GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
for the
NORTH STATE GROCERY ORLAND CENTER PHOTOVOLTAIC INSTALLATION
35 East Walker Street, Orland, California

Prepared for:
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Project No. 70589-01
May 1, 2016

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Mr. Kevin Berryhill
Bright Power, Inc.
PO Box 10637
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REFERENCE: ***North State Grocery Orland Center Photovoltaic Installation***
35 East Walker Street
Orland, Glenn County, California

SUBJECT: ***Geotechnical Engineering Investigation Report***

Dear Mr. Berryhill,

Holdrege & Kull (H&K) is pleased to provide geotechnical engineering services for the proposed photovoltaic arrays on the North State Grocery, Orland Center at the above referenced address.

H&K understands that the photovoltaic arrays will include carport type shade canopy installations within the parking lot. The findings, conclusions, and recommendations presented in this report are based on H&K's literature review, surface observations, subsurface exploration, and our experience with similar projects and sites and conditions in the area. It is our opinion that the site is suitable for the proposed construction provided the geotechnical engineering recommendations presented in this report are incorporated into the proposed improvements.

H&K appreciates the opportunity to provide geotechnical engineering services for this project. If you have questions or need additional information, please do not hesitate to contact the undersigned below at 530-894-2487.

Sincerely,
HOLDREGE & KULL

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1 INTRODUCTION

Holdrege & Kull (H&K) performed a geotechnical engineering investigation of the proposed photovoltaic (PV) array sites to be located at the North State Grocery, Orland Center (NSG1 Orland), 35 East Walker Street, in Orland, California. The work performed is consistent with the scope of services presented in the proposed scope of work and contract agreement executed on March 21, 2016. A copy of the scope of work, excluding the fee and contract sections, is included in Appendix A.

For your review, Appendix B presents a document prepared by ASFE entitled *"Important Information About Your Geotechnical Engineering Report."* This document summarizes project specific factors, limitations, content interpretation, responsibilities, and other pertinent information. Please read this document carefully.

1.1 SCOPE-OF-SERVICES

H&K performed a specific scope-of-services to develop geotechnical engineering design recommendations for earthwork and structural improvements and to identify potential conditions that may negatively impact the site and require design mitigation. Brief descriptions of each work scope task performed for the geotechnical engineering investigation are presented below. A detailed description of each work scope task is presented in Section 2 (Site Investigation) of this report.

- **Task 1, Site Investigation:** H&K performed a site investigation to characterize the existing surface and subsurface soil, rock, and groundwater conditions encountered to the maximum depth drilled and explored. H&K's field geologist made observations, collected representative soil samples, and performed field tests at a limited number of subsurface exploratory locations. H&K performed laboratory tests on selected soil samples to evaluate their geotechnical engineering material properties.
- **Task 2, Data Analysis and Engineering Design:** H&K evaluated the field and laboratory data, reviewed the proposed site improvements, and used this information to develop geotechnical engineering design recommendations for the proposed improvements.
- **Task 3, Report Preparation:** H&K prepared this report to present our findings, conclusions and recommendations for the proposed improvements.

1.2 SITE LOCATION AND DESCRIPTION

The NSG1 Orland site is located at 35 East Walker Street, in Orland, California. The site is centered at approximately latitude 39.747° North and longitude -122.188° and an elavation of approximately 250 feet above mean sea level. Figure 1 shows the site location and near vicinity. The property is a generally rectangular shaped parcel developed with a shopping center and grocery store. East Walker Street (State

Route 32) is situated along the southern property boundary, Swift Street is situated along the northern property boundary, and the east and west property boundaries have mixed commercial and residential properties.

At the time that our site investigation was performed on April 5, 2016 the site conditions and improvements consisted of a flat lying parking lot with flat lying topography surrounding the shopping center and grocery store buildings. Existing surface improvements at the proposed NSG1 Orland PV installation site includes an asphalt concrete (AC) paved parking lot and delivery truck drive route.

1.3 PROPOSED IMPROVEMENTS

The photovoltaic arrays will be constructed using pier foundations with design dimensions depending on the subsurface soil and rock conditions encountered at the site. Based on our experience with similar projects, we anticipate that the critical design loading for the structures will result from transient wind loading. Based on past experience with similar PV shade structures, H&K understands that the carport shade canopy PV arrays will be supported by cast-in-drill-hole (CIDH) concrete piers with hollow structural shape (HSS) steel columns and cantilevered beams. H&K assumes the proposed carport array will utilize a T-style array with a center column and cantilevered beams that support the purlin and PV modules.

1.4 INVESTIGATION PURPOSE

The purpose of our investigation was to obtain information about the soil, rock and groundwater conditions likely to be encountered at the site to allow us to prepare geotechnical engineering design recommendations for the proposed improvements. H&K did not evaluate the site for the presence of hazardous waste, mold, asbestos, and radon gas. Therefore, the presence and removal of these materials are not discussed in this report.

2 SITE INVESTIGATION

H&K performed a site investigation to characterize the existing subsurface conditions beneath the proposed NSG1 Orland to develop geotechnical engineering recommendations for earthwork and structural improvements. Each component of our site investigation is presented below.

2.1 LITERATURE REVIEW

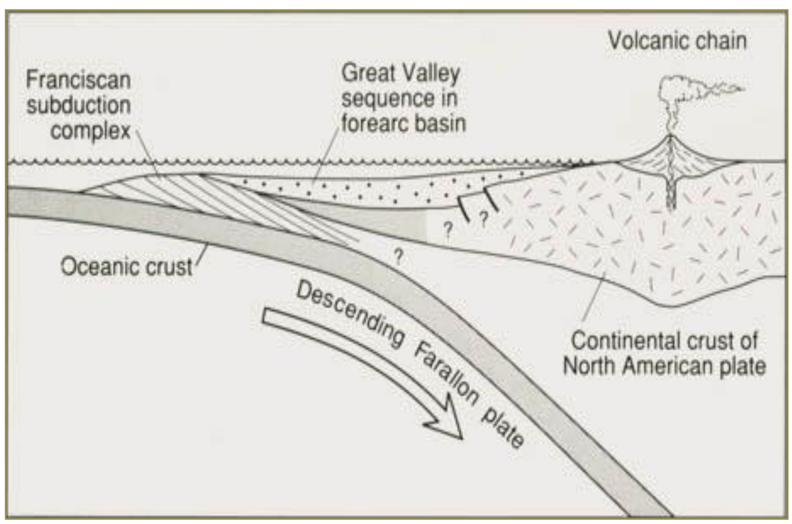
H&K performed a limited review of available literature that was pertinent to the project site. The following summarizes our findings.

2.1.1 Site Improvement Plan Review

The site improvement plans were not available at the time this report was prepared. Prior to implementing grading and site improvements, H&K should be allowed to review the final plans to determine whether our recommendations have been implemented, and if necessary, to provide additional and/or modified recommendations.

2.1.2 Geologic Setting and Regional Faulting

The subject property is situated in the northern Sacramento Valley within the Great Valley geologic province west of the boundary with the Cascade geologic province and east of the boundary of the Coast Range geologic province. The Great Valley geologic province is characterized as an asymmetrical synclinal trough filled with sequences of up to 80,000 feet thick of Jurassic age (138 to 205 million years before present [mybp]) and Eocene age (38 to 55 mybp) marine sedimentary units deposited during periods of inundation, and Pliocene age (2 to 5 mybp) to recent Holocene age (present to 11,000 years before present) terrestrial sediments originating from the Sierra Nevada, Cascade, and Coast Mountain Ranges during sea recession and periods of mountain uplift. The inset figure shows a generalized geologic cross-section of the ancient Great Valley depositional environment consisting of a forearc basin situated where the Farallon Oceanic Plate plunges beneath the North American Continental Plate.



In the central part of the Sacramento Valley, a mantle of Tertiary age (2 to 63 mybp) and Quaternary age (present to 2 mybp) detrital continental deposits overlies the Great Valley sequence; these deposits, which are derived from the Coast Range mountains to the west, grade eastward into coeval volcanic materials derived from the Cascade Range province (Blake, et al, 2009).

The *Geologic Map the Late Cenozoic Deposits of the Sacramento Valley and Northern Sierran Foothills, California* published by the United States Geological Survey, indicates that the geology immediately underlying the subject site consists of fluvial alluvium sediments of the Modesto Formation which were deposited during the Pleistocene Epoch (1.5 Million Years to 11,000 before present). The Modesto Formation is characterized as distinct alluvial terraces with some alluvial fans, and abandoned channel ridges comprised of unweathered gravel, sand, silt and clay (Helley, J., Harwood, D., 1985).

Regional faulting is associated with the northern extent of the Foothill Fault System and the northern section of the Great Valley Fault Zone. The 1997 edition of California Geological Survey Special Publication 43, *Fault Rupture Hazard Zones in California*, describes active faults and fault zones (activity within 11,000 years), as part of the Alquist-Priolo Earthquake Fault Zoning Act. This map and document indicates that the site is not located within an Alquist-Priolo active fault zone.

