APPENDIX A

SUBSURFACE EXPLORATIONS:
CONSOLIDATED ENGINEERING
LABORATORIES GEOLOGICAL ENGINEERING
AND GEOLOGICAL HAZARDS STUDY
84-0318-A
Dated
January 31, 2017

GEOTECHNICAL ENGINEERING
AND GEOLOGIC HAZARDS STUDY

Ellis Creek Water Recycling Facility
Biomass to Biofuel Project
3890 Cypress Drive
Petaluma, California 94954

Prepared for:

City of Petaluma
Public Works and Utilities
202 N. McDowell Boulevard
Petaluma, California 94954

Prepared by:

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CEL Project No. 84-03818-A

November 23, 2016
Second Revised: January 31, 2017

City of Petaluma
Public Works and Utilities
202 N. McDowell Boulevard
Petaluma, California 94954

Attention: Mr. Dan Herrera, P.E., Associate Civil Engineer

Subject: Geotechnical Engineering and Geologic Hazards Study
Ellis Creek Water Recycling Facility
Biomass to Biofuel Project
3890 Cypress Drive, Petaluma, California 94954
CEL Project No. 84-03818-A

Dear Mr. Herrera:

Consolidated Engineering Laboratories (CEL) has completed the Geotechnical Engineering and Geologic Hazards Study for the Ellis Creek Water Recycling Facility, Biomass to Biofuel Project located at 3890 Cypress Drive, in Petaluma, Sonoma County, California. This report has been prepared based on your request, our discussion via email with you, and review of the project plans provided by you. Transmitted herewith are the results of our findings, conclusions, and recommendations for foundation, seismic design parameters, interior and exterior concrete slabs, site preparation, grading, foundation excavation, drainage, and utility trench backfilling. In general, the proposed improvements at the site are considered to be geotechnically as well as geologically feasible provided the recommendations of this report are implemented in the design and construction of the project.

Should you or members of the design team have questions or need additional information, please contact either of the undersigned at (925) 314-7100, or by e-mail at rshrestha@ce-labs.com or eswenson@ce-labs.com. We greatly appreciate the opportunity to be of service to City of Petaluma, Public Works and Utilities Department and to be involved in the design of this project.

Sincerely,
CONSOLIDATED ENGINEERING LABORATORIES.

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GEOTECHNICAL ENGINEERING AND GEOLOGIC HAZARDS STUDY

Project: **Ellis Creek Water Recycling Facility- Biomass to Biofuel Project**
Petaluma, California

Client: **City of Petaluma, Public Works Utilities**
Petaluma, California

1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of this study was to evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the Ellis Creek Water Recycling Facility, Biomass to Biofuel Project. This study provides recommendations for foundations, seismic design parameter, interior and exterior concrete slabs, site preparation, grading, foundation excavation, drainage, and utility trench backfilling. This study was performed in accordance with the scope of work outlined in our revised proposal dated September 30, 2016.

The scope of this study included the review of available previous geotechnical and geologic literature for the site, the drilling of several subsurface borings within the project site, laboratory testing of selected samples retrieved from the borings, engineering analysis of the accumulated data, and preparation of this report. The conclusions and recommendations presented in this report are based on the data acquired and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.2 Site Description

The proposed project site is located on Ellis Creek Water Recycling Facility with general address at 3890 Cypress Drive in City of Petaluma, Sonoma County, California, as shown on *Figure 1 - Site Vicinity Map*. The proposed structure/digester gas storage will be located in the southwest undeveloped part of the facility and a screw press building will be located on AC paved section on the east part of the facility, next to the existing Bio-filter building as shown on *Figure 2 - Developmental Site Plan*. Current elevation for the storage system and screw press building is at approximately 11 feet and 16 feet, respectively above mean sea level (MSL) based on the Project Site Plans provided by you. General topography of the site is relatively flat except for the depression area where the storage system will be.

The project area is bounded by the wetland/ponds on the south and east sides, and the rest of the area is surrounded by vacant land of the treatment facility area. The project site is located at approximately 38.2289° north latitude and 122.5866° west longitude.

1.3 Proposed Development

It is our understanding that the proposed development will consist of the construction of a Digester Gas Storage system and a Screw Press Building. Digester system is expected to be about 50 feet in diameter and 40 feet high in semi-spherical shape. Planned foundation for the system will consist of near grade, reinforced concrete mat slabs imparting more than 1000 psf to the surrounding soils. The screw press building will be a single story concrete building that will be structurally tied into the existing single story concrete Screw Press Building. In addition, there will be a few concrete pads for supporting vessel, control panel, burner, etc. The locations of the proposed developments are shown on Figure 2.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

The geologic and geotechnical literature pertaining to the site area was reviewed. These included various publications and maps issued by the United States Geological Survey (USGS), water agencies, previous geotechnical report, and other government agencies, as listed in the References section. Previous geotechnical engineering studies at this site (Integrated Geotechnical Study (2005), prepared by Fugro West Inc.), were utilized to supplement our investigation.

2.2 Field Exploration

A total of five borings were drilled at the site within the proposed building footprint on November 1, 2016 at the approximate locations shown on Figures 2 and 3. The borings were drilled to a maximum depth of approximately 50 feet below the existing ground surface within the proposed structure footprints using a truck mounted, Mobile CME-45 drill rig equipped with 4-inch solid flight augers and rotary wash.

A CEL representative visually classified the materials encountered in the borings according to the Unified Soil Classification System as the borings were advanced. Relatively undisturbed soil samples were recovered at selected intervals using a three-inch outside diameter Modified California split spoon sampler containing six-inch long brass liners, and a two-inch outside diameter Standard Penetration Test (SPT) sampler. The samplers were driven by means of a 140-pound and 70-pound safety hammers with an approximate 30-inch fall. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. All of the field blow counts recorded using Modified California (MC) split spoon sampler were converted in the final logs to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948), i.e., a factor of 0.65 with inner diameter of 2.5 inches. Therefore, all blow counts shown on the final boring logs are either directly measured (SPT sampler) or equivalent SPT (MC sampler) blow counts.

The boring logs with descriptions of the various materials encountered in each boring, a key to the boring symbols, and select laboratory test results are included in Appendix A. Ground surface elevations indicated on the soil boring logs were recorded from the Project Plans provided to us.

2.3 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are either presented on the boring logs, and/or are included in Appendix B. The following soil tests were performed for this study:

Dry Density and Moisture Content (ASTM D2216 and ASTM 2937) – In-situ dry density and/or moisture tests were conducted on sixteen samples to measure the in-place dry density and moisture content of the subsurface materials. These properties provide information for evaluating the physical characteristics of the subsurface soils. Test results are shown on the boring logs.

Atterberg Limits (ASTM D4318 and CT204) - Atterberg Limits test was performed on two samples of cohesive soils encountered at the site. Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, and help to evaluate the expansive characteristics of the soil and determine the USCS soil classification. Test results are presented in Appendix B, and on the boring logs.

Particle Size Analysis (Wet and Dry Sieve) and Hydrometer (ASTM D422, D1140, and CT202) - Sieve analysis tests were conducted on three selected samples to determine the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Test results are presented in Appendix B.

