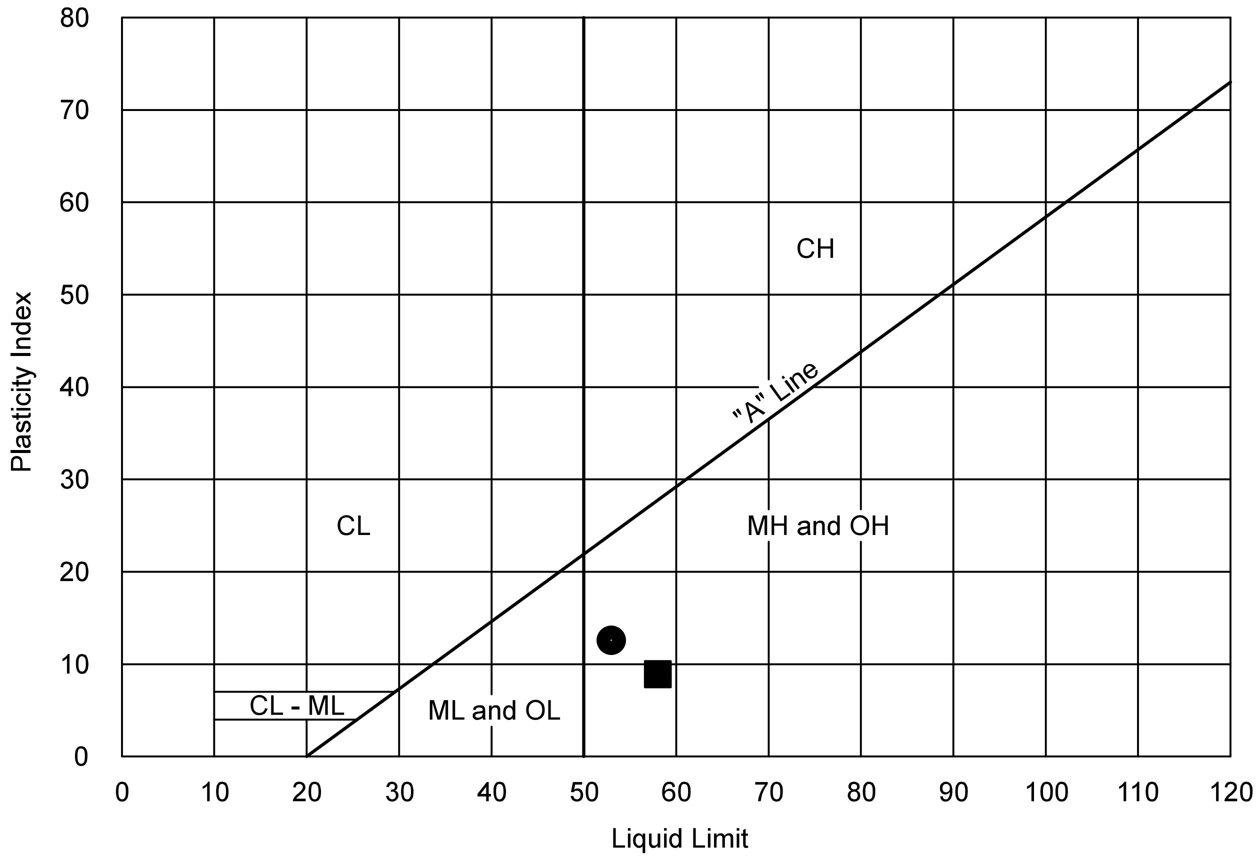
Two selected soil samples of near-surface soil was submitted to Sunland Analytical of Rancho Cordova, California, to determine the soil pH and minimum resistivity (California Test 643), Chloride concentration (California Test 422m), and Sulfate concentration (California Test 417, ASTM D516m). The results of these tests are presented in Figures A3 through A6.