H&K reviewed the Official Maps of Earthquake Fault Zones delineated by the California Geological Survey through December 2010, on the internet at http://www.quake.ca.gov/gmaps/ap/ap_maps.htm. These maps are updates to Special Publication 42, Interim Revision 2007 edition *Fault Rupture Hazard Zones in California*, which describes active faults and fault zones, as part of the Alquist-Priolo Earthquake Fault Zoning Act. Special Publication 42 and the 2010 on-line update indicate that the site is not located within an Alquist-Priolo active fault zone. Currently there are no proposed earthquake fault zone maps in the immediate area of Orland, California.

The 2010 Fault Activity Map of California by the California Geological Survey, Geologic Data Map No. 6 (http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#) shows the nearest known active faults with surface displacement within Holocene time (about the last 11,000 years). The Dunnigan Hill Fault, located approximately 60 miles south of the subject site and the Bartlett Springs Fault, located approximately 50 miles west-southwest of the subject site show evidence of Holocene faulting, and the Cleveland Hills Fault, located approximately 45 miles to the southeast of the subject site, is associated with ground surface rupture during the 1975 Oroville earthquake.

The nearest fault without documented Quaternary displacement is the Corning Fault located approximately 1 mile west of the subject site. The Corning Fault is a north trending fault that appears to have splayed off of the main stem of the Willows Fault which believed to be the axial Central Valley suture between oceanic and Sierran crust (Harwood and Helley, 1987, and Hausback, 1991).

2.2 FIELD INVESTIGATION

H&K performed a field investigation of the site on April 5, 2016. H&K's Field Geologist described the surface and subsurface soil, rock, and groundwater conditions observed at the site using the procedures cited in the American Society for Testing and Materials (ASTM), Volume 04.08,*"Soil and Rock; Dimension Stone; and Geosynthetics"* as general guidelines for our field and laboratory procedures. The Field Engineer/Geologist described the soil color using the general guideline procedures presented in the Munsel Soil Color Chart. Engineering judgment was used to extrapolate the observed surface and subsurface soil, rock, and groundwater conditions to areas located between and beyond our subsurface exploratory locations. The surface, subsurface, and groundwater conditions observed during our field investigation are summarized below.

2.2.1 Surface Conditions

H&K observed the following surface conditions during our field investigation of the property. Figure 2 shows the project site boundaries and our subsurface exploration location. The site asphalt concrete parking lot is generally flat lying with minor grade breaks to accommodate storm water drainage. The asphalt was in good conditions with no signs of subgrade distress, failure, or settlement. There were no surface conditions of concern identified during our site investigation.

2.2.2 Subsurface Conditions

The subsurface soil, rock and groundwater conditions were investigated by drilling exploratory borings and performing a seismic refraction survey at the site. The subsurface information obtained from these investigation methods are described herein.

2.2.2.1 Exploratory Boring Information

H&K provided engineering oversight for the advancement of two exploratory soil borings at the project site with a truck mounted CME 75; drill rig equipped with 7.25-inch-outside diameter, hollow stem, augers. Figure 2 shows the approximate location of the subsurface exploratory excavation. The borings were advanced to depths of 15.7 and 16.5 feet below ground surface (bgs). Engineering judgment was used to extrapolate the observed soil, rock, and groundwater conditions to areas located between and beyond our subsurface exploratory excavation.

H&K's Field Engineer/Geologist logged each exploratory boring using the Unified Soils Classification System (USCS) as guidelines for soil descriptions and the American Geophysical Union guidelines for rock descriptions. Representative relatively undisturbed soil samples were collected at 2 feet bgs then 5 feet bgs and continued at 5-foot-depth intervals bgs for the entire depth of the exploratory borings. Relatively undisturbed soil samples were collected with a 2.5-inch-inside-diameter, split-spoon sampler equipped with steel liner sample tubes and with an unlined standard penetration test (SPT) split barrel sampler. The samplers were driven into the soil using a 140 pound automatic trip hammer with a 30-inch-free fall. The steel liner tube samples were sealed with end-caps, labeled and transported to our soil laboratory facility.

Selected soil samples were tested in H&K's laboratory to determine their engineering material properties which included: natural moisture content, density and particle size gradation. These soil engineering material properties were used to develop the geotechnical engineering recommendations that are presented herein for: foundation.

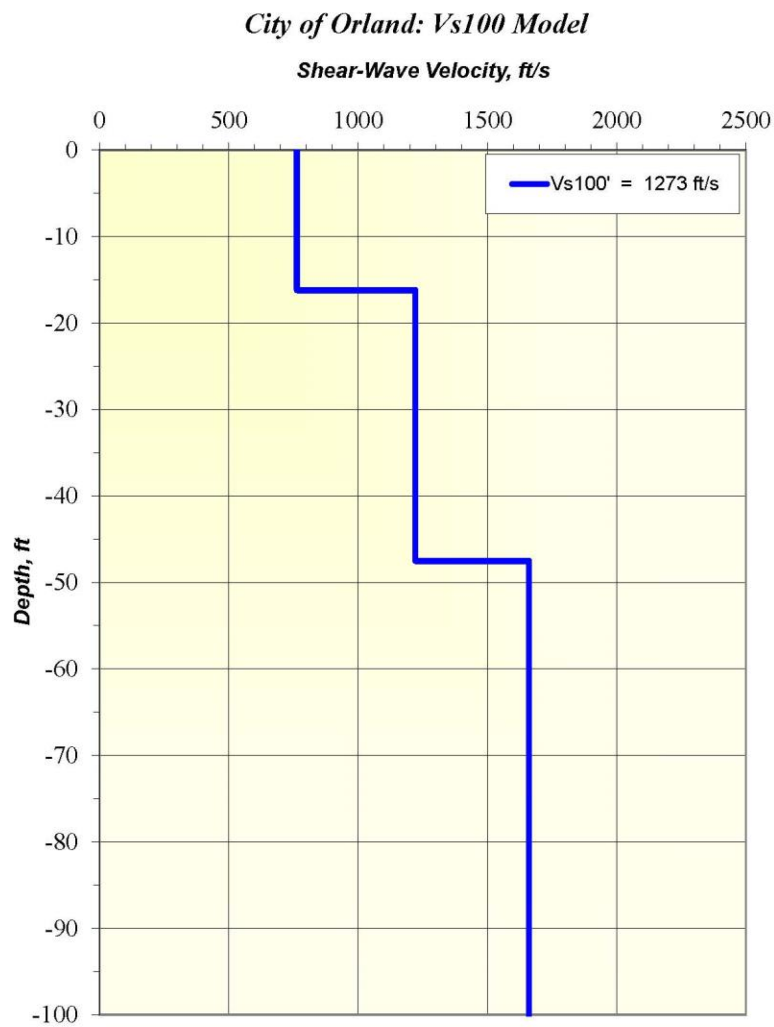
Detailed descriptions of the soil, rock, and groundwater conditions that were encountered in each subsurface exploratory location are presented on the exploratory boring logs included in Appendix C. The soil and rock descriptions are based on visual field estimates of the particle size percentages (by dry weight), color, relative density or consistency, moisture content, and cementation that comprise each soil material encountered.

A generalized profile of the soil, rock, and groundwater conditions encountered to the maximum depth drilled (16.5 feet) for the proposed PV array area is presented below. The soil and/or rock units encountered in our subsurface exploratory excavations were generally stratigraphically continuous across the site; however, the units may vary slightly in thickness. The units encountered in general stratigraphic sequence during our subsurface investigation of the site are described below.

- **GM, Silty Gravel Soil:** This soil consists of the following field estimated particle size percentages 40 percent fine gravel, 35 percent fine to medium sand, and 25 percent low plasticity clay and silt size particles. This soil is predominantly brown with a Munsel Color Chart designation of (7.5YR 4/2). This soil was medium dense and damp at the time of our subsurface investigation.
- **GW, Well Graded Gravel Soil:** This soil consists of the following field estimated particle size percentages 60 percent fine to coarse gravel, 35 percent fine sand, and 5 percent low plasticity fines. This soil is predominantly brown with a Munsel Color Chart designation of (7.5YR 5/2). This soil was dense and damp to wet at the time of our subsurface investigation.

2.2.2.2 Seismic Refraction Survey

H&K performed a seismic refraction microtremor survey in the City of Orland, California. The survey is located 1,900 feet southwest of the solar array and within the same geologic units. The refraction survey was performed on the site using the SeisOpt® ReMi™ Vs30 method to determine the in-situ shear-wave (S-wave) velocity profile of the first 100 feet of soil beneath the site.



SeisOpt® ReMi™ Vs30 method was used to determine the 2013 California Building Code (CBC) Site Class in accordance with Chapter 16, Section 1613.3.2 and Chapter 20 of ASCE 7-10. The seismic refraction survey was performed at the surface using conventional seismograph and vertical P-wave geophones used for refraction surveys. The seismic source consists of ambient seismic microtremors which were constantly being generated by cultural and natural noise in the area. H&K recorded the seismic vibrations generated by the drill rig, vehicle traffic along Interstate 5, and industrial operations in the area during the site investigation. The data was collected during a series of 23 recording periods that were each 30 seconds in duration. The Vs Model (depicted on this page) shows the subsurface shear-wave velocity profile that was developed for the site from the SeisOpt® ReMi™ data.