Direct Shear-Consolidated Undrained (Modified ASTM D3080M) - Direct shear tests were performed on one sample to determine the angle of internal friction and cohesion of the soils or rock materials. This data can be utilized in determining allowable bearing capacity, retaining wall design parameters, and strength characteristics of the materials. Direct shear specimens were saturated under a 100-psf surcharge for a period of 24 hours prior to testing.

Soil Corrosivity, Redox (ASTM D1498), pH (ASTM D4972), Resistivity (ASTM G57), Chloride (ASTM D4327), and Sulfate (ASTM D4327) - Soil corrosivity testing was performed to determine the effects of constituents in the soil on buried steel and concrete. Water-soluble sulfate testing is required by the UBC, CBC, and IBC.

3.0 GEOLOGIC AND SEISMIC OVERVIEW

3.1 Regional Geologic Setting

The site is located in the central portion of the northern Coast Ranges geomorphic province of California. The Coast Ranges extend from the Transverse Ranges in southern California to the Oregon border and are comprised of a northwest-trending series of mountain ranges and intervening valleys that reflect the overall structural grain of the province. The ranges consist of a variably thick veneer of Cenozoic volcanic and sedimentary deposits overlying a Mesozoic basement of sedimentary, metamorphic, and basic igneous Franciscan Formation and primarily marine sedimentary rocks of the Great Valley Sequence. East-dipping sedimentary rocks of the Coast Ranges are flanked on the east by sedimentary rocks of the Great Valley geomorphic province (Page, 1966).

The Geologic formation of the site is underlain by Pliocene and early Miocene age sedimentary rock. Most sediment has been eroded and washed from neighboring Pliocene age rocks of the Sonoma Volcanic formation that is thought to underlie alluvium at depth. These volcanic deposits of basalt, andesite, rhyolite and volcanic tuff are common in the Sonoma Mountains east of the site. The volcanics are thought to have intruded through the older Cretaceous to Jurassic age rocks of the Franciscan Formation between 5 and 20 million years ago. The older Franciscan Complex rocks of estimated age between 64 million and 180 million years are commonly found west of the Petaluma Plain in the coastal mountains and along the ridgeline of the Sonoma Mountains. Intermediate Pliocene age sedimentary deposits of both the non-marine Petaluma Formation and marine Wilson Grove Formation show that the area was subject to rising and falling sea level throughout the late Cenozoic era and into the Pleistocene. (Source: City of Santa Rosa, Geology and Soils; Geology and Ground Water in the Santa Rosa and Petaluma Area).

The Franciscan complex is composed of weakly to strongly metamorphosed greywacke (sandstone), argillite, limestone, basalt, serpentinite, chert and other rocks. This rock was accreted onto the edge of the North American continent during the long period of active subduction of the Pacific Plate beneath the North American Plate. The formation is derived from Jurassic oceanic crust and pelagic deposits that are overlain by Late Jurassic to Late Cretaceous sedimentary deposits. Metamorphic grade in this rock is highly variable which reflects the complicated history of the Franciscan.

Since the late Cenozoic era, subduction has been replaced by transform faulting along faults of the San Andreas System. There has also been major climate change and dramatic rising and lowering of sea level. Due to the complex geologic history of the area there is a wide variety of volcanic rocks and sedimentary rocks of varying metamorphic grade to be found in the region. These units are often juxtaposed along ancient fault contacts and the structure is complicated by not only ancient deformation, but by active fault deformation. Imprinted on this geology is the drainage pattern of the Petaluma Creek Watershed.

3.2 Local Geologic Setting

Non-marine, transitional-marine, and marine sedimentary rocks of the Petaluma Formation and Wilson Grove (formerly known as Merced) Formation are interbedded with the Sonoma Volcanics. The Petaluma Formation (Late Miocene to Late Pliocene) is largely confined to a one to two mile exposure along the southwestern portion of the Sonoma Mountains. It is composed of strongly folded continental and brackish water sedimentary rocks with interbeds of tuff. The Wilson Grove Formation (formerly the Merced Formation; Late Miocene to Late Pliocene) consists of a thick sequence of clastic fossiliferous sedimentary rocks interfingering to the east with the Petaluma Formation. It is the major water-bearing unit in the Santa Rosa Valley Groundwater Basin. It extends beneath the western hills, crops out along the western side of the valley from the Russian River (Wilson Grove) south towards Petaluma, and dips beneath the center of the valley. The mapped geologic units at the site are shown on *Figure 4 - Site Vicinity Geologic Map*. The site is shown to be underlain by majority of Quaternary Holocene Alluvium (Qha) deposits.

3.3 Geologic Evolution of the Northern Coast Ranges

The subject site is located within the tectonically active and geologically complex northern Coast Ranges, which have been shaped by continuous deformation resulting from tectonic plate convergence (subduction) beginning in the Jurassic period (about 145 million years ago). Eastward thrusting of the oceanic plate beneath the continental plate resulted in the accretion of materials onto the continental plate. These accreted materials now largely comprise the Coast Ranges. The dominant tectonic structures formed during this time include generally east-dipping thrust and reverse faults.

Beginning in the Cenozoic time period (about 25 to 30 million years ago), the tectonics along the California coast changed to a transpressional regime and right-lateral strike-slip displacements as well as thrusting were superimposed on the earlier structures resulting in the formation of northwest-trending, near-vertical faults comprising the San Andreas Fault System. The northern Coast Ranges were segmented into a series of tectonic blocks separated by major faults including the San Andreas, Rodgers Creek, Hayward, Lakeview, and Tolay. The project site is situated between the active Rodgers Creek and San Andreas faults, but no known active faults with Holocene movement (last 11,000 years) lie within the limits of the site. The site is not mapped within an Alquist-Priolo Earthquake Fault Zone.

3.4 Regional Faulting and Tectonics

Regional transpression has caused uplift and folding of the bedrock units within the Coast Ranges. This structural deformation occurred during periods of tectonic activity that began in the Pliocene and continues today. The site is located in a seismically active region that has experienced periodic, large magnitude earthquakes during historic times. This seismic activity appears to be largely controlled by displacement between the Pacific and North American crustal plates, separated by the San Andreas Fault zone located approximately 17.5 miles west of the site. This plate displacement produced regional strain that is concentrated along major faults of the San Andreas Fault System including the San Andreas, Rodgers Creek, and Hayward faults in this area, *Figure 5 - Regional Fault Map*. The proposed structure should be designed to resist deformation produced by such tectonic activity applying the relevant seismic design parameters as recommended in Section 6.2, Table 6.2.1 based on the current California Building Code.

4.0 SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

Digester Gas Tank Area

During our subsurface exploration program, we investigated the subsurface soils and evaluated soil conditions to maximum depths ranging from 10 feet to 50 feet. From our collected data, we conclude that where explored, the area of the proposed new structure is generally underlain by a layer of silty/sandy clay Topsoil to a depth of approximately three feet, underlain by a layer of stiff Fat Clay to a depth of approximately 7 to 12 feet, underlain by a layer of stiff Sandy Clay/Clayey Sand to an approximate depth of 17 feet, underlain by a layer of medium dense poorly graded sand and gravel to an approximate depth of 29 feet, underlain by alternating layers of lean clay to silty sand to sandy clay to silty sand to the maximum depth explored of 50 feet.