One sample of the highly weathered bedrock was submitted to MicroTest Laboratories, Inc. of Rancho Cordova, California, for Naturally Occurring Asbestos testing in accordance with California Air Resources Board 435 test method. The results are presented in Figure A7.

One bulk sample of the residual subgrade soil was collected to determine the maximum theoretical dry density and corresponding optimum moisture content in accordance with ASTM D1557. The results of the test, along with the test pit number and sample depth, are presented in Figure A8.

ATTERBERG LIMITS

ASTM D4318



KEY SYMBOL	LOCATION	SAMPLE DEPTH	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	PASSING No. 200 SIEVE (%)	UNIFIED SOIL CLASSIFI-CATION SYMBOL
●	TP1-2I	0'-5.0'	---	53	13	---	MH
■	TP4-2I	2.5'-3.0'	---	58	9	---	MH

WallaceKuhl & ASSOCIATES

ATTERBERG LIMITS

PALLET PROCESSING FACILITY

Jamestown, California

FIGURE	A1
DRAWN BY	RWO
CHECKED BY	GHZ
PROJECT MGR	MMW
DATE	06/2021
WKA NO.12774.03P	

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Multi colored, sandy clay with organics

LOCATION: TP7 (0' - 3')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	103	18.8	627	40	173	58
2	100	19.9	455	17	74	53
3	97	20.7	284	7	30	44

R-Value at 300 psi exudation pressure = 45

MATERIAL DESCRIPTION: Brown, silty sand with rock

LOCATION: TP11 (0' - 3')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	109	15.6	656	119	515	62
2	105	17.8	438	44	191	43
3	102	19.0	269	48	208	28