The resulting subsurface shear wave model for the site indicates that the harmonic mean seismic shear wave velocity for the upper 100 feet of the subsurface was 1,273 feet per second (ft/s). This weighted shear wave velocity corresponds to the upper range of Site Class C which represents very dense soil and soft rock profiles.

2.2.2.3 Groundwater Conditions

Groundwater was encountered at depths of approximately 15 feet bgs in the exploratory borings drilled at the site. Seasonal fluctuations in the local groundwater table at the project site and vicinity are unknown at this time; however it is generally understood that the groundwater table elevation is highest at the end of the winter rainy season and lowest at the end of the summer dry season. Based on a review of shallow groundwater data from monitoring wells situated west-southwest of the subject site along Interstate 5, indicates that the groundwater level ranged from approximately 241 to 246 feet above mean sea level and the flow direction is generally to the southeast. NSG1 Orland is located east of Interstate 5 at an approximate elevation of 250 feet above mean sea level, which indicates that the depth to historically high groundwater table underlying the NSG1 Orland site could be approximately 4 to 9 feet bgs.

3 LABORATORY TESTING

H&K performed laboratory tests on selected soil samples taken from the subsurface exploratory excavations to determine their engineering material properties. These engineering material properties were used to develop geotechnical engineering design recommendations for earthwork and structural improvements. The following laboratory tests were performed using the cited American Society for Testing and Materials (ASTM) and Caltrans Test Method (CTM) guideline procedures:

- ASTM D422, Particle Size Gradation (Sieve and Hydrometer Methods)
- ASTM D2216, Moisture Content
- ASTM D2487, Soil Classification by the USCS
- ASTM D2488, Soil Description (Visual Manual Method)
- ASTM D2937, Density

Table 3-1 presents a summary of the laboratory test results. Appendix D presents the laboratory test data sheets.

Table 3-1. Laboratory Test Results

			Results					
ASTM or CTM No.			D2487 D2488	D2216	D2937	D422		D2844 CTM301
Boring No.	Sample No.	Sample Depth (feet)	USCS (sym)	Moisture Content (%)	Dry Density (pcf)	Passing No. 4 (%)	Passing No. 200 (%)	R-Value (dim)
B16-1	L1-1-1	2.5	GW	2.9	123.7	58.4	4.0	
B16-1	L2-1-1	5	GW	2.9	122.7	78.9	5.6	

Notes:
ASTM = American Society for Testing and Materials
CTM = Caltrans Test Method
dim = dimensionless units
No. = number
pcf = pounds per cubic foot
sym = symbol
% = percent

4 CONCLUSIONS

The conclusions presented below are based on information developed from our field and laboratory investigations.

1. It is our opinion that the site is suitable for the proposed construction improvements provided that the geotechnical engineering design recommendations presented in this report are incorporated into the earthwork and structural improvement project plans.
2. Prior to construction, H&K should be allowed to review the proposed final earthwork grading plan and structural improvement plans to determine if our geotechnical engineering recommendations are applicable or need modifications.
3. Based on the site geology, the observations of our exploratory borings, and the SeisOpt ReMi Vs30 shear-wave profile analysis, the site soil profile can be modeled, according to the 2010 CBC, Chapter 16A, Table 1613A.5.2, and Section 1613A.5.5, as a Site Class C (very dense soil and soft rock profile) designation for the purposes of establishing seismic design loads for the proposed improvements.
4. Based on our literature review and knowledge of the geology in the Orland, California area, no active or potentially active faults are known to underlie the site.
5. Based on the subsurface exploratory boring sampler blow counts, field data, and literature review, H&K believes that the site soil and groundwater conditions makes the probability of liquefaction occurring during a nearby earthquake to be extremely low.
6. At the time of our investigation the site consisted of a relatively flat lying asphalt concrete parking lot for a shopping compelx and grocery store. The surrounding land use is mixed commercial and residental.
7. The soil conditions observed to a maximum depth of 16.5 feet below the existing ground surface in our subsurface exploratory excavations generally consisted of (described relative to the existing ground surface): brown, dense, damp to wet, silty gravel and well graded gravel. Based on the seismic refraction survey (ReMi) performed near the site, the estimated thickness of the well graded gravel (GW) unit is approximately 40 to 50 feet and is underlain by denser fluvial deposited material.
8. Our field and laboratory test data indicates that the soil underlying the project site generally consists of the following geotechnical engineering material properties: dense, damp, and high bearing capacity, moderate lateral residence, with very low expansion potential (volume change).
9. At the time of our subsurface site investigation, groundwater was encountered at a depth of approximately 15 feet bgs.

5 RECOMMENDATIONS

The following geotechnical engineering recommendations are based on our understanding of the project as currently proposed, our field observations, the results of our laboratory testing program, engineering analysis, and our experience in the area. Because excavation, fill placement, and the construction of occupied structures are not proposed as a part of this project, we are not providing detailed geotechnical recommendations addressing aspects of construction typically addressed, such as stripping and grubbing, the placement of fills or recommendations regarding asphalt placement, concrete slab-on-grade construction, or similar topics typically included in geotechnical reports. We can provide recommendations addressing these topics, if requested.

5.1 GRADING

Because the site improvements will be mainly limited to the AC parking lot, our grading recommendations presented herein are limited to underground utility trenches, construction de-dewatering, subsurface drainage, review of construction plans, and construction quality assurance/quality control (QA/QC) monitoring. Our grading recommendations are presented below.

5.1.1 Temporary Excavations

All temporary excavations must comply with applicable local, state and federal safety regulations, including the current Occupational Safety and Hazards Administration (OSHA) excavation and trench safety standards. Construction site safety is the responsibility of the contractor, who is solely responsible for the means, methods, and sequencing of construction operations. Under no circumstances should the findings, conclusions and recommendations presented herein be inferred to mean that H&K is assuming any responsibility for temporary excavations, or for the design, installation, maintenance, and performance of any temporary shoring, bracing, underpinning or other similar systems. H&K can provide observation of the subsurface conditions revealed in temporary excavations and recommendations for shoring systems, if requested.

5.1.2 Underground Utility Trenches

Underground utility trenches should be excavated and backfilled as described below for each trench zone as shown in the figure below.

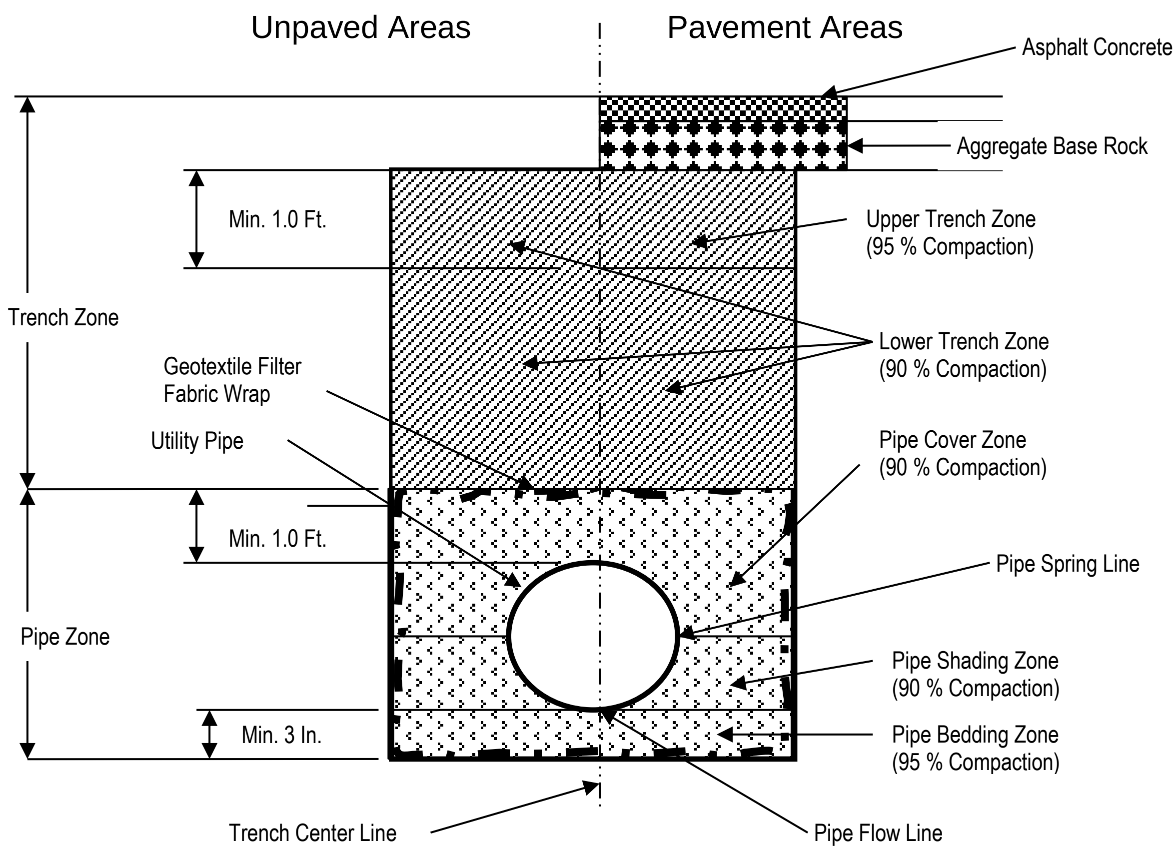
1. **Trench Excavation Equipment:** H&K anticipate that the contractor will be able to excavate shallow (up to 3 feet bgs) underground utility trenches with a Case 580 Backhoe or equivalent, however, deeper utility trenches (6-feet or greater) may require larger equipment.
2. **Trench Shoring:** All utility trenches that are excavated deeper than 4 feet below the surrounding ground surface are required by the California Occupational Safety

and Health Administration (OSHA) to be shored with bracing equipment or sloped back to an appropriate slope gradient prior to being entered by any individuals.