Test results of near-surface soil samples recovered in the uppermost 4.5 feet of the soil profile collected from Boring B-1 indicated a measured Liquid Limit of 59 and a corresponding Plasticity Index of 37. Based on these results, the near-surface soils are considered to have a high plasticity and a high expansion (shrink/swell) potential.

Screw Press Building

Subsurface soils encountered under the proposed screw press building generally consist of fills containing clayey sand/sandy clay with gravel to a depth of 7 to 10 feet. Underlying that are layers of various compositions of clay such as sandy clay and lean clay to the maximum depth explored of 20 feet.

Our interpretations of the subsurface geologic and soil conditions are presented in Figures 6a and 6b. Additional details of materials encountered in the exploratory borings are included in the boring logs in Appendix A, and laboratory test summaries are presented in Appendix B.

4.2 Groundwater Conditions

Groundwater was encountered during drilling at depths of approximately 5, 6 and 10 feet in borings B-1, B-2 and B-4 respectively. Review of The Department of Water Resources "Groundwater Levels for Station 382276N1225763W001 data, located approximately 2000 feet northeast of the project site indicated the highest water level measured on 2/24/1994, at 19.5 feet below the ground surface (Ground Elevation 23 feet). Based on this information we anticipated the groundwater level to be approximately 5 to 10 feet below existing ground at the project site. However, groundwater levels can vary in response to time of year, variations in seasonal rainfall, well pumping, irrigation, and alterations to site drainage. A detailed investigation of local groundwater conditions was not performed and is beyond the scope of this study.

Based on the review of previous geotechnical report prepared by Fugro (2003) the groundwater was recommended to consider at an Elevation +14 feet for all new facilities constructed in the Parcel A and Oxidation Pond 1 sites; for the Oxidation Ponds 9 and 10, the static design groundwater level was recommended as Elev. +5 feet. Under seismic loading, their recommendation for static groundwater level was to reduce by 2 feet to account for remote concurrences of high groundwater due to winter storms and the design seismic event.

4.3 Corrosion Testing

A sample collected from the upper two to four feet of the soil profile at Borings B-2 was tested to measure sulfate content, chloride content, redox potential, pH, resistivity, and presence of sulfides. Test results are included in Appendix B and are summarized on the following table.

Table 4.3.1: Summary of Corrosion Test Results

Soil Description	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	Redox (mV)	Resistivity (ohm-cm)	Sulfide	pH
Dark Gray Sandy CLAY	2 - 4	75	52	478	1,010	Negative	7.5

Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. Section 4.3.1 in American Concrete Institute (ACI) 318, as referenced by the CBC, provides the following evaluation criteria:

Table 4.3.2: Sulfate Evaluation Criteria

Sulfate Exposure	Water-Soluble Sulfate in Soil, Percentage by Weight or (mg/kg)	Sulfate in Water, ppm	Cement Type	Max. Water Cementitious Ratio by Weight	Min. Unconfined Compressive Strength, psi
Negligible	0.00-0.10 (0-1,000)	0-150	NA	NA	NA
Moderate	0.10-0.20 (1,000-2,000)	150-1,500	II, IP (MS), IS (MS)	0.50	4,000
Severe	0.20-2.00 (2,000-20,000)	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00 (20,000)	Over 10,000	V plus pozzolan	0.45	4,500

The water-soluble sulfate content was measured to be 75 mg/kg or 0.0075% by dry weight in the soil sample, suggesting the site soil should have negligible impact on buried concrete structures at the site. However, it should be pointed out that the water-soluble sulfate concentrations can vary due to the addition of fertilizer, irrigation, and other possible development activities.

Table 4.4.1 in ACI 318 suggests use of mitigation measures to protect reinforcing steel from corrosion where chloride ion contents are above 0.06 % by dry weight. The chloride content was measured to be 52 mg/kg or 0.0052% by dry weight in the soil sample. Therefore, the test result for chloride content does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures due to high concentration of chloride.

In addition to sulfate and chloride contents described above, pH, oxidation reduction potential (Redox), and resistivity values were measured in the soil sample. For cast and ductile iron pipes, an evaluation was based on the 10-Point scaling method developed by the Cast Iron Pipe Research Association (CIPRA) and as detailed in Appendix A of the American Water Works Association (AWWA) publication C-105, and shown on Table 4.3.3.

Table 4.3.3: Soil Test Evaluation Criteria (AWWA C-105)

Soil Characteristics	Points		Soil Characteristics	Points
Resistivity, ohm-cm, based on single probe or water-saturated soil box.			**Redox Potential, mV**	
<700	10		>+100	0
700-1,000	8		+50 to +100	3.5
1,000-1,200	5		0 to 50	4
1,200-1,500	2		Negative	5
1,500-2,000	1		**Sulfides**	
>2,000	0		Positive	3.5
PH			Trace	2
0-2	5		Negative	0
2-4	3		**Moisture**	
4-6.5	0		Poor drainage, continuously wet	2
6.5-7.5	0		Fair drainage, generally moist	1
7.5-8.5	0		Good drainage, generally dry	0
>8.5	5			

Assuming fair site drainage, the tested soil sample had a total score of 6 points, indicating a moderate corrosive rating. When total points on the AWWA corrosivity scale are at least 10, the soil is classified as corrosive to cast and ductile iron pipe, and use of cathodic corrosion protection is often recommended.

These results are preliminary, and provide information only on the specific soil sampled and tested. Other soil at the site may be more or less corrosive. Providing a complete assessment of the corrosion potential of the site soils are not within our scope of work. For specific long-term corrosion control design recommendations, we recommend that a California-registered professional corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures, steel pipe coated with cement-mortar, and ferrous metals.

5.0 GEOLOGIC HAZARDS

5.1 Seismic Induced Hazards

Seismic hazards resulting from the effects of an earthquake generally include ground shaking, liquefaction, lateral spreading, dynamic settlement, fault ground rupture and fault creep, dam inundation, and tsunamis and seiches. The site is not necessarily impacted by all of these potential seismic hazards. Nonetheless, potential seismic hazards are discussed and evaluated in the following sections in relation to the planned construction.

5.1.1 Ground Shaking

The site will likely experience severe ground shaking from a major earthquake originating from the major active Bay Area faults, particularly the nearby San Andreas Fault (approximately 17.5 miles from the site) or Rodgers Creek Fault (approximately 4.0 miles from the site).

5.1.2 Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. However, because of the higher inter-granular pressure of the soil at greater depths, the potential for liquefaction is generally limited to the upper 40 feet of the soil. Potential hazards associated with soil liquefaction below or near a structure include loss of foundation support, lateral spreading, sand boils, and areal and differential settlement.

Lateral spreading is lateral ground movement, with some vertical component, as a result of liquefaction. The soil literally rides on top of the liquefied layer. Lateral spreading can occur on relatively flat sites with slopes less than two percent under certain circumstances. Lateral spreading can cause ground cracking and settlement. Due to type of subsurface condition and relatively level surface condition, the potential for lateral spreading is low.