R-Value at 300 psi exudation pressure = 30

WallaceKuhl & ASSOCIATES

RESISTANCE VALUE TEST RESULTS

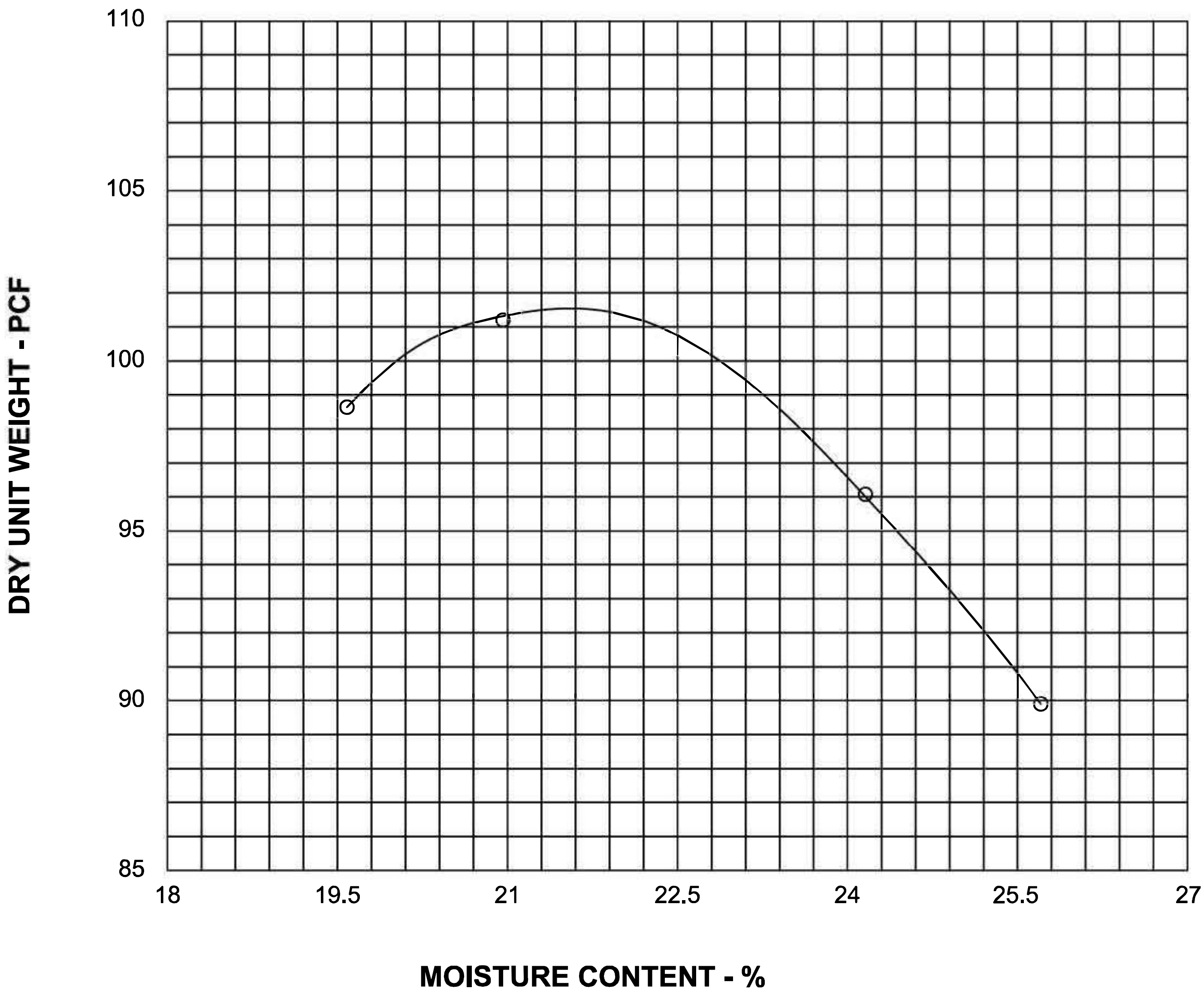
PALLET PROCESSING FACILITY

Jamestown, California

FIGURE	A2
DRAWN BY	RWO
CHECKED BY	GHZ
PROJECT MGR	MMW
DATE	06/2021

WKA NO. 12774.03P

MOISTURE-DENSITY RELATION



Material: Dark brown, sandy clay

Location: TP12 (0' - 3')

Test Method: ASTM D1557 "A"

Maximum Dry Unit Weight - PCF: 101.5

Optimum Moisture - %: 21.6

WallaceKuhl & ASSOCIATES	COMPACTION CURVE PELLET PROCESSING FACILITY Jamestown, California	FIGURE	A3
		DRAWN BY	RWO
		CHECKED BY	GHZ
		PROJECT MGR	MMW
		DATE	06/2021
		WKA NO.12774.03P	

Sunland Analytical

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

Date Reported 02/26/2021
Date Submitted 02/22/2021

To: Guang Zhu
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

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

The reported analysis was requested for the following location:
Location : 12774.03 Site ID : TP 7 (0-3).
Thank you for your business.

* For future reference to this analysis please use SUN # 84090-175313.

EVALUATION FOR SOIL CORROSION

Soil pH	5.75	
Minimum Resistivity	1.96 ohm-cm (x1000)	
Chloride	19.3 ppm	00.00193 %
Sulfate	48.1 ppm	00.00481 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

PALLET PROCESSING FACILITY

Jamestown, California

FIGURE	A4
DRAWN BY	RWO
CHECKED BY	GHZ
PROJECT MGR	MMW
DATE	06/2021
WKA NO. 12774.03P	

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

Date Reported 02/26/2021
Date Submitted 02/22/2021

To: Guang Zhu
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

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

The reported analysis was requested for the following location:
Location : 12774.03 Site ID : TP7 (0-3).
Thank you for your business.