3. **Trench Dewatering:** H&K does not anticipate that the proposed shallow underground utility trenches will encounter shallow groundwater. However, if the utility trenches are excavated during the winter rainy season, then shallow or perched ground water may be encountered. The earthwork contractor may need to employ de-watering methods as discussed in Section 5.1.3 in order to excavate, place and compact the trench backfill materials.

4. **Pipe Zone Backfill Type and Compaction Requirements:** The backfill material type and compaction requirements for the pipe zone which includes the bedding zone, shading zone and cover zone as shown in the Figure 5.1.2 are described below.

FIGURE 5.1.2. TYPICAL TRENCH BACKFILL ZONES



Not To Scale

- **Pipe Zone Backfill Material Type:** Trench backfill used within the pipe zone which includes the bedding zone, shading zone and cover zone should consist of ¾-inch minus, washed, and crushed rock. The crushed rock particle size gradation should meet the following requirements (percents are expressed as dry weights using ASTM D422 test method): 100 percent passing the 3/4 inch sieve, 80 to 100 percent passing the ½ inch sieve, 60 to 100 percent passing the 3/8 inch sieve, 0 to 30 percent passing the No. 4 sieve, 0 to 10 percent passing the No. 8 sieve, and 0 to 3 percent passing the No. 200 sieve. If ground water is encountered within the trench during construction or if it is expected to rise during the rainy season to a elevation that will infiltrate the pipe zone within the trench, then the pipe zone material should be wrapped with a minimum 6 ounce per square yard, non-woven, geotextile filter fabric such as Mirafi 140 or equivalent should be used.. The geotextile seam should be located along the trench centerline and have a minimum 1-foot overlap.
- **Pipe Bedding Zone Compaction:** Trench backfill soil placed in the pipe bedding zone (beneath the utilities) should be a minimum 3-inches thick, moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.
- **Pipe Shading Zone Compaction:** Trench backfill soil placed within the pipe-shading zone (above the bedding zone and to a height of one pipe radius length above the pipe spring line) should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. The pipe shading zone backfill material should be shovel sliced to remove voids and to promote compaction.
- **Pipe Cover Zone Compaction:** Trench backfill soil placed within the pipe cover zone (above the pipe shading zone to one foot over the pipe top surface) should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

5. **Trench Zone Backfill And Compaction Requirements:** The trench zone backfill materials consists of both lower and upper zones as discussed below.

   - **Trench Zone Backfill Material Type:** Soil used as trench backfill within the lower and upper intermediate zones as shown on the preceding figure should consist of non-expansive soil with a plasticity index (PI) of less than or equal to 15 (PI ≤ 15) based on ASTM D4318 and should not contain rocks greater than 3 inches in greatest dimension.
   - **Lower Trench Zone Compaction:** Soil used to construct the lower trench zone backfills should be uniformly moisture conditioned to within

0 to 4 percentage points of the ASTM D1557 optimum moisture content, placed in maximum 12-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

- **Upper Trench Zone Compaction (Road And Parking Lot Areas):** Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 8-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.
- **Upper Trench Zone Compaction (Non-Road And Non Parking Lot Areas):** Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 to 2 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 6-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

6. **CQA Testing And Observation Engineering Services:** The moisture content, dry density, and relative percent compaction of all engineered utility trench backfills should be tested by project engineer's field representative during construction to evaluate whether the compacted trench backfill material meet or exceed the minimum compaction and moisture content requirements presented in this report. The earthwork contractor shall assist the project engineer's field representative by excavating test pads with the on-site earth moving equipment.

   - **Compaction Testing Frequencies:** The field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 5.1.2 or as modified by the project engineer to better suit the site conditions.

Table 5.1.2, Minimum Testing Frequencies For Utility Trench Backfill

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 500 CY (1) Or Material Change (2)
D2922	Nuclear Moisture Content	1 per 100 LF per 12-Inch-Thick Compacted Backfill Layer (3) The maximum loose lift thickness shall not exceed 12-inches prior to compacting.
D3017	Nuclear Density	1 per 100 LF per 12-Inch-Thick Compacted Backfill Layer (3) The maximum loose lift thickness shall not exceed 12-inches prior to compacting.

Notes: (1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) CY = cubic yards.
(3) Whichever criteria provide the greatest number of tests

5.1.3 Utility Trench Construction De-watering

H&K does not anticipate the need to perform de-watering of the site during utility installation, but could encounter groundwater during deep foundation construction. The earthwork contractor should be prepared to de-water the drilled foundation excavations and any other excavations if perched water or the groundwater table are encountered during construction and prior to placement of concrete.

5.1.4 Soil Corrosion Potential

The selected materials used for constructing underground utilities should be evaluated by a corrosion engineer for compatibility with the onsite soil and groundwater conditions. H&K perform minimal testing to determine the corrosion potential of the on site shallow soils that are anticipated to be in contact with the underground pipes and concrete structures associated with the improvements. The corrosion test results are presented in Table 5.1.4-1 below. Based on these limited tests (i.e., Redox, pH, resistivity, chloride, and sulfate) it is our opinion that the corrosion potential of the soil is mildly corrosive. Buried iron and steel materials should be properly protected against corrosion depending on the critical nature of the structure. H&K recommends that the owner contract with a corrosion engineer to evaluate the corrosion potential of the soil relative to the materials to be used to construct the associated underground utilities. Appendix D presents the laboratory test report for these soil samples.

Table 5.1.4-1 Summary of Corrosion Potential Laboratory Test Data

Boring No.	Sample No.	Sample Depth. (ft)	Test No.	Description	Test Result
B16-1	L-1-2-2	5	ASTM D1498	Redox	330 mV
			ASTM D4327	Chloride	<15 mg/kg
				Sulfate	<15 mg/kg
			ASTM D4658M	Sulfide	Not Analyzed
			ASTM D4972	PH	7.83
			ASTM G57	Resistivity	12,000 ohms-cm

1) mg/Kg = milligrams per kilograms
2) ohms-cm = ohom-centimeters
3) mV = millivolts

5.1.5 Subsurface Groundwater Drainage

The contractor should expect to encounter groundwater in deep foundations (greater than 13 feet bgs) and during the wet weather season construction. The contractor may need to dewater the deep foundations prior to placing concrete, or pump the concrete using a tremie pipe set at the bottom of the deep foundation excavation to displace the water as the concrete is placed. If necessary, H&K could observe the conditions and provide site-specific de-watering recommendations.

5.1.6 Project Plan Review and Construction Monitoring

Construction quality assurance includes review of plans and specifications and performing construction monitoring as described below.

1. H&K should be allowed to review the final earthwork grading improvement plans prior to commencement of construction to determine whether our recommendations have been implemented, and if necessary, to provide additional and/or modified recommendations.
2. H&K should be allowed to perform CQA monitoring of all earthwork grading performed by the contractor to determine whether our recommendations have been implemented, and if necessary, to provide additional and/or modified recommendations.
3. Our experience, and that of our engineering profession, clearly indicates that during the construction phase of a project the risks of costly design, construction and maintenance problems can be significantly reduced by retaining the design geotechnical engineering firm to review the project plans and specifications and to provide geotechnical engineering CQA observation and testing services. If H&K is not retained to provide geotechnical engineering CQA services during the construction phase of the project, then H&K will not be responsible for geotechnical engineering CQA services provided by others nor any aspect of the project that fails to meet your or a third party's expectations in the future.

5.2 STRUCTURAL IMPROVEMENTS

H&K's structural improvement design criteria recommendations include: seismic design parameters, and CIDH pier foundations.. These recommendations are presented here after.