The site is mapped by the Geologic Hazards, Petaluma General Plan 2025 EIR (Figure 3.7-5) as being located out of the zone of high liquefaction potential (see attached *Figure 7 - Geologic Hazards Map*). However it is very close to the high liquefaction potential zone or at the border line. During our field investigation, we observed the site as being underlain by potentially liquefiable layer at depths between 17 feet to 29 feet and again at 33 feet to 37 feet.

Furthermore, due to the presence of the shallow groundwater level which is a key factor for triggering the liquefaction, we ran Liquefy-Pro software to analyze a potential for the liquefaction settlement on Boring B-1 and B-

4. For the analyses, we considered historical water table at a time of liquefaction as submerge condition or at the ground surface for B-1 and two feet below existing ground surface for B-4 due to difference in ground elevations, seismic Peak Ground Acceleration (PGA_M) coefficient of 0.752g, and maximum considered earthquake (MCE) of 7.0. The detailed result of the liquefaction analysis is attached as Appendix C. We estimated the maximum total settlements of **1.9"** and **0.47"** with maximum potential differential settlements of about **1.26"** and **0.31"** in Boring B-1 and B-4 respectively, based on the information collected during the field investigation, laboratory test results, types of soils encountered in the borings within the project site.

5.1.3 Dynamic Compaction (Settlement)

Dynamic compaction is a phenomenon where loose, sandy soil located above the water table densified from vibratory loading, typically from seismic shaking or vibratory equipment. The site is generally underlain by upper few layers of stiff to very stiff sandy clay/clay. Dynamic compaction at this site should not be an issue.

5.1.4 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo Act, the California Geological Survey established boundary zones or Earthquake Fault Zone surrounding faults or fault segments judged to be sufficiently active, well-defined, and mapped for some distance. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site.

The site is not currently within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997). The closest Active Earthquake Fault Zone is associated with Green Valley/Hayward Fault, located about 4.0/8.0 miles from the site (see *Figure 5 - Regional Fault Map*). Since the site is not within an Earthquake Fault Zone, the potential for fault ground rupture and fault creep hazards are judged to be very low.

5.2 Other Hazards

Potential geologic hazards other than those caused by a seismic event generally include ground failure and subsidence, landslides, expansive and collapsible soils, flooding, and soil erosion. These are discussed and evaluated in the following sections.

5.2.1 Ground Cracking and Subsidence

Withdrawal of groundwater and other fluids (i.e. petroleum and the extraction of natural gas) from beneath the surface has been linked to large-scale land subsidence and associated cracking on the ground surface. Other causes for ground cracking and subsidence include the oxidation and resultant compaction of peat beds, the decline of groundwater levels and consequent compaction of aquifiers, hydrocompaction and subsequent settlement of alluvial deposits above the water table from irrigation, or a combination of any of these causes. Due to the absence of any of these factors, the potential for subsidence or related ground cracking is considered low.

5.2.2 Consolidation Settlement

Consolidation is the densification of soil into a more dense arrangement from additional loading, such as new fills or foundations. Consolidation of clayey soils is usually a long-term process, whereby the water is squeezed out of the soil matrix with time. Sandy soils consolidate relatively rapidly with an introduction of a load. Consolidation of soft and loose soil layers and lenses can cause settlement of the ground surface or buildings. Based on testing in the field, laboratory testing, and type of soils and depth of groundwater level, potential for consolidation settlement at this site is moderate. Based on our analysis using soils index parameters we estimated a consolidation settlement of approximately 1.25 inches from the upper 2 to 12 feet layer. Details about a remediation will be discussed more in the structure pad preparation section. Our Consolidation analysis utilized a combination of the data from our investigation in addition to the previous data from Fugro.

5.2.3 Expansive and Collapsible Soils

The result of the laboratory testing performed on representative sample of the near-surface soils indicated high plasticity soils. Hence, there is a high potential for expansion of the near subgrade soils at this site. This issue of high expansion will be addressed in our structure pad preparation recommendations.

The subsurface deposits encountered during the drilling program generally consisted of stiff to very stiff or medium dense clay and clayey sand. Collapsible soils are loose chemically bonded fine sandy and silty soils that have been laid down by the action of flowing water, usually in alluvial fan deposits. Terrace deposits and fluvial deposits can also contain collapsible soil deposits. The soil particles are usually bound together with a mineral precipitate. The loose structure is maintained in the soil until a load is imposed on the soil and water is introduced. The water breaks down the inter-particle bonds and the newly imposed loading densifies the soil. These types of soils are not present at this site. Therefore, the potential for collapsible soils underlying the site is considered to be low for this project site.

5.2.4 Flooding

The site is located in an area of minimal flooding hazard. FIRM (2009) has mapped the site vicinity as Zone X, areas determined to be outside the 0.2% annual chance floodplain, see attached *Figure 8 - FEMA National Flood Insurance Program Map, Flood Hazard Map*. Based on the site's proximity to this Flooding Zone, the topography of the proposed construction site can be considered to have a low hazard potential for seasonal flooding. Determining the flood hazard of the site is beyond the scope of this study or our expertise, and a flood specialist should be contacted if a more in-depth flooding analysis is desired.

As requested by the Project Design Team (Christine Gharagozian and Wei Tzeng with Carollo Engineers) by issuing a Memo dated December 20, 2016, we also have referenced the Intergraded Geotechnical Study, Section 3.9.3 Inundation in 100-Year Flood, prepared by Fugro dated 2005. Based on the report, "The flood-hazard map by Limerinos et al. (1973) shows portions of the site in an area prone to inundation in a 100-year flood. Roughly the southern 1/3 of Parcel A site adjacent to Ellis Creek is in this flood-prone area. The map does not explicitly indicate the potential height of floodwater. The edge of the flood-prone area was drawn approximately at Elevation +10 feet above mean sea level (before modification with dikes). A more recent flood-hazard map by FEMA (1989) shows the estimated 100-year flood is at Elevation +7 feet. Considering general site grade will be raised to at least +16 feet in all the clusters, the flooding potential to the proposed WRF may be minimal under the design 100-year flood".

5.2.5 Landsliding

The site is not yet evaluated by CGS sources as being located within an existing landslide or potential landslide area. Based on the USGS rainfall induced landslide map and surrounding topography, the site is not considered prone to potential landslide.

5.2.6 Soil Erosion

Present construction techniques and agency requirements have provisions to limit soil erosion and resultant siltation during construction. These measures will reduce the potential for soil erosion at the site during the various construction phases. Long-term erosion at the site will be reduced by landscaping and hardscape areas, such as parking lots and walkways, designed with appropriate surface drainage facilities.

5.2.7 Naturally Occurring Asbestos (NOA)

The borings did not encounter any soils which are a concern for potential asbestos hazard. Additionally this site is not known to have past history or potential for NOA hazard.

5.2.8 Other Geologic Hazards

Due to the sites location, subsurface soil conditions, groundwater levels and land use factors, the site is not subject to the potential geologic hazards of loss of mineral resources, volcanism, tsunamis, seiches, dam failure inundation, cyclic softening of soils or loss of unique geologic features.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

6.1 Conclusions

The site is considered geotechnically suitable for the proposed improvements provided the recommendations of this report are incorporated into the design and implemented during construction. The predominant geotechnical and geological issues that need to be addressed at this site are summarized below.