* For future reference to this analysis please use SUN # 84090-175314.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	47.7	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl
& ASSOCIATES

CORROSION TEST RESULTS
PALLET PROCESSING FACILITY
Jamestown, California

FIGURE	A5
DRAWN BY	RWO
CHECKED BY	GHZ
PROJECT MGR	MMW
DATE	06/2021
WKA NO. 12774.03P	

Sunland Analytical

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

Date Reported 02/26/2021
Date Submitted 02/22/2021

To: Guang Zhu
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

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

The reported analysis was requested for the following location:
Location : 12774.03 Site ID : TP 11 (0-3).
Thank you for your business.

* For future reference to this analysis please use SUN # 84090-175315.

EVALUATION FOR SOIL CORROSION

Soil pH	5.95		
Minimum Resistivity	0.59 ohm-cm (x1000)		
Chloride	176.6 ppm	00.01766	%
Sulfate	464.0 ppm	00.04640	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

PALLET PROCESSING FACILITY

Jamestown, California

FIGURE	A6
DRAWN BY	RWO
CHECKED BY	GHZ
PROJECT MGR	MMW
DATE	06/2021
WKA NO. 12774.03P	

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

Date Reported 02/26/2021
Date Submitted 02/22/2021

To: Guang Zhu
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

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

The reported analysis was requested for the following location:
Location : 12774.03 Site ID : TP 11 (0-3).
Thank you for your business.

* For future reference to this analysis please use SUN # 84090-175316.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	433.4	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl & ASSOCIATES	**CORROSION TEST RESULTS** PALLET PROCESSING FACILITY Jamestown, California	FIGURE A7 DRAWN BY RWO CHECKED BY GHZ PROJECT MGR MMW DATE 06/2021 WKA NO. 12774.03P

MicroTest Laboratories, Inc. | NVLAP Code: 200999-0
3110 Gold Canal Dr, Ste. A, Rancho Cordova, CA 95670
PH 916.567.9808 | FX 916.404.0302
www.microtestlabsinc.com | service@microtestlabsinc.com

for office use only

Accession Numbers:

247746

CLIENT INFORMATION

Company	CLS Labs
Name	
Address	3249 Fitzgerald Road Rancho Cordova, CA 95742
Phone	916-638-7301
Email	Sub@californialab.com

SAMPLE

Date	Friday, February 19, 2021
Time	2:30 PM

JOB SITE INFORMATION

Sampler	
Project	21B1202
Address	

MicroTest Laboratories

Analytical Data

POLARIZED LIGHT MICROSCOPY (PLM) EPA METHOD 600 / R-93 / 116-Carb 435 Level A (0.25%)

Sample ID	Accession Number	Location	Description	Non Fibrous / Fibrous Materials	Asbestiform Minerals %
TP12-3I	247746		Brown Soil	99+% Binder <1% Other	None Detected

REPORT

Date	Monday, March 1, 2021

Samples Received: 1

Samples Analyzed: 1

Analyst: Nolan Starbuck

Authorized Signatory:

Kelly Favero - Lab Manager

This analytical data sheet constitutes a final report. Due to the limitation of Polarized Light Microscopy (PLM), some samples classified as containing no asbestos in materials, None Detected (ND), such as floor tiles or like materials, warrant a recommendation for further analysis by Transmission Electron Microscopy (TEM). This report is limited to items analyzed here within.This report must not be used by the client to claim product endorsement by NVLAP or any agency of the U.S. Government. All Samples will be held for not less than 30 days, upon which they will then be disposed of. This report shall not be reproduced in full without written authorization from MicroTest Laboratories, Inc. Soil and rock matrices are considered problematic matrices and MicroTest recommends sample homogenization prior to PLM analysis. Thermal decomposition of asbestos fibers will yield non-asbestiform mineral properties.

Analytical Page # 1 of 1

WallaceKuhl & ASSOCIATES

ASBESTOS TEST RESULTS

PALLET PROCESSING FACILITY

Jamestown, California

FIGURE	A8
DRAWN BY	RWO
CHECKED BY	GHZ
PROJECT MGR	MMW
DATE	06/2021
WKA NO. 12774.03P	