5.2.1 Seismic Design Parameters

H&K developed the code-based seismic design parameters in accordance with Section 1613 and 1614 of the 2013 CBC and the USGS, *U.S. Seismic "DesignMaps" Web Application, Version 3.1.0.* The internet based application can be found at (https://geohazards.usgs.gov/secure/designmaps/us/) and is used for determining seismic design values from ASCE/SEI-7-10 (erratum released July, 2013), the 2012 International Building Code (2013 CBC). The spectral acceleration, site class, site coefficients and adjusted maximum considered earthquake (MCE) spectral response acceleration, and design spectral acceleration parameters are presented in the following Table 5.2.1.

Table 5.2.1-1 2013 CBC Seismic Design Parameters

Description	Value	Reference
Latitude	39.747 deg	Google Earth
Longitude	-122.188 deg	Google Earth
Site Coefficient, F_A	1.258	*2013 CBC, Table 1613.5.3(1), USGS, USSDM, v 3.1.0, 2013*
Site Coefficient, F_V	1.790	*2013 CBC, Table 1613.5.3(2), USGS, USSDM, v 3.1.0, 2013*
Site Class (Stiff Soil Profile)	D	*2013 CBC, Section 1613.5.2, Table 1613.5.2*
Short (0.2 sec) Spectral Response, S_S	0.677 *g*	*2013 CBC, Figure 1613.5(3), USGS, USSDM, v 3.1.0, 2013*
Long (1.0 sec) Spectral Response, S_1	0.305 *g*	*2013 CBC, Figure 1613.5(4), USGS, USSDM, v 3.1.0, 2013*
Short (0.2 sec) MCE Spectral Response, S_{MS}	0.852 *g*	*2013 CBC, Section 1613.5.3, USGS, USSDM, v 3.1.0, 2013*
Long (1.0 sec) MCE Spectral Response, S_{M1}	0.546 *g*	*2013 CBC, Section 1613.5.3, USGS, USSDM, v 3.1.0, 2013*
Short (0.2 sec) Design Spectral Response, S_{DS}	0.568*g*	*2013 CBC, Section 1613.5.4, USGS, USSDM, v 3.1.0, 2013*
Long (1.0 sec) Design Spectral Response, S_{D1}	0.364 *g*	*2013 CBC, Section 1613.5.4, USGS, USSDM, v 3.1.0, 2013*

deg = degrees
CBC = California Building Code
MCE = *Maximum Considered Earthquake*
sec = second
USSDM = *U.S. Seismic Design Maps*
USGS = United States Geological Survey

5.2.1 CIDH Pier Foundation Systems

H&K recommends that the foundation systems for the proposed PV structures utilize CIDH piers. Based on the dense silty gravel (GM) and sandy gravel (GM) soils encountered in the borings, H&K recommends the following.

1. H&K recommends for design of the CIDH piers that the soil conditions onsite be modeled as consisting of an approximate 5-foot-thick layer of medium dense silty gravel possessing an in place wet unit weight of 125 pounds per cubic foot (pcf), an internal friction angle of 35 degrees, and a cohesion of 500 pounds per square foot (psf). Below this depth, we recommend that the soil be modeled as sandy gravel possessing an in place wet unit weight of 135 pcft, an internal friction angle of 35 degrees, and a cohesion of 0 psf (i.e., negligible or unreliable cohesion).
2. H&K recommends that the upper two feet of soil be ignored for design of both axial bearing and uplift CIDH pier capacities and for design of lateral resistance capacities of the CIDH piers. However, the weight of the upper 2 feet of soil can be considered when calculating the friction and lateral resistance of the soil below depths of 2 feet.
3. H&K recommends for CIDH pier design methods employing lateral bearing approaches, such as the traditional CBC approach for constrained or non-constrained foundations, that an allowable lateral bearing (passive) pressure of 300 pounds per square foot, per foot of depth below the ground surface, be used for long-duration loads. For short term, dynamic loading such as would result from wind or seismic events, an allowable lateral bearing (active) pressure of 600 pounds per square foot, per foot of depth may be used. The use of this relatively high value assumes that the proposed PV structures would not be adversely impacted by an approximate ½-inch displacement at the ground surface due to short term lateral loads. If this magnitude of displacement is considered excessive for the proposed installation, then the recommended values above should be reduced by 50 percent or a more detailed foundation analysis including evaluation of lateral deflection should be performed.
4. Our experience has revealed that the CBC constrained and non-constrained equations are often conservative for CIDH piers and drilled shafts, and do not provide an estimate of pier deflection under lateral loading. We can provide a more detailed review of pier performance under lateral loading, including an estimate of deflection, if requested, once pier reactions and design has been established.
5. For depths greater than 6 feet below the ground surface, we recommend that an allowable end bearing capacity of 4,000 pounds per square foot be used for CIDH pier design. This value may be increased by a factor of 1.33 for transient or dynamic loads such as wind or seismic loads. In order to utilize end bearing values for CIDH pier design, the excavations must be cleaned thoroughly with a spin bucket capable of removing loose material from the bottom of the shaft. If

end bearing is used for the design, skin friction should be considered as an additional factor of safety.

6. The structural engineer should design the pier dimension, reinforcing steel, and connections between the foundation system and the structure.

6 REFERENCES

The following presents the references cited in this report:

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7 LIMITATIONS

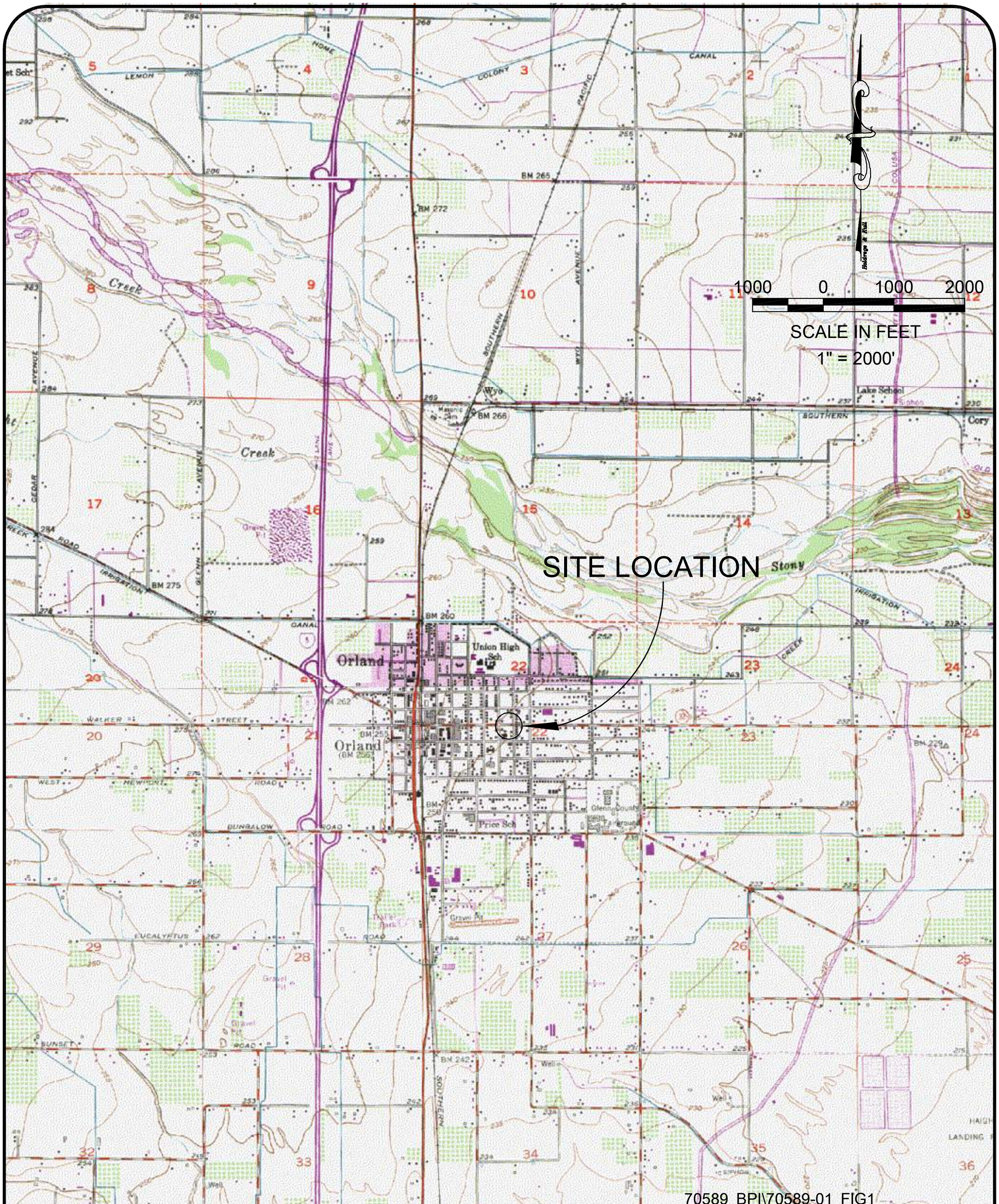
The following limitations apply to the findings, conclusions and recommendations presented in this report:

1. This report was prepared to provide information to Alternatives Energy Solutions for design purposes.
2. This report should not be relied upon without review by H&K if a period of 24 months elapses between the issuance report date shown on the signature and stamp page of this report and the date when construction commences.
3. Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in northern California. No warranty is either expressed or implied.
4. H&K provided engineering services for the site project consistent with the work scope and contract agreement presented in our proposal and agreed to by our client. The findings, conclusions and recommendations presented in this report apply to the conditions existing when H&K performed our services and are intended only for our client, purposes, locations, time frames, and project parameters described herein. H&K is not responsible for the impacts of any changes in environmental standards, practices, or regulations subsequent to completing our services. H&K does not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of our client unless noted otherwise. Any reliance on this report by a third party is at the party's sole risk.
5. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report should be considered invalid by all parties. The validity of the conclusions and recommendations presented in this report can only be made by H&K; therefore, H&K should be allowed to review all project changes and prepare written responses with regards to their impacts on the conclusions and recommendations presented in this report. However, additional fieldwork and laboratory testing may be required for H&K to develop any modifications to the recommendations presented in this report. The cost to review project changes and perform additional fieldwork and laboratory testing necessary to modify the recommendations presented in this report is beyond the scope-of-services presented in this report. Any additional work will be performed only after receipt of an approved scope-of-work, budget, and written authorization to proceed.
6. The analyses, conclusions an recommendations presented in this report are based on the site conditions as they existed at the time H&K performed the surface and subsurface field investigations. H&K assumed that the subsurface soil and groundwater conditions encountered at the location of the exploratory borings are generally representative of the subsurface conditions throughout the

entire project site. However, if the actual subsurface conditions encountered during construction are different than those described in this report, then H&K should be notified immediately so that an H&K representative can review these differences and, if necessary, modify the recommendations presented in this report.

7. The elevation or depth to the groundwater table underlying the project site may differ with time and location. Therefore, the depth to the groundwater table encountered in our exploratory borings is only representative of the specific time and location where it was observed.
8. The project site map shows the approximate locations of the H&K exploratory borings and/or trenches as determined by pacing distances from identifiable site features; therefore, their locations should not be relied upon as being exact nor located with the accuracy of a California licensed land surveyor.
9. Our geotechnical investigation scope-of-services did not include an evaluation of the project site for the presence of hazardous materials. Although, H&K did not observe the presence of hazardous materials at the time of our field investigation all project personnel should be careful and take the necessary precautions should hazardous materials be encountered during construction.
10. Our geotechnical investigation scope-of-services did not include an evaluation of the project site for the presence of mold nor for the future potential development of mold at the project site. If an evaluation of the presence of mold and/or for the future potential development of mold at the site is desired, then the property owner should contact a consulting firm specializing in these types of investigations. Holdrege & Kull does not perform mold evaluation investigations.
11. Our experience and that of the civil engineering profession clearly indicates that during the construction phase of a project the risks of costly design, construction and maintenance problems can be significantly reduced by retaining the design geotechnical engineering firm to review the project plans and specifications and to provide geotechnical engineering CQA observation and testing services. Upon your request H&K will prepare a CQA geotechnical engineering services proposal that will present a work scope, tentative schedule and fee estimate for your consideration and authorization. If H&K is not retained to provide geotechnical engineering CQA services during the construction phase of the project, then H&K will not be responsible for geotechnical engineering CQA services provided by others.

FIGURES:



HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

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CHICO, CA 95928
(530) 894-2487 FAX 894-2437

SITE LOCATION MAP
NORTH STATE GROCERY, ORLAND CENTER
ORLAND, GLENN COUNTY, CALIFORNIA

PROJ NO.: 70589-01
DATE: MAY,2016
FIGURE NO.: 1

70589__BPI\70589-01__FIG2

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS
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CHICO, CA 95928
530-894-2487 FAX 894-2437

SITE SKETCH AND EXPLORATORY BORING LOACTIONS
NORTH STATE GROCERY ORLAND CENTER
ORLAND, GLENN COUNTY, CALIFORNIA

DRAWN BY: SDC	FIGURE NO.: 2
CHECKED BY: SDC	
PROJECT NO.: 70589-01	
DATE: MAY,2016	

APPENDIX A:

Proposal for Geotechnical Engineering Services for the North State Grocery, Orland Center, Photovoltaic Installation (fee and contract agreement sections excluded)

March 10, 2016
Proposal No.: PC16.050

Mr. Kevin Berryhill
Bright Power, Inc.
PO Box 10637
Napa, California 94581
kevin@bpi-power.com

REFERENCE: ***North State Grocery Photovoltaic Installation***
35 E. Walker Street
Orland, Glenn County, California

SUBJECT: ***Proposal for Geotechnical Engineering Services***

Dear Mr. Berryhill,

In accordance with your request, Holdrege & Kull (H&K) prepared this proposal to provide geotechnical engineering investigation and consulting services for the development of the above referenced proposed photovoltaic (PV) carport canopy ~~and ground mount~~ installation at the North State Grocery (NSG) Orland facility, at the above referenced address. H&K will perform the appropriate geotechnical investigation in accordance with the requirements of the 2013 California Building Code (CBC) guidelines and prepare a geotechnical engineering investigation report presenting the findings, conclusions, and recommendations for the proposed improvements. The following presents our understanding of the project and our proposed engineering services.

1.0 PROJECT DESCRIPTION

H&K assumes the proposed carport array will utilize a T-style array with a center column and cantilevered beams that support the purlin and PV modules and ground mount arrays will utilize vibration driven piles typical of a Cobra Fixed Tilt System or a surface application using a Cobra Ballasted Rack System. Based on previous PV carport projects, H&K assumes a standardized foundation plan utilizing cast-in-drilled hole (CIDH) has been prepared that requires confirmation of the subsurface soil conditions. The CIDH piers will support tube steel columns and the canopy. As such H&K will perform a subsurface drilling investigation to classify the soil encountered below the proposed pier and pile locations.

2.0 SCOPE OF SERVICES

H&K proposes to perform the following tasks as basic services with no other additional services included: Task 1 Site Investigation and Laboratory Testing, Task 2 Data Analysis, Task 3 Report Preparation. Each task is described in the following:

exploratory boring will be backfilled immediately after logging and sampling activities are completed.

H&K' field engineer/geologist will collect both relatively undisturbed and disturbed soil samples from each exploratory boring. Relatively undisturbed soil samples will be collected with a standard penetration test (SPT) sampler and a 2.5-inch-diameter (inside diameter) split-spoon barrel sampler equipped with brass liner tubes. Generally, soil samples will be collected at the following depths below the existing ground surface: 0 feet, 2.0 feet, 5 feet, 10 feet, and continuing on five foot intervals, or change in geologic material, until the boring is terminated. Additional soil samples may be collected and/or the sample intervals may be changed depending upon the soil conditions encountered. The soil samples will be labeled, sealed, and transported to our laboratory facility where selected samples will be tested to determine their engineering material properties. If the groundwater table is encountered, the depth to groundwater below the existing ground surface will be measured.

H&K will perform an in-situ shear-wave velocity profile of the upper 30 meters of the site using SeisOpt® ReMi™ Vs30 Method for shear-wave profiling. The shear wave velocity data will be used to determine a Site Class and seismic design parameters in accordance with chapter 16 of the 2013 CBC, and for evaluating the liquefaction potential of the subsurface soil. Each seismic survey line will include 12 geophones on approximate 8-meter spacing, for a total seismic line length of 96 meters. A 48-channel, microprocessor control signal enhancement seismograph will be used to record ambient seismic noise, or micro-tremors, which are constantly being generated by cultural and natural noise. Additional ambient noise will be initiated from vehicles and during exploratory excavations on site.

2.1.3 Laboratory Testing Investigation

H&K will perform laboratory tests on selected soil samples to determine their engineering material properties. All laboratory tests will be performed consistent with the guidelines of the American Society for Testing and Materials (ASTM). The ASTM soil characterization tests may include:

- D2487 & D2488, Unified Soil Classification System, Description Visual Method
- D2937 & D2216, Density and Moisture Content
- D422, Particle Size Distribution, Sieve and Hydrometer Analysis
- D2166 Unconfined Shear Strength
- D3080, Direct Shear Strength
- D2166, Unconfined Compressive Strength
- D4318, Atterberg Plasticity Indices
- D4829, Expansion Index

~~and provide periodic to daily construction status reports of the contractors progress. The cost proposed with performing earthwork CQA is an ideal estimate at this time as actual plans, specifications, finished grades, and contractor schedule are not prepared or available at this time. The cost to perform CQA services is directly dependent on the contractor and their approach, sequencing, effort, and craftsmanship. H&K can also provide construction materials testing and special inspection services and prepare a cost estimate for such services once the plans and specification are complete. H&K can prepare a contract amendment to include these services following approval of the final plans and specification and selection of a construction contractor.~~

3.0 SCHEDULE

H&K's proposed work schedule is based on our present and expected workload. H&K is prepared to commence work on this project following receipt of a sign contract and notice to proceed. H&K can provide verbal preliminary design recommendations immediately following the site investigation based on the field investigation data; however, the final recommendations will be developed from both the field and laboratory data. Therefore, the final recommendations will govern the design. H&K estimates that the final report can be completed within 4 weeks following receipt of the signed contract and a notice to proceed, weather and site access permitting.