Seismic Ground Shaking – The site is located within a seismically active region. As a minimum, the building design should consider the effects of seismic activity in accordance with the latest edition of the California Building Code (CBC-2016).

Cut and Fill – The proposed site will require minor cuts and major fills for development of the structure pad. Minor fill might be required to reduce the potential for consolidation settlement due to high moisture and high plastic soils. Significant amount of fill will be required for building the pad to a design subgrade level. Cutting could range from one to two feet. We anticipated the required engineered fill after cut would be 9 to 10 feet as per proposed design.

Settlement:

Digester Gas Tank - The site could experience a maximum settlement of about 3.1 inches (seismic settlement 1.9" + 1.2" static/consolidation settlement) and differential settlement of about 1.0" to 2.0", if no other remedial measures are taken to minimize the effects. Consolidation settlement can be reduced by early surcharge with engineered fill.

Screw Press Building- The site could experience a maximum settlement of about ½" and differential settlement of ¼" for allowable bearing pressure as indicated on Table 6.6.2.1, Allowable Bearing Pressures. For higher seismic load the static combined with the seismic could experience up to 1" and differential of ½".

Corrosive Soil – The preliminary corrosion evaluation indicated the test sample was very low corrosive to buried cast and ductile iron pipe. There is not potential for corrosion at the project site.

Winter Construction – If grading occurs in the winter rainy season, appropriate erosion control measures will be required and weatherproofing of the building pads, foundation excavations, and/or pavement areas should be considered. Winter rains may also impact foundation excavations and underground utilities.

Groundwater – Groundwater level on the proposed project site is shallow. Groundwater might be problematic if deep excavation is planned during winter rainy season.

Utility Connections – As a general suggestion, where utility damage during a design seismic event may be an issue, the Structural Engineer may wish to consider utility connections at building perimeters designed for at least one (1”) inch of potential movement in any direction where the utility enters the screw press building and two inches at Bio-Digester location. This flexibility would help accommodate potential differential movement during a seismic event.

6.2 Seismic Design Parameters

The proposed structures should be designed in accordance with local design practice to resist the lateral forces generated by ground shaking associated with a major earthquake occurring within the greater San Francisco Bay region. Based on the subsurface conditions encountered in our borings we judge Site Class “D”, representative of stiff/very stiff sandy clays and medium dense clayey sand averaged over the uppermost 100 feet of the subsurface profile to be appropriate for this site. For design of the proposed site structures in accordance with the seismic provisions of the CBC 2016 and American Society of Civil Engineers (ASCE) 7-10, the following seismic ground motion values should be used as a minimum for design.

Table 6: Seismic Coefficients Based on 2016 CBC (per ASCE 7-10)

Item	Value	2016 CBC Source[R1]	ASCE 7-10 Table/Figure[R2]
Site Class	D	Table 1613.3.2.	Table 20.3-1
Mapped Spectral Response Accelerations			
Short Period, S_s	2.011 g		Figure 22-1
1-second Period, S_1	0.819 g		Figure 22-2
Site Coefficient, F_a	1.0	Table 1613.3.3(1)	Table 11.4-1
Site Coefficient, F_v	1.5	Table 1613.3.3(2)	Table 11.4-2
MCE (S_{MS})	2.011 g	Equation 16-37	Equation 11.4-1
MCE (S_{M1})	1.229 g	Equation 16-38	Equation 11.4-2
Design Spectral Response Acceleration			
Short Period, S_{DS}	1.341 g	Equation 16-39	Equation 11.4-3
1-second Period, S_{D1}	0.819 g	Equation 16-40	Equation 11.4-4
Peak Ground Acceleration, PGA_M	0.774 g		Equation 11.8-1

R1 California Building Standards Commission (CBSC), “California Building Code,” 2016 Edition.
R2 U.S. Seismic “Design Maps” Web Application, https://geohazards.usgs.gov/secure/designmaps/us/application.php

ASCE 7-15 § 11.6-1 and 11.6-2 indicate that the Seismic Design Category for all Occupancy Categories is “E”.

6.3 Site Grading and Site Preparations

6.3.1 General Grading, Demolition, Preparation, and Drainage

Site grading should be performed in accordance with these recommendations. A pre-construction conference should be held at the jobsite with representatives from the owner, general contractor, grading contractor, and CEL prior to starting the clearing and demolition operations at the site.

Site grading at this site is generally anticipated to consist of minor to moderate cuts and fills required to construct the new proposed structures. Screw press building is anticipated to have minor grading or site preparation requirements while digester tank is anticipated to have moderate grading and site preparation. Import fill required as backfill for this project should be non-expansive, having a Plasticity Index of 12 or less, an R-Value greater than 40, and enough fines so the soil can bind together but not more than 20 percent. Imported soils should be free of organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. The Geotechnical Engineer should approve imported fill prior to delivery onsite.

Minor demolition of existing structures or other underground utilities at the project site may be required. Prior to commencement of grading activities, all the existing pavements, foundation remnants, utilities, trees and roots, surface vegetation, organic-laden soils, building materials, existing loose soil, concrete, debris and other deleterious materials should be cleared. Debris resulting from site stripping operations should be removed from the site, unless otherwise permitted by the Geotechnical Engineer.

Excavations resulting from the removal of abandoned underground utilities, or deleterious materials should be cleaned down to firm soil, processed as necessary, and backfilled with engineered fill in accordance with the grading sections of this report. The Geotechnical Engineer's representative should verify the adequacy of site clearing operations during construction, prior to placement of engineered fill.

Existing underground utilities proposed to be abandoned, if present, should be properly grouted, closed, or removed as needed. The extent of removal/abandonment depends on the diameter of the pipe, depth of the pipe, and proximity to buildings and pavement.

Final grading should be designed to provide drainage away from structures and the top of slopes. Soil areas within 10 feet of proposed structures or as applicable from the site condition should slope at a minimum of five percent away from the building. Adjacent concrete hardscape should slope a minimum two percent away from the building. Roof leaders and downspouts should not discharge into landscape areas adjacent to buildings, and should discharge onto paved surfaces sloping away from the structure or into a closed pipe system channeled away from the structure to an approved collector or outfall.

6.3.2 Project Compaction Recommendations

The following table provides the recommended compaction requirements for this project. Not all soils, aggregates and scenarios listed below may be applicable for this project. Specific grading recommendations are discussed individually within applicable sections of this report.

Table 6.3.2.1: Project Compaction Requirements

Description	Min. Percent Relative Compaction (per ASTM D1557)	Percent Above/below Optimum Moisture Content
Fill Areas, Engineered Fill, Onsite Soil	90	+ 3
Fill Areas, Engineered Fill, Select Fill	95	± 3
Building Pads, Onsite Soil – Scarified Subgrade or used as Fill	90	+ 3
Building Pads, Baserock or Select (non-expansive) Engineered Fill	95	± 3
Building Pads – Treated Soil	95	± 3
Concrete Flatwork, Subgrade Soil	90	+ 3
Concrete Flatwork, Baserock	95	± 3
Underground Utility Backfill - Below 3 feet	90	+ 3
Underground Utility Backfill - Upper 3 feet	95	+ 3
AC Pavement – Onsite Subgrades (upper 12 inches)	95	+ 3
AC Pavement – Non-Expansive Subgrades in Traffic Areas (upper 12 inches)	95	± 3
Pavement – Class 2 Aggregate Base Section	95	± 3

6.3.3 Building Pad Grading

Digester Gas Tank

After cutting the existing organic top soils to required depth, the structure pad will require engineered fill to establish new grades as required. We understand that the floor finished level of the proposed structure has been planned at an elevation close to +20 feet (amsl). We anticipated at least a foot or two feet of top soil should be removed before placement of non-expansive fill to build up the structure pad to a desire finished grade. Based on the existing tank site subgrade elevation of +11.0, we anticipated about 9 to 10 feet of engineered fill will be required after excavation of native top soils to prepare finished grade. The estimated total settlement due to seismic liquefaction settlement and consolidation settlement, which is about 3.1 inches, we recommend placing an engineered fill at least 100 days prior to installation of digester tank to reduce potential consolidation settlement. Otherwise higher settlement of 3.1 inches should be considered in design of the structure or utility lines.

Before placement of engineered fill, the building pad subgrade soil should be scarified to a depth of at least eight-inches below the existing subgrade soils acceptable to the Geotechnical Engineer, moisture conditioned to at least three percent over optimum moisture, and compacted to 90 percent relative compaction determined by ASTM D1557 (Modified Proctor) as required by the project compaction Table 6.3.2.1. The engineered fill should extend a minimum of five feet laterally from edge of the footing or from outer most part of the proposed structure or its components bearing in the subgrade soil and slope required to maintaining 1.5:1 (H:V) on the engineered fill. If loose or soft soil is encountered, these soils should be removed to expose firm soil and backfilled with engineered fill. Engineered fill should be placed in maximum eight-inch thick, un-compacted lifts. The fill should be moisture conditioned and thoroughly mixed during placement to provide uniformity in each layer. Requirements for a non-expansive select fill underlying the building pad are presented in Section 6.3.1.

Screw Press Building

Based on the information provided to us, we understand that the proposed new screw press building will be constructed similar to one adjacent to the northwest of the site. The finished adjacent level is anticipated to be at existing surface elevation of +16.0. Since the proposed site is in a fully built up area with proper compacted sandy clay or clayey sand, very minimum grading work will be required.

6.3.4 Grading Flatwork Areas

Areas to receive pavements, if any, should be scarified to a depth of eight inches below existing grade or final subgrade, whichever is lower. Scarified areas should be moisture conditioned and compacted. Where required, engineered fill should be placed and compacted to reach design subgrade elevation. Once the compacted pavement subgrade has been reached, it is recommended that baserock in paved and on-grade concrete slab areas be placed immediately after grading to protect the subgrade soil from drying. Alternatively, the subgrade should be kept moist by watering until baserock is placed.

Rubber-tired heavy equipment, such as a full water truck, should be used to proofload exposed subgrade areas where pumping is suspected. Proof loading will determine if the subgrade soil is capable of supporting construction equipment without excessive pumping or rutting.

6.3.5 Site Winterization and Unstable Subgrade Conditions

If grading occurs in the winter rainy season, unstable and unworkable subgrade conditions may be present and compaction of onsite soils may not be feasible. These conditions may be remedied using soil admixtures, such as lime/cement. A four percent mixture of lime based on a soil unit weight of 120 pcf is recommended for planning purposes. Treatment should vary between 12 to 18 inches, depending on the anticipated construction equipment loads. More detailed and final recommendations can be provided during construction. Stabilizing subgrade in small, isolated areas can be accomplished with the approval of the Geotechnical Engineer by over-excavating one foot, placing Tensar BX1100 or equivalent geogrid on the soil, and then placing 12-inches of Class 2 baserock on the geogrid. The upper six inches of the baserock should be compacted to at least 95 percent relative compaction.

6.4 Utility Trench Construction

6.4.1 Trench Backfilling

Utility trenches may be backfilled with onsite selected soil above the utility bedding and shading materials. If rocks or concrete larger than four inches in maximum size are encountered, they should be removed from the fill material prior to placement in the utility trenches. Utility bedding and shading compaction requirements should be in conformance with the requirements of the local agencies having jurisdiction and as recommended by the pipe manufacturers. Jetting of trench backfill is not recommended. Compaction recommendations are presented in Table 6.3.2.1, Project Compaction Recommendations.

Pea gravel, rod mill, or other similar self-compacting material should not be utilized for trench backfill since this material will transmit the shallow groundwater to other locations within the site and potentially beneath the buildings. Additionally, fines may migrate into the voids in the pea gravel or rod mill, which could cause settlement of the ground surface above the trench.

If rain is expected and the trench will remain open, the bottom of the trench may be lined with one to two inches of gravel. This would provide a working surface in the trench bottom. The trench bottom may have to be sloped to a low point to pump the water out of the trench.

6.4.2 Utility Penetrations at Building Perimeter

Flexible connections at building perimeters should be considered for utility lines going through perimeter foundations. This would provide flexibility during a seismic event. This could be provided by special flexible connections, pipe sleeving with appropriate waterproofing, or other methods.

Utility trenches should be sealed with concrete, clayey soil, sand-cement slurry, or controlled density fill (CDF) where the utility enters the building under the perimeter foundation. This would reduce the potential for migration of water beneath the building through the shading material in the utility trench.

6.4.3 Pipe Bedding and Shading

Pipe bedding material is placed in the utility trench bottom to provide a uniform surface, a cushion, and protection for the utility pipe. Shading material is placed around the utility pipe after installation and testing to protect the pipe. Bedding and shading material and placement are typically specified by the pipe manufacturer, agency, or project designer. Agency and pipe manufacturer recommendations may supersede our suggestions. These suggestions are intended as guidelines and our opinions based on our experience to provide the most cost-effective method for protecting the utility pipe and surrounding structures. Other geotechnical engineers, agency personnel, contractors, and civil engineers may have different opinions regarding this matter.

Bedding and Shading Material - The bedding and shading material should be the same material to simplify construction. The material should be clean, uniformly graded, fine to medium grained sand. It is suggested that bedding and shading material contain less than three percent fines with 100 percent passing the No. 8 sieve. Coarse sand, angular gravel or baserock should be avoided since this type of shading material may bridge when backfilling around the pipe, possibly creating voids, and may be too stiff as bedding material. Open graded gravel should be avoided for shading since this material contains voids, and the surrounding soil could wash into the voids, potentially causing future ground settlement. However, open graded gravel may be required for bedding material when water is entering the trench. This would provide a stable working surface and a drainage path to a sump pit in the trench for water in the trench. The maximum size for bedding material should be limited to about ¾ -inch.

Bedding Material Placement - The thickness of the bedding material should be minimized to reduce the amount of trench excavation, soil export, and imported bedding material. Two to three inches for pipes less than eight-inches in diameter and about four to six inches for larger pipes are suggested. Bedding for very large diameter pipes are typically controlled by the pipe manufacturer. Compaction is not required for thin layers of bedding material. The pipe needs to be able to set into the bedding, and walking on a thin layer of bedding material should sufficiently compact the sand. Rounded gravel may be unstable during construction, but once the pipe and shading material is in place, the rounded gravel will be confined and stable.