The time required to complete our geological investigation field work may be increased as a result of encountering unforeseen subsurface conditions, adverse weather conditions, soil stability, property access problems, or scheduling of exploratory equipment.

4.0 COST ESTIMATE

H&K proposes to perform the geological and geotechnical investigations and prepare the reports on a fixed cost lump sum basis of $, in accordance with the professional services agreement task order. This fee includes the cost of a drill rig and operator. Invoices will be generated on a monthly basis; terms of payment are net 30 days. Full payment is due upon completion of the work and issuance of the report. This cost associated with this scope of service is valid for a period of 60 days from the date of this proposal.

This cost estimate may require modification if unusual or unexpected site conditions are encountered which significantly change the work scope and increase the associated costs, if the client requests an expansion of the work scope, or if Glenn County or the City of Orland requires the purchase of any additional permits in order to complete the site investigation. H&K will not perform additional work outside the scope of services presented above until a written authorization to proceed and an approved budget augmentation is received.

5.0 CLOSING

Holdrege & Kull appreciates the opportunity to provide you with a proposal on this important project. If you should have questions or comments, please do not hesitate to contact the undersigned at (530) 894-2487.

Sincerely,

Holdrege & Kull

Shane D. Cummings, PG, CHG, CEG
Operations Manager/Engineering Geologist

APPENDIX B:

Important Information About Your Geotechnical Engineering Investigation Report (Presented with permission of ASFE, Copyright 2004)

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

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

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
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e-mail: info@asfe.org www.asfe.org

APPENDIX C:

Exploratory Boring Logs

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

EXPLORATORY BORING LOG

48 BELLARMINE CT., SUITE 40, CHICO, CA 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. **B16-1**

Project Name: BPi-NSG ORLAND | **Project No.:** 70589-01 | **Task:** 01 | **Date Start:** 04/05/16

Location: 35 WALKER ST. ORLAND | **Ground Elev. (Ft. AMSL):** -- | **Date Finish:** 04/05/16 | **Sheet:** 1 **Of** 1

Logged By: SHANE CUMMINGS | **Drilling Cmpny:** PC EXPLORATION | **Drill Rig Type:** GEFCO SS15

Driller: SCOTT FLEMING | **Drilling Method:** HOLLOW STEM AUGER (HSA) | **Hammer Type:** 140 LBF. AUTOMATIC TRIP HAMMER

Boring Dia. (In.): 7.25 | **Total Depth (Ft.):** 15.7 | **Backfill or Well Casing:** --

Ground Water Information

Date	04/05/16			
Time (24 Hour)	09:10			
Depth (Ft.)	15			

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Sample Interval And Symbol	Well Construction Details	Graphic Log	Soil And/Or Rock Material Descriptions
08:25			HSA			0				ASPHALT & AGGREGATE BASE (0.3' AC)
						1				(GM) SILTY GRAVEL; FLD. EST.: 40% FINE GRAVEL, 35% FINE TO MEDIUM SAND, 25% LOW PLASTICITY FINES; BROWN (7.5YR 4/2); MEDIUM DENSE; DAMP.
						2				
07:55		7	2.5 SS							
		15				3				
	>4.5	15		1.0	L-1-1-1	4				
			HSA							
08:50		9	2.5 SS			5				(GW) SANDY GRAVEL; FLD. EST.: 60% FINE TO COARSE GRAVEL, 35% FINE TO COARSE SAND, 5% LOW PLASTICITY FINES; BROWN (7.5YR 5/2); DENSE; DAMP.
		13			L-2-2-2	6				
	>4.5	19		1.3	L-2-1-2					
			HSA			7				
						8				
						9				
08:55										
08:55		32	2.5 SS			10				INCREASE COARSE GRAVEL
		21				11				
	>4.5	35		1.3	L-3-1-1					
			HSA			12				
						13				
						14				
09:10		49	2.5 SS			15				GROUNDWATER AT 15 FEET NO RECOVERY
	--	50/2	--	0	--	16				TERMINATED AT 15.7 FEET
						17				
						18				
						19				
						20				

NOTES:

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

EXPLORATORY BORING LOG

48 BELLARMINE CT., SUITE 40, CHICO, CA 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. B16-2

Project Name: BPi-NSG ORLAND	**Project No.:** 70589-01	**Task:** 01	**Date Start:** 04/05/16	
Location: 35 WALKER ST. ORLAND	**Ground Elev. (Ft. AMSL):** --		**Date Finish:** 04/05/16	**Sheet:** 1 **Of** 1
Logged By: SHANE CUMMINGS	**Drilling Cmpny:** PC EXPLORATION	**Drill Rig Type:** GEFCO SS15		
Driller: SCOTT FLEMING	**Drilling Method:** HOLLOW STEM AUGER (HSA)	**Hammer Type:** 140 LBF. AUTOMATIC TRIP HAMMER		
Boring Dia. (In.): 7.25	**Total Depth (Ft.):** 16.5	**Backfill or Well Casing:** --		

Ground Water Information

Date	04-05-16			
Time (24 Hour)	10:05			
Depth (Ft.)	15			

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Sample Interval And Symbol	Well Construction Details	Graphic Log	Soil And/Or Rock Material Descriptions
09:35			HSA			0				ASPHALT & AGGREGATE BASE (0.3' AC)
						1				(GM) SILTY GRAVEL; FLD. EST.: 40% FINE GRAVEL, 35% FINE TO MEDIUM SAND, 25% LOW PLASTICITY FINES; BROWN (7.5YR 4/2); MEDIUM DENSE; DAMP.
						2				
						3				
						4				(GW) SANDY GRAVEL; FLD. EST.: 60% FINE TO COARSE GRAVEL, 35% FINE TO COARSE SAND, 5% LOW PLASTICITY FINES; BROWN (7.5YR 5/2); DENSE; DAMP.
09:45		9	SPT			5				
		18								
	>4.5	19		1.0	SPT-1	6				
			HSA							
						7				
						8				
						9				
09:55		19	SPT			10				
		19								
	>4.5	21		1.2	SPT-2	11				
			HSA							
						12				
						13				
						14				
10:00		6	SPT			15				GROUNDWATER AT 15 FEET
		12								
	>4.5	13		1.1	SPT-3	16				
						17				BORING TERMINATED AT 16.5 FEET
						18				
						19				
						20				

NOTES:

APPENDIX D:

Laboratory Test Sheets

Holdrege & Kull Laboratory
Cerco Analytical Laboratory

HOLDREGE & KULL

CONSULTING ENGINEERS • GEOLOGISTS

Moisture & Density

ASTM D2216 & D2937

DSA File #:

DSA Appl #:

Project No.:	70589-01	Project Name:	Bpi NSG Orland	Date:	4/11/2016
Lab No.:	15-16-069	Performed By:	MLH	Checked By:	MLH

SAMPLE LOCATION DATA										
Boring/Trench No.	Units	B16-1	B16-1							
Sample No.		L-1-1-1	L-2-1-2							
Depth Interval	(ft.)	2.5	5							
Sample Description		Very Dark Greyish Brown (2.5Y 3/2) Well Graded Sand with Gravel	Olive Grey (5Y 4/2) Well Graded Sand with Silt and Gravel							
USCS Symbol										
SAMPLE DIMENSION AND WEIGHT DATA										
Sample Length	(in)	5.840	5.390							
Sample Diameter	(in)	2.390	2.380							
Sample Volume	(cf)	0.0152	0.0139							
Wet Soil + Tube Wt.	(gr)	1148.84	1066.90							
Tube Wt.	(gr)	273.59	272.22							
Wet Soil Wt.	(gr)	875.25	794.68							
MOISTURE CONTENT DATA										
Tare No.		5	DE							
Tare Wt.	(gr)	117.62	259.17							
Wet Soil + Tare Wt.	(gr)	190.15	356.07							
Dry Soil + Tare Wt.	(gr)	188.09	353.36							
Water Wt.	(gr)	2.06	2.71							
Dry Soil Wt.	(gr)	70.47	94.19							
Moisture Content	(%)	2.9	2.9							
TEST RESULTS										
Wet Unit Wt.	(pcf)	127.3	126.3							
Moisture Content	(%)	2.9	2.9							
Dry Unit Wt.	(pcf)	123.7	122.7							
MOISTURE CORRECTION DATA										
Gauge Moisture	(%)									
K Value Correction Factor										
COMPACTION CURVE DATA (ASTM D698, ASTM D1557, or CAL216)										
Test Method										
Curve No.										
Max Wet Unit Wt.	(pcf)									
Max Dry Unit Wt.	(pcf)									
Optimum Moisture	(%)									
Wet Relative Comp.	(%)									
Dry Relative Comp.	(%)									

HOLDREGE & KULL

(530) 478-1305 - Fax (530) 478-1019 - 792 Searls Ave.- Nevada City, CA 95959 - A California Corporation