Shading Material Placement – Jetting is not typically recommended since the type of shading material is unknown when preparing the geotechnical report and agencies typically do not permit jetting. If the sand contains fines or if the sand is well graded, jetting will not work. Additionally, if too much water is used during jetting, this could create a wet and unstable condition. However, clean, uniformly graded and fine to medium sand can be placed by jetting. The shading material should be able to flow around and under the utility pipe during placement. Some compactive effort along the sides of the pipe should be made by the contractor to consolidate the shading material around the pipe. A minimum thickness of about six-inches of shading material should be placed over the pipe to protect the pipe from compaction of the soil above the shading material. The contractor should provide some compactive effort to densify the shading material above the pipe. Relative compaction testing is not usually performed on the shading material. However, the contractor is ultimately responsible for the integrity of the utility pipe.

6.5 Temporary Excavation Slopes and Shoring

Construction of the below grade structure will require either temporary excavation slopes or shoring to construct the building foundations. The Contractor should incorporate all appropriate requirements of OSHA/Cal OSHA into the design of any temporary construction slopes or shoring system, whichever is used. Excavation safety regulations are provided in the OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P, and apply to excavations greater than five feet in depth.

The Contractor, or his specialty subcontractor, should design temporary construction slopes to conform to the OSHA regulations and should determine actual temporary slope inclinations based on the subsurface conditions exposed at the time of construction. For pre-construction planning purposes, the subsurface materials in the areas of the site where excavation may take place may be assumed to consist of stiff Sandy Clay/Lean Clay categorized as OSHA Type A with temporary slope inclination of no steeper than 1:1 (horizontal: vertical). However in some areas subsurface soils are soft rock which should be able to maintain a temporary slope inclination of no steeper than 2:1 (horizontal: vertical). The type of slope material and temporary construction slopes should be confirmed during construction by a competent engineering geologist responsible to the grading contractor.

If temporary slopes are left open for extended periods of time, exposure to weather and rain could have detrimental effects such as sloughing and erosion on surficial soils exposed in the excavations. We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes, and that such temporary slopes are protected from excessive drying or saturation during construction. In addition, adequate provisions should be made to prevent water from ponding on top of the slope and from flowing over the slope face. Desiccation or excessive moisture in the excavation could reduce stability and require shoring or laying back side slopes.

If use of temporary slopes is not feasible, a temporary shoring system may be required to protect adjacent properties/structures against undesirable movement or deflection. The Contractor, or his specialty shoring subcontractor, should design and install temporary shoring. Possible shoring systems include soldier beam and lagging walls with or without tiebacks or sheet piles.

We recommend that the geotechnical and structural engineers review any temporary shoring plan to confirm compliance with the anticipated soil conditions encountered at the site. In addition, we recommend that the geotechnical engineer's representative observe the installation of the temporary shoring systems. The Contractor should incorporate all appropriate requirements of OSHA into the design of the temporary shoring system.

6.6 Building Foundation Recommendations

6.6.1 Shallow Mat Foundation for Digester Gas Tank

We believe that the proposed digester tank can be supported on conventional structural slab or mat foundation bearing on engineered fill. The engineered fill should extend a minimum of five feet laterally from the edge of the footings. Bottom of the mat foundation can be founded a minimum of six inches below lowest adjacent finished grade on a minimum of four inch Class II aggregate base cushion over engineered fills. An allowable bearing pressure of 1000 psf can be used for foundation design and subgrade reaction, k of 150 pci can be used for mat slab design.

If site preparation and foundation observation services are conducted as outlined in the Geotechnical Study report, static vertical settlement is not expected to exceed more than 1.2 inches for footings bearing within the materials described in the report and designed to the aforementioned allowable bearing pressures. Differential settlement across the structure is not expected to exceed more than 1/2 inch with columns spaced at 25 feet. However, seismic settlement due to liquefaction could experience settlement of about 2.4 inches and differential settlement of 1.6 inches. Therefore utility lines connecting this structure should be designed to accommodate these settlements.

6.6.2 Shallow Spread Foundation for Screw Press Building

We believe that the proposed screw press building structures can be supported on conventional isolated and/or continuous spread footings bearing on well compacted firm subgrade soils. Or, may be supported on 12 inches of Class II aggregate compacted to a minimum of 90 percent relative compaction (ASTM D1557) at the bottom of the footings extending one foot laterally from both edges of footing, consistent with the adjacent existing Screw Press Building foundation. Footings should be founded a minimum of 24 inches below lowest adjacent finished grade. Continuous footings should have a minimum width of at least 24 inches, and isolated column footings should have a minimum width of 30 inches. Footings located adjacent to other footings or utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches. Footing reinforcement should be determined by the project Structural Engineer.

For the design of footings bearing within tested and approved improved subgrade soil or engineered fill, we recommend the following allowable net bearing pressures, assuming design Factors-of-Safety of 2.0 and 1.5 for dead loads or dead plus live loads and total loads including transient, respectively, from the estimated ultimate bearing pressure.

Table 6.6.2.1: Allowable Bearing Pressures for Spread Footings

Load Condition	Allowable Bearing Pressure (psf)
Dead Load	1,800
Dead plus Live Loads	1,900
Total Loads (including wind or seismic)	2,530
Total Loads (including wind or seismic)	4,000 (with a total settlement of 1" and diff settlement of 0.5")

CEL personnel should be retained to observe, test, and confirm that foundations are prepared as recommended in the Geotechnical Report. Geotechnical Engineer should approve the foundation prior to placement of formwork and reinforcing steel. If unsuitable soil is present, the excavation should be deepened until suitable supporting material is encountered. The over excavation should be backfilled using structural or lean concrete (or a sand-cement slurry mix acceptable to the Geotechnical Engineer) up to the bottom of the footing concrete.

Footing excavations should have firm bottoms and be free from excessive slough prior to concrete or reinforcing steel placement. Care should also be taken to prevent excessive wetting or drying of the bearing materials during construction. Extremely wet or dry or any loose or disturbed material in the bottom of the footing excavations should be removed prior to placing concrete. If construction occurs during the winter months, a thin layer of concrete (sometimes referred to as a rat slab) could be placed at the bottom of the footing excavations. This will protect the bearing soil and facilitate removal of water and slough if rainwater fills the excavations.

If site preparation and foundation observation services are conducted as outlined in the Geotechnical Study report, vertical settlement is not expected to exceed more than one-half inch for footings bearing within the materials described in the report and designed to the aforementioned allowable bearing pressures. Differential settlement across the structure is not expected to exceed more than 1/4 inch within a distance of 25 feet.

We understand that the proposed new structure will be tied into the old existing structure therefore special caution should be taken to distribute new foundation loading not overstressing the old foundation and also should considered differential settlement between the old structure and the new structure. The soils underneath the existing footings disturbed during construction should be backfilled with lean concrete, or sand-cement slurry, or controlled density fill (CDF).

6.6.3 Lateral Resistance

Shallow foundations can resist lateral loads with a combination of bottom friction and passive resistance. An allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an allowable passive resistance equal to an equivalent fluid weighing 300 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of footings perpendicular to the direction of loading where the footing is poured neat against undisturbed material. The top foot of passive resistance at foundations not adjacent to pavement or hardscape should be neglected. In order to fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied. The friction between the bottom of a slab-on-grade floor and the underlying soil should not be utilized to resist lateral forces.

6.6.4 Interior Slabs-on-Grade

Floor slab can be constructed with the interior slab-on-grade floor slabs. These slabs are subject to moisture variation, treatment of the slab subgrade will be required due to shallow historic groundwater and surrounding expansive soils. For non-structural concrete slab-on-grade floors we recommend a minimum of five-inch thick slab.

However, actual thickness of the slab should be determined by the Structure Engineer. Additionally, on-grade concrete floor slabs should be underlain by a minimum of four inches of Class II AB over a minimum 12 inch thickness of select, non-expansive fill or native soil. However, one foot of Class II Aggregate Base can be used in lieu of four inches of AB and over 12 inches of non-expansive fill.

Slab-on-grade concrete floors with moisture sensitive floor coverings should be underlain by a moisture retarder system constructed between the slab and subgrade. Such a system could consist of four inches of free-draining gravel, such as 3/4-inch, clean, crushed, uniformly graded gravel with less than three percent passing No. 200 sieve, or equivalent, overlain by a relatively impermeable vapor retarder placed between the subgrade soil and the slab. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class C Underslab Vapor Retarders (e.g., Griffolyn Type 65, Griffolyn Vapor Guard, Moistop Ultra C, or equivalent). If additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.006 gr/ft^2/hr (i.e., 0.012 perms) per ASTM E 96 (e.g., 15-mil thick "Stego Wrap Class A"), or to Class B (Griffolyn Type 85, Moistop Ultra B, or equivalent) may be used in place of a Class C retarder.

The vapor retarder or barrier should be placed directly under the slab. A capillary rock layer or rock cushion is not required if a Class A barrier is used beneath the floor slab, and a sand layer is not required over the vapor retarder from a geotechnical standpoint. If sand on top of the vapor retarder is required by the design structural engineer, we suggest the thickness be minimized to less than one inch. If construction occurs in the winter months, water may pond within the sand layer since the vapor retarder may prevent the vertical percolation of rainwater. The thickness of the capillary rock layer and sand, if either or both are used, may be considered to comprise part of the recommended non-expansive fill layer underlying the floor slab.

ASTM E1643 should be utilized as a guideline for the installation of the vapor retarder. During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using a waterproof tape or mastic applied in accordance with the vapor retarder manufacturer's specifications. The vapor retarder or barrier should extend to the perimeter cutoff beam or footing.

<u>6.7 Below Grade Walls</u>

We believe that the proposed project may require soil retaining structures. We anticipated that there wouldn't be any other loads that would influence the wall other than lateral soils pressure, and the backfill soils behind will be either level/flat. The pressures on the wall for the design of the retaining structures are provided in 6.7.1 Lateral Earth Pressures.

The lateral pressures for soils given below assume the backfill behind the retaining wall is clayey sand/non expansive. If the walls are not provided with a drainage system then hydrostatic pressure should be added (as appropriate), which significantly increases the lateral pressure on the wall. An allowable bearing capacity of 3,000 pounds/square-foot can be used for designing the wall foundation. Any surcharges should be considered if imposed behind the top of the wall within a zone established by a 45 degree projection upward from the bottom of the wall footing.

6.7.1 Lateral Earth Pressures

For clayey sand/non expansive soils above any free water surface, recommended equivalent fluid pressures for foundation elements are presented below.

Active:	Compacted clayey sand/non expansive soils	45 psf/ft
At Rest:	Compacted clayey sand/non expansive soils	65 psf/ft
Passive:	Compacted clayey sand/non expansive soils	300 psf/ft
Coefficient of Friction:		0.35

Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.3 times the anticipated surcharge load for unrestrained walls, and 0.4 times the anticipated surcharge load for restrained walls.

The lateral earth pressures herein do not include any factor-of-safety and are not applicable for submerged soils/hydrostatic loading. Additional recommendations may be necessary if submerged conditions are to be included in the design. Below-grade walls such as elevator pit walls, utility vaults can be designed to accommodate an additional hydrostatic pressure increment.

Below-grade wall backfill less than five feet deep should be compacted to at least 90 percent relative compaction using light compaction equipment. Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors. Over compaction may cause excessive lateral earth pressures which could result in wall movement. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced.

6.7.2 Retaining Wall Drainage (as warranted by site condition)

To reduce hydrostatic loading on retaining/below grade walls, a subsurface drain system should be placed behind the wall. The drain system should consist of free-draining granular soils containing less than five percent fines passing a No. 200 sieve, placed adjacent to the wall. The free-draining granular material should be graded to prevent the intrusion of fines, or else should be encapsulated in a suitable filter fabric. A drainage system consisting of either weep holes or perforated drain lines (minimum 4" diameter placed near the base of the wall) should be used to intercept and discharge water which would tend to saturate the backfill. Where used, drain lines should be embedded in a uniformly graded filter material and provided with adequate clean-outs for periodic maintenance. An impervious soil should be used in the upper one foot layer of backfill to reduce the potential for water infiltration. As an alternative, a prefabricated drainage structure, such as geo-composite, may be used as a substitute for the granular backfill adjacent to the wall.

The retaining wall drainage system should be sloped to outfall to the storm drain system or other appropriate facility. The foundation of the retaining wall should be protected and prevented from any erosion of the surroundings.

CEL personnel should be retained to observe and evaluate that foundation excavations terminate in soils suitable for the design bearing pressure. If unsuitable soil is present, the excavation should be extended until suitable material is encountered. Unsuitable soil or fill removal should also extend at least eight inches beyond the foundation edge for each 12-inch thickness of unsuitable soil being removed. The material removed should be replaced with an approved engineered fill/granular soil, placed and compacted.

6.8 Plan Review

We recommend that CEL be provided the opportunity to review the final project plans prior to construction. The purpose of this review is to assess the general compliance of the plans with the recommendations provided in this report and confirm the incorporation of these recommendations into the project plans and specifications.

6.9 Observation and Testing During Construction

We recommend that CEL be retained to provide observation and testing services during site preparation, mass grading, underground utility construction, foundation excavation, and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

7.0 VALIDITY OF REPORT

This report is valid for three years after publication. If construction begins after this time period, CEL should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, CEL should be notified to determine if additional recommendations are required. Additionally, if CEL is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid; CEL's geotechnical personnel should be retained to verify that the subsurface conditions anticipated when preparing this report are similar to the subsurface conditions revealed during construction. CEL's involvement should include grading and foundation plan review, grading observation and testing, foundation excavation observation, and utility trench backfill testing.

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the borings. If variations or undesirable conditions are encountered during construction, CEL should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Accordingly the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by CEL after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered CEL should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that CEL be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that CEL will be retained to provide these services.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

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FIGURES

APPENDIX A

Key to Boring Log Symbols
Boring Logs

APPENDIX B

LABORATORY TEST RESULTS

Liquid and Plastic Limits Test Report
Sieve Analysis Result
Direct Shear Test Report
Corrosivity Test Results

APPENDIX C

LIQUEFACTION ANALYSIS RESULT