Particle Size Distribution

ASTM D422

Project No.:	**70589-01**	Project Name:	**Bpi NSG Orland**	Date:	**4/11/2016**
Sample No.:	**L-1-1-1**	Boring/Trench:	**B16-1** Depth, (ft.): **2.5**	Tested By:	**SJS/MLH**
Description:	**Very Dark Greyish Brown (2.5Y 3/2) Well Graded Sand with Gravel**			Checked By:	**MLH**
Sample Location:	**0**			Lab. No.:	**15-16-069**

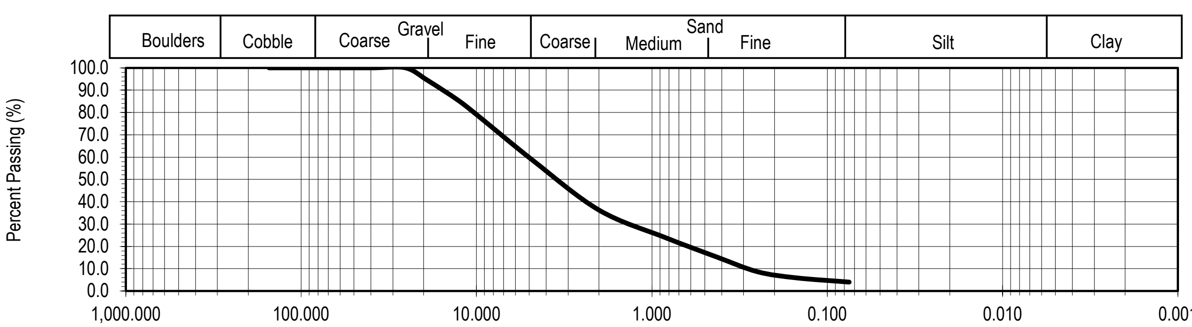
Sieve Size (U.S. Standard)	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Dry Weight on Sieve Retained On Sieve (gm)	Dry Weight on Sieve Accumulated On Sieve (gm)	Dry Weight on Sieve Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.4	0.00	0.0	785.1	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	785.1	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	785.1	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	785.1	**100.0**
1.0 Inch	1.0000	25.4	0.00	0.0	785.1	**100.0**
3/4 Inch	0.7500	19.1	43.70	43.7	741.4	**94.4**
1/2 Inch	0.5000	12.7	70.80	114.5	670.6	**85.4**
3/8 Inch	0.3750	9.5	60.40	174.9	610.2	**77.7**
#4	0.1870	4.7500	151.70	326.6	458.5	**58.4**
#10	0.0787	2.0000	173.98	500.6	284.5	**36.2**
#20	0.0335	0.8500	95.08	595.7	189.5	**24.1**
#40	0.0167	0.4250	70.09	665.8	119.4	**15.2**
#60	0.0098	0.2500	51.38	717.1	68.0	**8.7**
#100	0.0059	0.1500	21.59	738.7	46.4	**5.9**
#200	0.0030	0.0750	14.90	753.6	31.5	**4.0**

Cc = 1.35

Cu = 17.24

Hydrometer

Particle Size Gradation



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Particle Size Distribution

ASTM D422

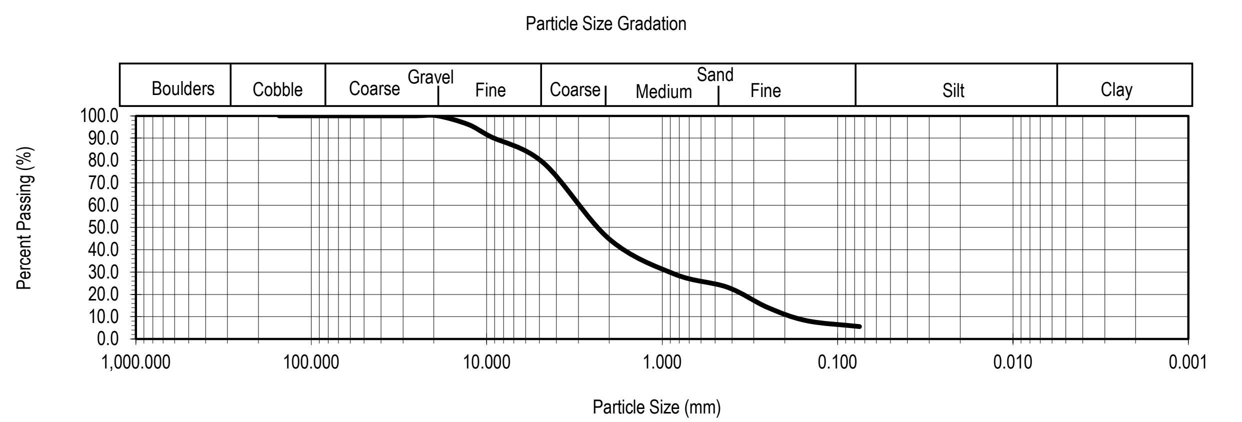
Project No.:	**70589-01**	Project Name:	**Bpi NSG Orland**	Date:	**4/11/2016**
Sample No.:	**L-2-1-2**	Boring/Trench:	**B16-1** Depth, (ft.): **5**	Tested By:	**SJS/MLH**
Description:	**Olive Grey (5Y 4/2) Well Graded Sand with Silt and Gravel**			Checked By:	**MLH**
Sample Location:	**0**			Lab. No.:	**15-16-069**

Sieve Size (U.S. Standard)	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Dry Weight on Sieve Retained On Sieve (gm)	Dry Weight on Sieve Accumulated On Sieve (gm)	Dry Weight on Sieve Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.4	0.00	0.0	1,604.4	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	1,604.4	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	1,604.4	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	1,604.4	**100.0**
1.0 Inch	1.0000	25.4	0.00	0.0	1,604.4	**100.0**
3/4 Inch	0.7500	19.1	0.00	0.0	1,604.4	**100.0**
1/2 Inch	0.5000	12.7	62.60	62.6	1,541.8	**96.1**
3/8 Inch	0.3750	9.5	86.10	148.7	1,455.7	**90.7**
#4	0.1870	4.7500	189.30	338.0	1,266.4	**78.9**
#10	0.0787	2.0000	548.23	886.2	718.2	**44.8**
#20	0.0335	0.8500	251.31	1,137.5	466.9	**29.1**
#40	0.0167	0.4250	94.37	1,231.9	372.5	**23.2**
#60	0.0098	0.2500	145.99	1,377.9	226.5	**14.1**
#100	0.0059	0.1500	94.59	1,472.5	131.9	**8.2**
#200	0.0030	0.0750	42.35	1,514.8	89.6	**5.6**

Cc = 1.42

Cu = 15.79

Hydrometer



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Chain of Custody

Page of

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775
CERCO analytical

Job No.	CU#	Client Project I.D.	Schedule	Standard TAT	Date Sampled	Date Due
			Analyte		4/5/16	4/27/16

Full Name Shane Cummings (sdcummings@handk.net)
Fax 530-894-2437
Phone 530-894-2487

Company Holdrege & Kull
Cell 530-362-0142 ☒

Sample Source commercial property H&K Project No. 70589-01

ASTM w/Brief Evaluation — **ANALYSIS**

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated		Brief Evaluation						
	B16-1-L1-2-2@5	4/5/16	0850	s	bag	1 gall	no	1	x	x	x	x	x		✔						

MATRIX
DW - Drinking Water
GW - Ground Water
SW - Surface Water
WW - Waste Water
Water
SL - Sludge
S - Soil
Product

ABBREVIATIONS
HB - Hosebib
PV - Petcock Valve
PT - Pressure Tank
PH - Pump House
RR - Restroom
GL - Glass
PL - Plastic
ST - Sterile

SAMPLE RECEIPT
Total No. of Containers
Rec'd Good Cond/Cold
Conforms to Record
Temp. a t Lab -°C
Sampler

	Date	Time
Relinquished By:		
Received By:		
Relinquished By:		
Received By:		
Relinquished By:		
Received By:		

Comments:
THERE IS AN ADDITIONAL CHARGE FOR METAL/POLY TUBES

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

18 April 2016

Job No. 1604113
Cust. No.12830

Mr. Shane Cummings
Holdrege & Kull
8 Seville Court, Suite 100
Chico, CA 95928

Subject: Project No.: 70589-01
Project Name: Commercial Property
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Mr. Cummings:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on April 13, 2016. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, the sample is classified as "mildly corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is none detected to 15 mg/kg.

The sulfate ion concentration is none detected to 15 mg/kg.

The pH of the soil is 7.83 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 330-mV which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

Client: Holdrege & Kull
Client's Project No.: 70589-01
Client's Project Name: Commercial Property
Date Sampled: 5-Apr-16
Date Received: 13-Apr-16
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 18-Apr-2016

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1604113-001	B16-1-L1-2-2 @ 5'	330	7.83	-	12,000	-	N.D.	N.D.

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	14-Apr-2016	14-Apr-2016	-	15-Apr-2016	-	14-Apr-2016	14-Apr-2016

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits