SOILS ENGINEERING REPORT
CAYUCOS SUSUTAINABLE WATER PROJECT
TORO CREEK ROAD, CAYCUCOS AREA
SAN LUIS OBISPO COUNTY, CALIFORNIA

PROJECT SL10070-1

Prepared for

Attn: Rick Koon
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Prepared by

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June 9, 2017

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Client:
Attn: Rick Koon
Cayucos Sanitary District
PO Box 333
Cayucos, California 93430

Project name:
Cayucos Sustainable Water Project, Toro Creek Road, Cayucos Area, San Luis Obispo County, California

Dear Mr. Koon:

This Soils Engineering Report has been prepared for the proposed Cayucos Sustainable Water Project to be located along Toro Creek Road in the Cayucos area of San Luis Obispo County, California. Geotechnically, the site is suitable for the proposed development provided the recommendations in this report for site preparation, earthwork, foundations, slabs, retaining walls, and pavement sections are incorporated into the design.

It is anticipated that all foundations will be excavated into engineered fill. All foundations are to be excavated into uniform material to limit the potential for distress of the foundation systems due to differential settlement. If cuts steeper than allowed by State of California Construction Safety Orders for "Excavations, Trenches, Earthwork" are proposed, a numerical slope stability analysis may be necessary for temporary construction slopes.

Natural seepage at the interface of two materials with different densities is very common. This interface occurs at the Site and will require sub-surface drains. Sub-drains should be placed in established drainage courses, potential seepage areas, and during the development of all key and bench grading operations.

Thank you for the opportunity to have been of service in preparing this report. If you have any questions or require additional assistance, please feel free to contact the undersigned at (805) 543-8539.

Sincerely,
GeoSolutions, Inc.

Kelly Robinson

Kelly M. Robinson, PhD, PE
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SAN LUIS OBISPO COUNTY, CALIFORNIA

PROJECT SL10070-1

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation for the proposed Cayucos Sustainable Water Project to be located on Toro Creek Road in the Cayucos area of San Luis Obispo County, California. See Figure 1: Site Location Map for the general location of the project area, hereafter referred to as the Site. Figure 1: Site Location Map was obtained from the computer program *Topo USA 8.0* (DeLorme, 2009).

1.1 Site Description

The proposed construction area consists of about 11 acres of agricultural land located along the east side of Toro Creek Road, approximately 0.6 miles northeast of the intersection between Highway 1 and Toro Creek Road. Site coordinates are estimated to be about 35.4200 degrees latitude and -120.8632 degrees longitude.

The Site is relatively flat with a gentle gradient descending to the west at a gradient of about 5% towards Toro Creek Road. Toro Creek Road bounds the property on the west and ascending hillsides extend along the east property boundary. Toro Creek is located approximately 500 feet northwest of the Site and generally flows generally to the southwest towards the Pacific Ocean, located about ¾ mile from the Site.

Figure 1: Site Location Map

Annual grasses currently vegetate the Site. Site drainage generally follows the topography to the west and southwest towards Toro Creek Road and Toro Creek as well as into a local drainage gully trending east to west across the Site, descending from the adjacent hillsides towards Toro Creek.

1.2 Project Description

Based on review of project plans provided by Water Consulting Systems (WCS) and discussions with the client, the project will consist of constructing about 2.4 acres of solar panels in the northern portion of the property and a sewer treatment plant in the area south of the existing drainage gully crossing the site. As part of the treatment plant, various buildings and tanks are proposed including an office/lab building, maintenance building, an HDPE-lined equalization basin, below-grade aeration/membrane tanks, a recycled water storage tank, and various smaller structures associated with the treatment process. Based on information provided by the client, site grading for the project will likely include removal of the upper two feet of material in the area of the proposed treatment facilities.

It is anticipated that the proposed office/lab building and maintenance building will utilize a slab-on-grade . Dead and sustained live loads are currently unknown, but they are anticipated to be relatively light with maximum continuous footing and column loads estimated to be approximately 1.5 kips per linear foot and 15 kips, respectively, for the buildings.

Based on our project understanding, the proposed aeration/membrane tank will be founded approximately 15 to 20 feet below finish grade and will be concrete-lined and filled with water or remain partially empty. The proposed above-grade, recycled water storage tank will be approximately 50 feet in diameter and will hold approximately 250,000 gallons of recycled water.

Previous work for the project includes an engineering geologic hazard evaluation prepared for the EIR (GeoInsite, Inc. 2016) and a preliminary geotechnical report performed by Yeh and Associates (2016). As part of additional services for the project, GeoSolutions, Inc. provided a Subsurface Data Report and Engineering Geology Investigation for the proposed pipeline portion of the project located along Toro Creek Road, provided under separate cover.

2.0 PURPOSE AND SCOPE

The purpose of this study was to explore and evaluate the surface and sub-surface soil conditions at the Site and to develop geotechnical information and design criteria. The scope of this study includes the following items:

1. A literature review of available published and unpublished geotechnical data pertinent to the project site including geologic maps, and available on-line or in-house aerial photographs.
2. A field study consisting of site reconnaissance and subsurface exploration including exploratory borings in order to formulate a description of the sub-surface conditions at the Site.
3. Laboratory testing performed on representative soil samples that were collected during our field study.
4. Engineering analysis of the data gathered during our literature review, field study, and laboratory testing.
5. Development of recommendations for site preparation and grading as well as geotechnical design criteria for building foundations, retaining walls, pavement sections, underground utilities, and drainage facilities.

3.0 FIELD AND LABORATORY INVESTIGATION

The field investigation was conducted on March 9, 2017, using a track-mounted CME 55 drill rig. Two eight-inch diameter exploratory borings were advanced to depths of 50 feet (S-1) and 40 feet (S-2) below ground surface (bgs) at the approximate locations indicated in Figure 2: Field Exploration Plan. Sampling methods included the Standard Penetration Test utilizing a standard split-spoon sampler (SPT) without liners and a Modified California sampler (CA) with liners. The CME 55 drill rig was equipped with an automatic hammer, which has an efficiency of approximately 80 percent and was used to obtain test blow counts in the form of N-values. During the boring operations the soils encountered were continuously examined, visually classified, and sampled for general laboratory testing. A project engineer reviewed continuous logs of the soils encountered at the time of field investigation which are provided in **Appendix A**.

Laboratory tests were performed in our in-house lab on select soil samples obtained from the Site during the field investigation. Testing for various engineering properties and soils classification included expansion index testing, grain size distribution, Atterberg limits, direct shear, and consolidation, Laboratory data reports and explanations of the laboratory tests performed in general accordance with the applicable ASTM standards are provided in **Appendix B**.

As part of previous work for the project, Yeh and Associates (2016) advanced seven (7) cone penetrometer tests (CPTs) to depths of about 31 to 73 feet below ground surface (bgs) and one (1), 50-foot deep exploratory boring at the site. The approximate locations of the 2016 field explorations are provided in Figure 2: Field Exploration Plan

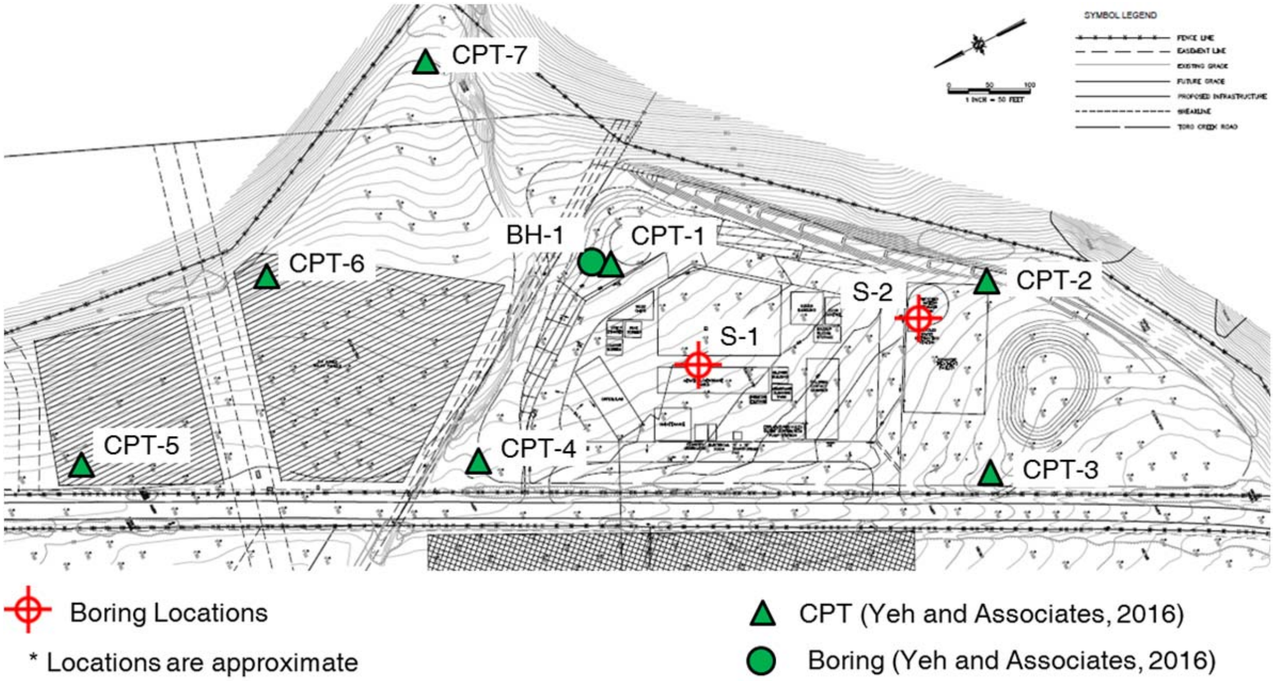

Figure 2: Field Exploration Plan

4.0 SUBSURFACE CONDITIONS

4.1 Site Geology

Regional site geology was obtained using the MapView internet application, available from the United States Geological Survey website (USGS, 2013) which compiles existing geologic maps. As indicated by the *Geologic Map of the Cayucos Quadrangle* (Dibblee, 2006) provided in Figure 3: Regional Geologic Map, the geologic units mapped in the Site vicinity consist of surficial deposits of alluvium (Qa) and formational material of Serpentine (sp) and Franciscan Rocks (fm, fg).

4.2 Soil Conditions

Data gathered during the field investigation suggest that the soil materials at the Site consist of interbedded layers of alluvial soil extending to the maximum depth explored of 50 feet bgs. Dark brown sandy CLAY (CL) was encountered in our borings in a stiff to very stiff at slightly moist condition extending from the ground surface to depths of about 29 to 44 feet bgs. Medium dense clayey SAND with grave (SC) and very dense clayey GRAVEL (GC) was encountered underlying the sandy clay materials to the maximum depth explored. The subsurface conditions encountered were generally consistent with previous explorations at the Site (Yeh and Associates 2016) which generally indicated clayey soils underlain by dense to very dense material encountered at depths ranging from about 32 feet (CPT-07) to 72 feet (CPT-04) bgs across the Site.

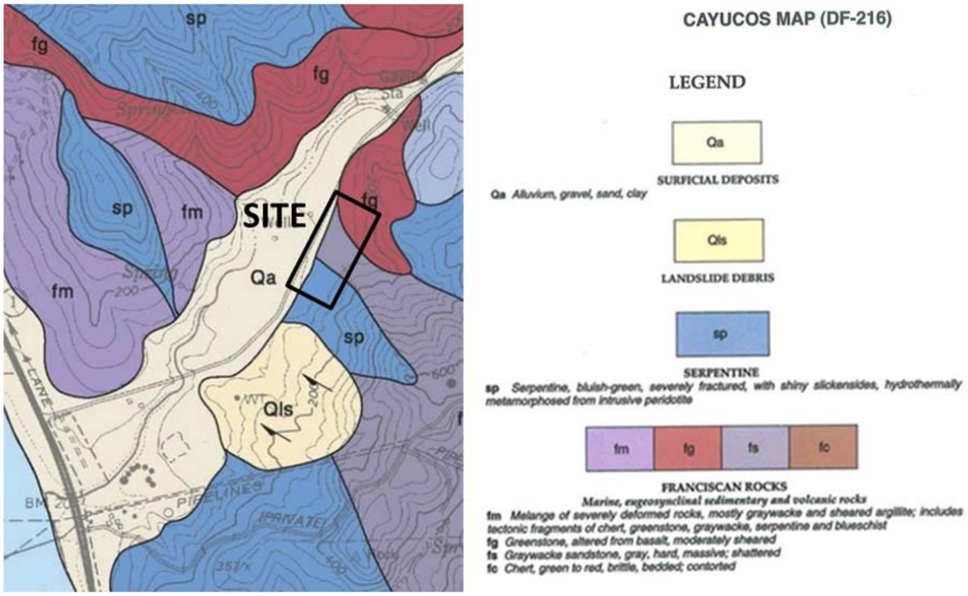

Figure 3: Regional Geologic Map

Results of laboratory testing performed to assist in our understanding of the materials at the Site are summarized in Table 1: Engineering Properties. The test results indicated a MEDIUM potential for expansion in the subsurface soils.

Table 1: Engineering Properties

Sample Name	Location	Sample Description	USCS Specification	Expansion Index	Expansion Potential	Plasticity Index	Fines Content (%)	Angle of Internal Friction, ϕ (deg.)	Cohesion, c (psf)	Compression Index, C_c	Recompression Index, C_r
C	S-1@2-5’	Very Dark Brown Sandy CLAY	CL	74	Medium	31	65	-	-	-	-
D	S-1@10-15’	Very Dark Grayish Brown Sandy CLAY	CL	53	Medium	34	71	-	-	-	-
-	S-1@5’	Very Dark Brown Sandy CLAY	CL	-	-	-	-	24.3	911	-	-
-	S-2@5’	Very Dark Grayish Brown Sandy CLAY	CL	-	-	-	-	-	-	0.129	0.013
-	S-2@20’	Dark Yellow Brown Sandy CLAY	CL	-	-	-	-	-	-	0.152	0.015

4.3 Groundwater Conditions

Groundwater was encountered at depths of 28 feet (S-1) and 23 feet (S-2) bgs during the field investigation. Yeh and Associates (2016) reported groundwater depths ranging from 22 to 32 feet bgs in

2016 field investigation. It should be anticipated that groundwater depths may vary seasonally and with irrigation practices.

5.0 SEISMIC DESIGN CONSIDERATIONS

Estimating the design ground motions at the Site depends on many factors including the distance from the Site to known active faults; the expected magnitude and rate of recurrence of seismic events produced on such faults; the source-to-site ground motion attenuation characteristics; and the Site soil profile characteristics. According to section 1613 of the 2016 CBC (CBSC, 2016), all structures and portions of structures should be designed to resist the effects of seismic loadings caused by earthquake ground motions in accordance with the *ASCE 7 2010 Minimum Design Loads for Buildings and Other Structures*, hereafter referred to as ASCE7-10 (ASCE, 2013). The Site soil profile classification (Site Class) can be determined by the average soil properties in the upper 100 feet of the Site profile and the criteria provided in Table 20.3-1 of ASCE7-10.

Spectral response accelerations, peak ground accelerations, and site coefficients provided in this report were obtained using the computer-based U.S. Seismic Design Map tool available from the United States Geological Survey website (USGS, 2013). This program utilizes the methods developed in the 1997, 2000, 2003, 2008 and 2013 errata editions of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures in conjunction with user-inputted Site latitude and longitude coordinates to calculate seismic design parameters and response spectra (both for period and displacement) for soil profile Site Classes A through E.

Site coordinates of 35.4200 degrees latitude and -120.8632 degrees longitude were used in the web-based probabilistic seismic hazard analysis (USGS, 2013). Based on the results from the in-situ tests performed during the field investigation, the Site was defined as **Site Class D**, “Stiff Soil” profile per ASCE7-10, Chapter 20. Relevant seismic design parameters obtained from the program area summarized in Table 2: Seismic Design Parameters. Refer to **Appendix C** for more information regarding the seismic hazard analysis performed for the project and detailed results.

Table 2: Seismic Design Parameters

Site Class	**D, “Stiff Soil”**
Seismic Design Category	**D**
1-Second Period Design Spectral Response Acceleration, S_{D1}	**0.438 g**
Short-Period Design Spectral Response Acceleration, S_{DS}	**0.789 g**
Site Specific MCE Peak Ground Acceleration, PGA_M	**0.466 g**

6.0 LIQUEFACTION HAZARD ASSESSMENT

Liquefaction occurs when saturated cohesionless soils lose shear strength due to earthquake shaking. Ground motion from an earthquake may induce cyclic reversals of shear stresses of large amplitude. Lateral and vertical movement of the soil mass combined with the loss of bearing strength can result from this phenomenon. Liquefaction potential of soil deposits during earthquake activity depends on soil type, void ratio, groundwater conditions, the duration of shaking, and confining pressures on the potentially liquefiable soil unit. Fine, poorly graded loose sand, shallow groundwater, high intensity earthquakes, and long duration of ground shaking are the principal factors leading to liquefaction.

Based on the consistency and relative density of the soils underlying the Site, the potential for liquefaction to occur is considered very low. These findings are consistent with that reported by Yeh and Associates (2016).

7.0 GENERAL SOIL-FOUNDATION DISCUSSION

In general, the subsurface materials at the site consist of stiff to very stiff sandy clay underlain by medium dense to very dense clayey sand with gravel and clayey gravel encountered at depths of 30 to 72 feet across the Site. Groundwater was encountered at depths of 23 and 28 feet below ground surface during the field investigation. Groundwater may be encountered during site grading for the proposed deep excavations at the site (aeration/membrane tank). The difficulty of working at or below groundwater is a primary concern. Any contractor working at the site needs to be prepared to work in this environment.

Based on our project understanding and site conditions, there is a potential for large settlements to occur as a result of significant loading associated with the proposed above grade recycled water storage tank. Settlements on the order of 6 to 9 inches were reported by Yeh and Associates (2016) for anticipated loading conditions at the Site and are consistent with our findings. In order to mitigate the potential for static settlements, it is recommended the recycled water storage utilize a concrete-ringwall with gravel in-fill foundation system supported by a geogrid reinforced fill pad. As an alternative, a mat slab foundation system founded in a geogrid reinforced fill pad may be used.

It is anticipated that the proposed light-weight structures (with maximum continuous footing and column loads estimated to be approximately 1.5 kips per linear foot and 15 kips) will be supported by conventional shallow foundations founded in engineered fill. All foundations are to be excavated into uniform material to limit the potential for distress of the foundation systems due to differential settlement. If cuts steeper than allowed by State of California Construction Safety Orders for “Excavations, Trenches, Earthwork” are proposed, a numerical slope stability analysis may be necessary for temporary construction slopes.

Natural seepage at the interface of two materials with different densities, such as native soil and engineered fill, is very common. This interface occurs at the Site and may require sub-surface drains. Sub-drains should be placed in established drainage courses, potential seepage areas, and during the development of all key and bench grading operations.

8.0 CONCLUSIONS AND RECOMMENDATIONS

The Site is suitable for the proposed development provided the recommendations presented in this report are incorporated into the project plans and specifications.

The primary geotechnical concerns at the Site are:

1. The potential for groundwater seepage.

2. The presence of loose surface and subsurface soils.

3. The potential for static settlement in the subsurface soils.

4. The presence of potentially expansive material. Influx of water from irrigation, leakage, or natural seepage could cause expansive soil problems. Foundations supported by expansive soils should be designed by a Structural Engineer in accordance with the 2016 California Building Code.

5. The potential for differential settlement occurring between foundations supported on two soil materials having different settlement characteristics, such as native soil and engineered fill. Therefore, it is important that all of the foundations are founded in equally competent uniform material in accordance with this report.

8.1 Preparation of Paved Areas

1. Pavement areas should be excavated to approximate sub-grade elevation or to competent material; whichever is deeper. The exposed surface should be scarified an

additional depth of 12 inches, moisture conditioned to 3% over optimum moisture content, and compacted to a minimum relative density of 95 percent (ASTM D1557-12$_{e1}$ test method). The top 12 inches of sub-grade soil under all pavement sections should be compacted to a minimum relative density of 95 percent based on the ASTM D1557-12$_{e1}$ test method at slightly above optimum.

2. Sub-grade soils should not be allowed to dry out or have excessive construction traffic between moisture conditioning and compaction, and placement of the pavement structural section.

3. Due to the expansive potential of the soils at the Site, the base courses beneath un-reinforced pavement sections may fail, causing cracking of the pavement surfaces, as the sub-grade materials move laterally during expansive shrink-swell cycles.

4. Therefore, in order to minimize the potential for failure of pavement sections at the Site, GeoSolutions, Inc. recommends that a laterally-reinforcing geotextile grid, such as Tensar BX1100, Syntec SBX11, ADS BX114GG, or equivalent, be installed to reinforce the base courses under paved areas at the Site.

5. GeoSolutions, Inc. should be contacted prior to the design and construction of pavement sections at the Site in order to assist in the selection of an appropriate laterally-reinforcing biaxial geogrid product and to provide recommendations regarding the procedures for the installation of geogrid products at the Site.

8.2 Pavement Design

1. All paving construction and materials used should conform to applicable sections of the latest edition of the State of California Department of Transportation Standard Specifications.

2. As indicated previously, the top 12 inches of sub-grade soil under asphaltic concrete pavement sections should be compacted to a minimum relative density of 95 percent based on the ASTM D1557-12$_{e1}$ test method at slightly above optimum moisture content. Aggregate bases and sub-bases should also be compacted to a minimum relative density of 95 percent based on the aforementioned test method.

3. Results of R-value testing performed on a sample of anticipated subgrade soils obtained during the field investigation indicated an R-Value of 4. It is anticipated a laterally-reinforcing geotextile grid, such as Tensar BX1100, Syntec SBX11, ADS BX114GG, or equivalent will be placed beneath the base course under paved areas to improve the subgrade soils. Table 3: Recommended Pavement Structural Sections provides the recommended Hot Mix Asphalt (HMA) pavement sections based on an R-Value of 4 (no geotextile grid) and an R-value of 20 (Caltrans 2016) for the subgrade soils.

4. All pavement sections should be crowned for good drainage. All pavement construction and materials used should conform to Sections 25, 26 and 39 of the latest edition of the State of California Department of Transportation Standard Specifications.

Table 3: Recommended Pavement Structural Sections

Traffic Index	Street Section Thickness in Inches			
	No Geotextile Grid		With Geotextile Grid	
	HMA [1]	AB [2]	HMA [1]	AB [2]
5.0	3.0	10.0	3.0	7.5
6.0	3.0	14.0	3.0	10.5
7.0	3.0	18.0	3.0	14.0

[1] HMA = Hot Mix Asphalt meeting Caltrans Spec. HMA Type A ½ inch mix
[2] AB = Aggregate Base meeting Caltrans Spec. for Class II aggregate base (R-Value = 78 Min)

8.3 Preparation of Geogrid Reinforced Fill Pad for Recycled Water Tank

1. It is anticipated a graded geogrid reinforced engineered fill will be developed for the recycled water tank with foundations founded in engineered fill.

2. For the development of a geogrid reinforced engineered fill pad, the on-site material should be over-excavated to a depth of **96 inches** (8 feet) below existing grade or to a depth of **72 inches** (6 feet) below the bottom of the footings, whichever is greatest. The limits of over-excavation should extend a minimum of 10 feet beyond the perimeter foundation where possible. The exposed surface should be scarified to a depth of 8 inches, moisture conditioned to approximately 3 percent over optimum value, and compacted to a minimum relative density of 95 percent (ASTM D1557-12_{e1}).

3. A Tensar TX7 geogrid or equivalent should be placed over the bottom of the tank pad excavation per manufacturer's specifications and with a 2-foot overlap where required. No traffic should be allowed directly on the geogrid. A maximum 12-inch section of engineered fill should then be placed over the geogrid and compacted to a minimum relative density of 90 percent (ASTM D1557-12_{e1}). Additional layers of geogrid separated by a maximum 12-inch section of engineered fill should be placed up to 12 inches below the bottom of the foundation system for a minimum of **6 layers** of geogrid. The final lift of engineered fill, extending up to finished grade should be compacted to a minimum relative density of 95 percent (ASTM D1557-12_{e1}).

4. The over-excavated, on-site material is **not** suitable for use as engineered fill. Imported non-expansive, granular material such as **Class II Base** may be used as engineered fill within the tank pad area. All material to be used as non-expansive, granular engineered fill must be observed and approved by a representative of GeoSolutions, Inc. prior to its delivery to the Site. Refer to **Appendix D** for more details on fill placement.

8.4 Preparation of Equalization Basin Area

1. It is anticipated that grading for the proposed HDPE-lined equalization basin will consist of excavating the existing soils to proposed subgrade, as observed and approved by a representative of GeoSolutions, Inc.

2. The exposed surface should be scarified to a depth of 8 inches, moisture conditioned to 3% over optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-12_{e1}). The over-excavated material should then be processed as engineered fill. Refer to **Appendix D** for more details on fill placement.

3. It is recommended the slopes for the basin not exceed gradients steeper than 3-to-1 (horizontal-to-vertical). However, based on discussions with the project team, gradients of

up to 2-to-1 (horizontal-to-vertical) are desired for the project. Gradients of up to 2-to-1 (horizontal-to-vertical) may be used at the Site but could cause expensive and/or extended maintenance requirements.

8.5 Preparation of Aeration/Membrane Tank Pad

1. It is anticipated that proposed aeration/membrane tank pad area will be excavated to proposed subgrade and founded in engineered fill, as observed and approved by a representative of GeoSolutions, Inc.

2. The exposed surface should be scarified to a depth of 8 inches, moisture conditioned to 3% over optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-12_{e1}). The over-excavated material should then be processed as engineered fill. Refer to **Appendix D** for more details on fill placement.

3. For areas that are saturated or pumping, additional excavation and or rock stabilization will be required. The typical method of stabilization is to place 3 to 6 inch crushed stone into the saturated soil followed by smaller diameter rock (3/4 to 1 ½ inch crushed). Then a non-woven fabric or geogrid is placed over the aggregate, followed by 24 to 36 inches of Class II base.

8.6 Preparation of Building Pads for Light-Weight Structures

1. It is anticipated that graded engineered fill pads will be developed for the proposed light-weight structures (office/maintenance buildings) with footings founded in engineered fill.

2. For the development of an engineered fill pad, the native material should be over-excavated at least **60 inches** below existing grade, **24 inches** below the bottom of the footings, to competent material, or to two-thirds the depth of the deepest fill (measured from the bottom of the deepest footing); whichever is greatest. The limits of over-excavation should extend a minimum of 5 feet beyond the perimeter foundation, to property lines, or existing improvements, whichever is least.

3. The exposed surface should be scarified to a depth of 6 inches; moisture conditioned to 3% over optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-12_{e1}). The over-excavated material may then be processed as engineered fill. Onsite soil and rock material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and other particles. Imported fill should meet the requirements of the grading plan. GeoSolutions, Inc. should be notified at least 72 hours prior to delivery to the site to sample and test proposed imported fill materials. Refer to (Section 8.11) for under-slab drainage material and **Appendix D** for more details on fill placement.

4. The recommended soil moisture content should be maintained during construction and following construction of the proposed development. Where soil moisture content is not maintained, desiccation cracks may develop which indicate a loss of soil compaction, leading to the potential for damage to foundations, flatwork, pavements, and other improvements. Soils that have become cracked due to moisture loss should be removed sufficient depth to repair the cracked soil as observed by the soils engineer, and the removed materials should then be moisture conditioned to approximately **3 percent over optimum value**, and compacted.

8.7 Preparation of Industrial Equipment Pad Areas

1. It is anticipated that industrial pad areas will be excavated to proposed bottom of foundation and founded in engineered fill, as observed and approved by a representative of GeoSolutions, Inc.

2. The exposed surface should be scarified to a depth of 8 inches, moisture conditioned to 3% over optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-12$_{e1}$). The over-excavated material should then be processed as engineered fill. Refer to **Appendix D** for more details on fill placement.

8.8 Concrete Ringwall Foundation System

1. It is anticipated a concrete ringwall with gravel in-fill foundation system will be used to support the recycled water tank with all foundations founded in imported, non-expansive engineered fill placed in accordance with section 8.3. An aerial loading of **1,500 psf** was assumed for the concrete-ringwall with gravel in-fill foundation system.

2. Foundations must be designed in accordance to section 1808.6, 2016 CBC, Foundations on Expansive Soils.

3. Minimum footing and grade beam sizes and depths in engineered fill should conform to Table 4: Minimum Concrete Ringwall Footing Recommendations, as observed and approved by a representative of GeoSolutions, Inc.

Table 4: Minimum Concrete Ringwall Footing Recommendations

Design Parameter	Perimeter Footings
Minimum Width	15 inches
Embedment Depth	24 inches
Minimum Reinforcing*	4 #5 bars (2 top / 2 bottom)
* Steel should be held in place by stirrups at appropriate spacing to ensure proper positioning of the steel (see WRI Design of Slab-on-Ground Foundations).	

4. Minimum reinforcing for footings should conform to the recommendations provided in Table 5: Minimum Slab Recommendations which meets the specifications of Section 1808.6 of the 2016 California Building Code for the soil conditions at the Site. Reinforcing steel should be held in place by stirrups at appropriate spacing to ensure proper positioning of the steel in accordance with WRI Design of Slab-on-Ground Foundations, and ACI 318, Section 7.5 – Placing Reinforcement.

5. A representative of this firm should observe and approve all foundation excavations for required embedment depth prior to the placement of reinforcing steel and/or concrete. Concrete should be placed only in excavations that are free of loose, soft soil and debris and that have been maintained in a moist condition with no desiccation cracks present.

6. An allowable dead plus live load bearing pressure of **2,000 psf** may be used for the design of the perimeter footings founded in imported, non-expansive engineered fill. Allowable bearing capacities may be increased by one-third when transient loads such as wind and/or seismicity are included.

7. Differential and total settlement of less than 1 inch are anticipated.

8. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the engineered fill and the bottom of the footings. For resistance to lateral loads, a friction factor of **0.40** may be utilized for sliding resistance at the base of footings extending a minimum of 24 inches into imported, non-expansive engineered fill. A passive pressure of **350 pcf** equivalent fluid weight may be used against the side of shallow footings in imported, non-expansive engineered fill. If friction and passive pressures are combined to resist lateral forces acting on shallow footings, the lesser value should be reduced by 50 percent.

9. Foundation excavations should be observed and approved by a representative of this firm prior to the placement of reinforcing steel and/or concrete.

10. Foundation design should conform to the requirements of Chapter 18 of the latest edition of the CBC (CBSC, 2016).

11. The base of all footings should be level and stepped as required to accommodate any change in grade while still maintaining the minimum required footing embedment and slope setback distance.

12. A minimum 24inch section of ¾-inch crushed rock be placed within the ringwall footing. ½-inch crushed rock can be used from the top of the ¾-inch crushed rock section to finished pad elevation.

8.9 Mat Foundations

1. Mat slab foundations may be used to support the aeration/membrane tanks, industrial equipment, and the recycled water tank as an alternative to the concrete ringwall with gravel in-fill foundation system. All foundations will be founded in engineered fill in accordance with the recommendations provided in this report

2. Based on our experience, a mat slabs may be approximately 12-15 inches thick. Differential and total settlements of less than 1 inch should be anticipated. Minimum reinforcing should be as directed by the project Structural Engineer.

3. A modulus of sub-grade reaction (k_s) of **100 pci** may be used in design. Allowable dead plus live load bearing pressure of **1,500 psf** may be used for design of mat foundations.

4. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the native material and the bottom of the footings. For resistance to lateral loads, a friction factor of **0.30** may be utilized for sliding resistance at the base of footings.

5. Foundation excavations should be observed and approved by a representative of this firm prior to the placement of reinforcing steel and/or concrete.

6. Foundation design should conform to the requirements of Chapter 18 of the latest edition of the California Building Code.

8.10 Conventional Foundations

1. Conventional continuous and spread footings with grade beams may be used for support lightweight structures at the Site. Foundations must be designed in accordance to section 1808.6, 2016 CBC, Foundations on Expansive Soils.

2. Minimum footing and grade beam sizes and depths in engineered fill should conform to the following table, as observed and approved by a representative of GeoSolutions, Inc.

3. Minimum reinforcing for footings should conform to the recommendations provided in Table 5: Minimum Slab Recommendations which meets the specifications of Section 1808.6 of the 2016 California Building Code for the soil conditions at the Site. Reinforcing steel should be held in place by stirrups at appropriate spacing to ensure proper positioning of the steel in accordance with WRI Design of Slab-on-Ground Foundations, and ACI 318, Section 7.5 – Placing Reinforcement.

Table 3: Minimum Footing and Grade Beam Recommendations

Design Parameter	Perimeter Footings	Grade Beams
Minimum Width	12 inches (one story) 15 inches (two story)	12 inches
Embedment Depth	30 inches	18 inches
Minimum Reinforcing*	6 #5 bars (3 top / 3 bottom)	4 #5 bars (2 top / 2 bottom)
Spacing	-	16 feet on-center each way
***** Steel should be held in place by stirrups at appropriate spacing to ensure proper positioning of the steel (see WRI Design of Slab-on-Ground Foundations).		

4. A representative of this firm should observe and approve all foundation excavations for required embedment depth prior to the placement of reinforcing steel and/or concrete. Concrete should be placed only in excavations that are free of loose, soft soil and debris and that have been maintained in a moist condition with no desiccation cracks present.

5. An allowable dead plus live load bearing pressure of **2,000 psf** may be used for the design of footings founded in engineered fill.

6. Allowable bearing capacities may be increased by one-third when transient loads such as wind and/or seismicity are included.

7. A total settlement of less than 1 inch and a differential settlement of less than 1 inch in 30 feet are anticipated.

8. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the engineered fill and the bottom of the footings. For resistance to lateral loads, a friction factor of **0.30** may be utilized for sliding resistance at the base of footings extending a minimum of 24 inches into engineered fill. A passive pressure of **275-pcf** equivalent fluid weight may be used against the side of shallow footings in engineered fill. If friction and passive pressures are combined to resist lateral forces acting on shallow footings, the lesser value should be reduced by 50 percent.

9. Foundation excavations should be observed and approved by a representative of this firm prior to the placement of reinforcing steel and/or concrete.

10. Foundation design should conform to the requirements of Chapter 18 of the latest edition of the CBC (CBSC, 2016).

11. The base of all grade beams and footings should be level and stepped as required to accommodate any change in grade while still maintaining the minimum required footing embedment and slope setback distance.

8.11 Slab-On-Grade Construction

2. Concrete slabs-on-grade and flatwork should not be placed directly on unprepared native materials. Preparation of sub-grade to receive concrete slabs-on-grade and flatwork should be processed as discussed in the preceding sections of this report. Concrete slabs should be placed only over sub-grade that is free of loose, soft soil and debris and that has been maintained in a moist condition with no desiccation cracks present.

3. Concrete slabs-on-grade should be in conformance with the recommendations provided in Table 5: Minimum Slab Recommendations. Reinforcing should be placed on-center both ways at or slightly above the center of the structural section. Reinforcing bars should have a minimum clear cover of 1.5 inches. Where lapping of the slab steel is required, laps in adjacent bars should be staggered a minimum of every five feet (see WRI Design of Slab-on-Ground Foundations, Steel Placement). The recommended reinforcement may be used for anticipated uniform floor loads not exceeding 200 psf. If floor loads greater than 200 psf are anticipated, a Structural Engineer should evaluate the slab design.

Table 5: Minimum Slab Recommendations

Minimum Thickness	5 inches
Reinforcing*	#4 bars at 16 inches on-center each way
* Where lapping of the slab steel is required, laps in adjacent bars should be staggered a minimum of every five feet (see WRI/CSRI-81 recommendations for Steel Placement)	

4. Concrete for all slabs should be placed at a maximum slump of less than 5 inches. Excessive water content is the major cause of concrete cracking. If fibers are used to aid in the control of cracking, a water-reducing admixture may be added to the concrete to increase slump while maintaining a water/cement ratio, which will limit excessive shrinkage. Control joints should be constructed as required to control cracking.

5. Where concrete slabs-on-grade are to be constructed for interior conditioned spaces, the slabs should be underlain by a minimum of four inches of clean free-draining material, such as a ½ inch coarse aggregate mix, to serve as a cushion and a capillary break. Where moisture susceptible storage or floor coverings are anticipated, a 15-mil Stego Wrap membrane (or equivalent installed per manufacturer's specifications) should be placed between the free-draining material and the slab to minimize moisture condensation under the floor covering. See Figure 4: Sub-Slab Detail for the placement of under-slab drainage material. It is suggested, but not required, that a two-inch thick sand layer be placed on top of the membrane to assist in the curing of the concrete, increasing the depth of the under-slab material to a total of six inches. The sand should be lightly moistened prior to placing concrete.

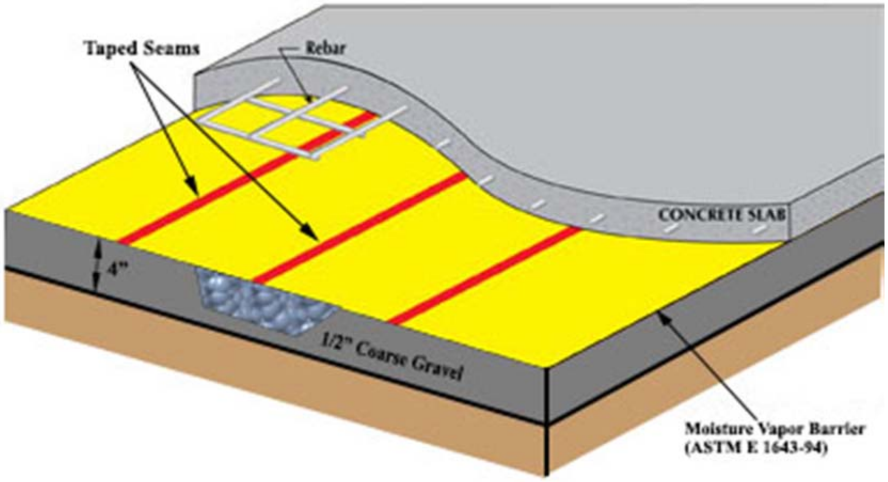

Figure 4: Sub-Slab Detail

6. It should be noted that for a vapor barrier installation to conform to manufacturer's specifications, sealing of penetrations, joints and edges of the vapor barrier membrane are typically required. As required by the California Building Code, joints in the vapor barrier should be lapped a minimum of 6 inches. If the installation is not performed in accordance with the manufacturer's specifications, there is an increased potential for water vapor to affect the concrete slabs and floor coverings.

7. The most effective method of reducing the potential for moisture vapor transmission through concrete slabs-on-grade would be to place the concrete directly on the surface of the vapor barrier membrane. However, this method requires a concrete mix design specific to this application with low water-cement ratio in addition to special concrete finishing and curing practices, to minimize the potential for concrete cracks and surface defects. The contractor should be familiar with current techniques to finish slabs poured directly onto the vapor barrier membrane.

8. Moisture condensation under floor coverings has become critical due to the use of water-soluble adhesives. Therefore, it is suggested that moisture sensitive slabs not be constructed during inclement weather conditions.

8.12 Exterior Concrete Flatwork

1. Due to the presence of expansive surface soils within the proposed development areas, there is a potential for considerable soil movement and distress to reinforced concrete flatwork if conventional measures are used, such as the placement of 4 to 6 inches of imported sand materials placed beneath concrete flatwork. Heaving and cracking are anticipated to occur. To reduce the potential for movement associated with expansive soils, we recommend the placement of a minimum of **24 inches of approved non-expansive import material placed as engineered fill beneath the flatwork**.

2. The exposed surface should be scarified to a depth of 6 inches; moisture conditioned to **3% over optimum moisture content**, and compacted to a minimum relative density of 90 percent (ASTM D1557-12e1).

7. Minimum flatwork for conventional pedestrian areas should be a minimum of 4 inches thick and consist of No. 3 (#3) rebar spaced at 24 inches on-center each-way at or slightly above the center of the structural section. Concrete should be placed only in excavations that have been kept moist and are free of loose, soft soil or debris.

3. Flatwork should be constructed with frequent joints to allow for movement due to fluctuations in temperature and moisture content in the adjacent soils. Flatwork at doorways, driveways, curbs and other areas where restraining the elevation of the flatwork is desired, should be doweled to the perimeter foundation by a minimum of No. 3 reinforcing steel dowels, spaced at a maximum distance of 24 inches on-center.

4. As an alternative, interlocking concrete pavers may be utilized for exterior improvements in lieu of reinforced concrete flatwork. Concrete pavers, when installed in accordance with manufacturers' recommendations and industry standards (ICPI), allow for a greater degree of soil movement as they are part of a flexible system. If interlocking concrete pavers are selected for use in the driveway area, the structural section should be underlain by a woven geotextile fabric, such as Mirafi 500x or equivalent, to function as a separation layer and to provide additional support for vehicle tire loads.

8.13 Retaining Walls

1. Retaining walls should be designed to resist lateral pressures from adjacent soils and surcharge loads applied behind the walls. We recommend using the lateral pressures presented in Table 6: Retaining Wall Design Parameters and Figure 5: Retaining Wall Detail for the design of retaining walls at the Site. The Active Case may be used for the design of unrestrained retaining walls, and the At-Rest Case may be used for the design of restrained retaining walls.

Table 6: Retaining Wall Design Parameters

Lateral Pressure and Condition	Equivalent Fluid Pressure, pcf
Static, Active Case, Engineered Fill or Competent Native Material ($\gamma' K_A$)	50
Static, At-Rest Case, Engineered Fill or Competent Native Material ($\gamma' K_O$)	70
Static, Passive Case, Engineered Fill or Competent Native Material ($\gamma' K_P$)	275

2. The above values for equivalent fluid pressure are based on retaining walls having level retained surfaces, having an approximately vertical surface against the retained material, and retaining granular backfill material or engineered fill composed of native soil within the active wedge. See Figure 5: Retaining Wall Detail and Figure 6: Retaining Wall Active and Passive Wedges for a description of the location of the active wedge behind a retaining wall.

3. For cantilever walls, import granular material can be used within the active wedge (Figure 6), extending at a 1-to-1 slope up from the base of the wall, to reduce the lateral earth pressures. An active earth pressure, K_a, equal to **30 pcf** can be used for the import granular material placed within the active

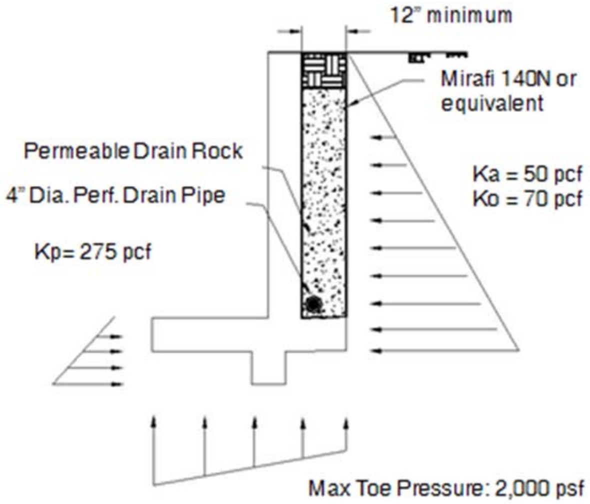

Figure 5: Retaining Wall Detail

wedge, as observed and approve by a representative of GeoSolutions, Inc.

4. Proposed retaining walls having a retained surface that slopes upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for the active case and 1.5 pcf for the at-rest case, for every degree of slope inclination.

   We recommend that the proposed retaining walls at the Site have an approximately vertical surface against the retained material. If the proposed retaining walls are to have sloped surfaces against the retained material, the project designers should contact the Soils Engineer to determine the appropriate lateral earth pressure values for retaining walls located at the Site.

5. Retaining wall foundations should be founded a minimum of 24 inches below lowest adjacent grade in engineered fill as observed and approved by a representative of GeoSolutions, Inc. A coefficient of friction of **0.30** may be used between engineered fill and concrete footings. Project designers may use a maximum toe pressure of **2,000 psf** for the design of retaining wall footings founded in engineered fill.

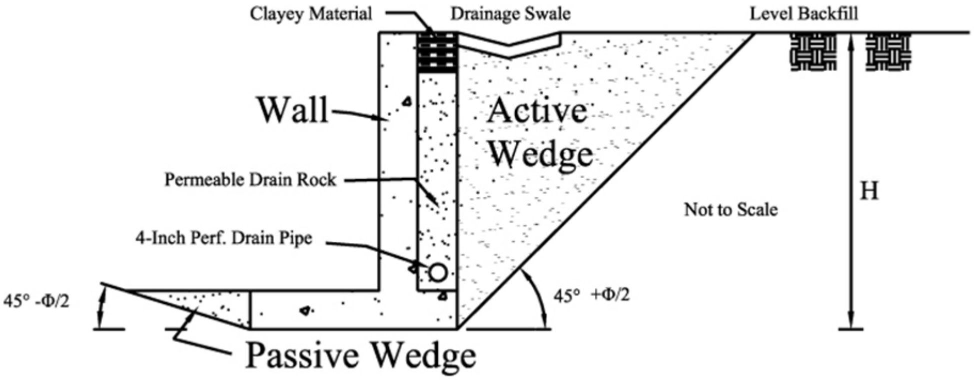

Figure 6: Retaining Wall Active and Passive Wedges

6. For earthquake conditions, unrestrained walls (active condition) greater than 6 feet in height should be designed to resist an additional seismic lateral soil pressure of **30 pcf** equivalent fluid pressure considering native backfill conditions, or **20 pcf** for granular import wall backfill conditions. The pressure resultant force from earthquake loading should be assumed to act a distance of $^1/_3 H$ above the base of the retaining wall, where *H* is the height of the retaining wall. Seismic active lateral earth pressure values were determined using the simplified dynamic lateral force component (SEAOC 2010) utilizing the design peak ground acceleration, PGA_M, discussed in Section 4.3 (**PGA_M = 0.466 g**). The dynamic increment in lateral earth pressure due to earthquakes should be considered during the design of retaining walls at the Site. Based on research presented by Dr. Marshall Lew (Lew et al., 2010), lateral pressures associated with seismic forces should not be applied to restrained walls (at-rest condition).

7. Seismically induced forces on retaining walls are considered to be short-term loadings. Therefore, when performing seismic analyses for the design of retaining wall footings, we recommend that the allowable bearing pressure and the passive pressure acting against the sides of retaining wall footings be increased by a factor of one-third.

8. In addition to the static lateral soil pressure values reported in Table 6: Retaining Wall Design Parameters, the retaining walls at the Site should be designed to support any design live load, such as from vehicle and construction surcharges, etc., to be supported

by the wall backfill. If construction vehicles are required to operate within 10 feet of a retaining wall, supplemental pressures will be induced and should be taken into account in the design of the retaining wall.

9. The recommended lateral earth pressure values are based on the assumption that sufficient sub-surface drainage will be provided behind the walls to prevent the build-up of hydrostatic pressure. To achieve this we recommend that a granular filter material be placed behind all proposed walls. The blanket of granular filter material should be a minimum of 12 inches thick and should extend from the bottom of the wall to 12 inches from the ground surface. The top 12 inches should consist of moisture conditioned, compacted, clayey soil. Neither spread nor wall footings should be founded in the granular filter material used as backfill.

10. A 4-inch diameter perforated or slotted drainpipe (ASTM D1785 PVC) should be installed near the bottom of the filter blanket with perforations facing down. The drainpipe should be underlain by at least 4 inches of filter type material and should daylight to discharge in suitably projected outlets with adequate gradients. The filter material should consist of a clean free-draining aggregate, such as a coarse aggregate mix. If the retaining wall is part of a structural foundation, the drainpipe must be placed below finished slab sub-grade elevation.

11. The filter material should be encapsulated in a permeable geotextile fabric. A suitable permeable geotextile fabric, such as non-woven needle-punched Mirafi 140N or equal, may be utilized to encapsulate the retaining wall drain material and should conform to Caltrans Standard Specification 88-1.03 for underdrains.

12. For hydrostatic loading conditions (i.e. no free drainage behind retaining wall), an additional loading of 45-pcf equivalent fluid weight should be added to the active and at-rest lateral earth pressures. If it is necessary to design retaining structures for submerged conditions, the allowed bearing and passive pressures should be reduced by 50 percent. In addition, soil friction beneath the base of the foundations should be neglected.

13. Precautions should be taken to ensure that heavy compaction equipment is not used adjacent to walls, so as to prevent undue pressure against, and movement of the walls.

14. The use of water-stops/impermeable barriers should be used for any basement construction, and for building walls that retain earth.

8.14 Temporary Cut Slopes

1. Temporary cut slopes are anticipated to facilitate construction of the aeration/membrane tanks founded approximately 20 feet below ground surface.

2. Preliminarily, for temporary construction slopes on the order of 20 feet total height, a 10-foot-high 1-to-1 cut slope over a 10-foot-high vertical cut slope can be used, as observed and approved by a representative of GeoSolutions, Inc. At the time of construction, a representative of GeoSolutions, Inc. should observe the resulting cut slopes to verify field conditions. Additional recommendations may be provided at that time.

8.15 Temporary Shoring

If cut slope configurations during construction differ from the preliminary assumptions provided in this report, temporary shoring may be required for construction of the proposed aeration/membrane tanks.

Cantilever Shoring:

1. For design of cantilever shoring an equivalent fluid pressure of **35 pcf** may be used in design.

2. In addition to the recommended earth pressures, the upper 10 feet of shoring adjacent to streets or vehicle traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot. If traffic is kept at least 10 feet away, the surcharge may be neglected.

3. For hydrostatic loading conditions, an additional loading of **45 pcf** equivalent fluid weight should be added to the active and at-rest lateral earth pressures.

Anchored Shoring:

1. Tied-back friction anchors may used to resist lateral loads. It is recommended that anchors extend at least 10 feet beyond a 1-to-1 line projecting upward from 2 feet below the bottom of the excavation, see Figure 7: Anchored Shoring.

2. Soldier piles with lateral support (anchors) may be designed for a rectangular pressure (psf) distribution of 35 H, where H is the height of shoring in feet.

3. A friction value of 10 psi may be used for native soil and grout bond strength. Higher values may be obtained but must be demonstrated by testing a minimum of 2 anchors.

4. As an alternative, helical type anchors have been used as a method of additional lateral support. Helical type anchors have varied strength capabilities based on the size of the anchor (i.e. 8 inch or double 6 and 10 inch, etc).

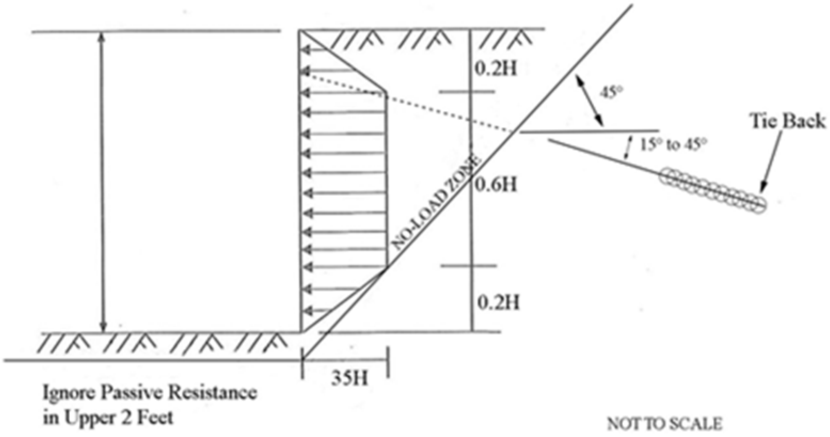

Figure 7: Anchored Shoring

9.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations contained in this report are based on a limited number of borings and on the continuity of the sub-surface conditions encountered. GeoSolutions, Inc. assumes that it will be retained to provide additional services during future phases of the proposed project. These services would be provided by GeoSolutions, Inc. as required by County of San Luis Obispo, the 2016 CBC, and/or industry standard practices. These services would be in addition to those included in this report and would include, but are not limited to, the following services:

1. Consultation during plan development.

2. Plan review of grading and foundation documents prior to construction and a report certifying that the reviewed plans are in conformance with our geotechnical recommendations.

3. Consultation during selection and placement of a laterally-reinforcing biaxial geogrid product.

4. Construction inspections and testing, as required, during all grading and excavating operations beginning with the stripping of vegetation at the Site, at which time a site meeting or pre-job meeting would be appropriate.

5. Special inspection services during construction of reinforced concrete, structural masonry, high strength bolting, epoxy embedment of threaded rods and reinforcing steel, and welding of structural steel.

6. Preparation of construction reports certifying that building pad preparation and foundation excavations are in conformance with our geotechnical recommendations.

7. Preparation of special inspection reports as required during construction.

8. In addition to the construction inspections listed above, section 1705.6 of the 2016 CBC (CBSC, 2016) requires the following inspections by the Soils Engineer for controlled fill thicknesses greater than 12 inches as shown in Table 7: Required Verification and Inspections of Soils:

Table 7: Required Verification and Inspections of Soils

Verification and Inspection Task	Continuous During Task Listed	Periodically During Task Listed
1. Verify materials below footings are adequate to achieve the design bearing capacity.	-	X
2. Verify excavations are extended to proper depth and have reached proper material.	-	X
3. Perform classification and testing of controlled fill materials.	-	X
4. Verify use of proper materials, densities and lift thicknesses during placement and compaction of controlled fill.	X	-
5. Prior to placement of controlled fill, observe sub-grade and verify that site has been prepared properly.	-	X

10.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed during our study. Should any variations or undesirable conditions be encountered during the development of the Site, GeoSolutions, Inc. should be notified immediately and GeoSolutions, Inc. will provide supplemental recommendations as dictated by the field conditions.

2. This report is issued with the understanding that it is the responsibility of the owner or his/her representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project, and incorporated into the project plans and specifications. The owner or his/her representative is responsible to ensure that the

necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. As of the present date, the findings of this report are valid for the property studied. With the passage of time, changes in the conditions of a property can occur whether they are due to natural processes or to the works of man on this or adjacent properties. Therefore, this report should not be relied upon after a period of 3 years without our review nor should it be used or is it applicable for any properties other than those studied. However many events such as floods, earthquakes, grading of the adjacent properties and building and municipal code changes could render sections of this report invalid in less than 3 years.

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REFERENCES

REFERENCES

American Society of Civil Engineers (ASCE). *Minimum Design Loads for Buildings and Other Structures,* ASCE Standard 7-10, ASCE, Reston, VA, 2013.

California Building Standards Commission (CBSC). (2016). 2016 California Building Code, California Code of Regulations, Title 24. Part 2, Vol. 2.

DeLorme. *Topo USA 8.0*. Vers.8.0.0 Computer software. DeLorme, 2009.

Dibblee, Thomas W., Jr.. *Geologic Map of the Cayucos North Quadrangle*. Dibblee Geologic Center Map Number DF-216. Santa Barbara Museum of Natural History: April 2006.

GeoInsite, Engineering Geologic Hazards Evaluation for Environmental Impact Report, Cayucos Sustainable Water Project, San Luis Obispo County, California, Project No. C1510A, dated October 17, 2016.

Lew, M., Sitar, N., Al Atik, L., Paourzanjani, M., and Hudson, M. "Seismic Earth pressure on Deep Building Basements," SEAOC 2010 Convention Proceedings, 2010.

State of California. Department of Industrial Relations*. California Code of Regulations.* 2001 Edition. Title 8. Chapter 4: Division of Industrial Safety. Subchapter 4, Construction Safety Orders. Article 6: Excavations. http://www.dir.ca.gov/title8/sub4.html.

State of California, Department of Transportation (Caltrans). *Highway Design Manual*. State of California Department of Transportation Central Publication Distribution Unit, 2016.

State of California, Department of Transportation (Caltrans). *Standard Specifications*. State of California Department of Transportation Central Publication Distribution Unit, 2015.

United States Geological Survey (USGS) Geologic Hazards Science Center, *U.S. Seismic Design Maps*, accessed April 10, 2017. <http://geohazards.usgs.gov/designmaps/us/application.php>.

United States Geological Survey. *MapView – Geologic Maps of the Nation*. Internet Application. USGS, accessed April 17, 2017. <http://ngmdb.usgs.gov/maps/MapView/>.

Wire Reinforcement Institute, Design of Slab-on-Ground Foundations, A Design, Construction $ Inspection Aid for Consulting Engineers, TF 700-R-03 Update, dated 2003.

Yeh and Associates, Preliminary Geotechnical Report, Cayucos Sustainable Water Project, Toro Site 5, Toro Creek Road, Cayucos, San Luis Obispo County, California, dated May 13, 2016.

APPENDIX A

Field Investigation

Soil Classification Chart

Boring Logs

FIELD INVESTIGATION

The field investigation was conducted on March 9, 2017, using a track-mounted CME 55 drill rig. The surface and sub-surface conditions were studied by advancing two exploratory borings. This exploration was conducted in accordance with presently accepted geotechnical engineering procedures consistent with the scope of the services authorized to GeoSolutions, Inc.

The CME 55 drill rig with an eight -inch diameter hollow-stem continuous flight auger bored two exploratory borings near the approximate locations indicated on Figure 2: Field Exploration Plan. The drilling and field observation was performed under the direction of the project engineer. A representative of GeoSolutions, Inc. maintained a log of the soil conditions and obtained soil samples suitable for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System. See the Soil Classification Chart in this appendix.

Standard Penetration Tests with a two-inch outside diameter standard split tube sampler (SPT) without liners (ASTM D1586-99) and a three-inch outside diameter Modified California (CA) split tube sampler with liners (ASTM D3550-01) were performed to obtain field indication of the in-situ density of the soil and to allow visual observation of at least a portion of the soil column. Soil samples obtained with the split spoon sampler are retained for further observation and testing. The split spoon samples are driven by a 140-pound hammer free falling 30 inches. The sampler is initially seated six inches to penetrate any loose cuttings and is then driven an additional 12 inches with the results recorded in the boring logs as N-values, which area the number of blows per foot required to advance the sample the final 12 inches.

The CA sampler is a larger diameter sampler than the standard (SPT) sampler with a two-inch outside diameter and provides additional material for normal geotechnical testing such as in-situ shear and consolidation testing. Either sampler may be used in the field investigation, but the N-values obtained from using the CA sampler will be greater than that of the SPT. The N-values for samples collected using the CA can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. A commonly used conversion factor is 0.67 $({}^{2}/_{3})$. More information about standardized samplers can be found in ASTM D1586-99 and ASTM D3550-01.

Disturbed bulk samples are obtained from cuttings developed during boring operations. The bulk samples are selected for classification and testing purposes and may represent a mixture of soils within the noted depths. Recovered samples are placed in transport containers and returned to the laboratory for further classification and testing.

Logs of the borings showing the approximate depths and descriptions of the encountered soils, applicable geologic structures, recorded N-values, and the results of laboratory tests are presented in this appendix. The logs represent the interpretation of field logs and field tests as well as the interpolation of soil conditions between samples. The results of laboratory observations and tests are also included in the boring logs. The stratification lines recorded in the boring logs represent the approximate boundaries between the surface soil types. However, the actual transition between soil types may be gradual or varied.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS		LABORATORY CLASSIFICATION CRITERIA		GROUP SYMBOLS	PRIMARY DIVISIONS
COARSE GRAINED SOILS More than 50% retained on No. 200 sieve	GRAVELS More than 50% of coarse fraction retainined on No. 4 (4.75mm) sieve	Clean gravels (less than 5% fines*)	C_u greater than 4 and C_z between 1 and 3	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			Not meeting both criteria for GW	GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with fines (more than 12% fines*)	Atterberg limits plot below "A" line or plasticity index less than 4	GM	Silty gravels, gravel-sand-silt mixtures
			Atterberg limits plot below "A" line and plasticity index greater than 7	GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS More than 50% of coarse fraction passes No. 4 (4.75mm) sieve	Clean sand (less than 5% fines*)	C_u greater than 6 and C_z between 1 and 3	SW	Well graded sands, gravely sands, little or no fines
			Not meeting both criteria for SW	SP	Poorly graded sands and gravelly and sands, little or no fines
		Sand with fines (more than 12% fines*)	Atterberg limits plot below "A" line or plasticity index less than 4	SM	Silty sands, sand-silt mixtures
			Atterberg limits plot above "A" line and plasticity index greater than 7	SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS 50% or more passes No. 200 sieve	SILTS AND CLAYS (liquid limit less than 50)	Inorganic soil	PI < 4 or plots below "A"-line	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
		Inorganic soil	PI > 7 and plots on or above "A" line**	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic Soil	LL (oven dried)/LL (not dried) < 0.75	OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (liquid limit 50 or more)	Inorganic soil	Plots below "A" line	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		Inorganic soil	Plots on or above "A" line	CH	Inorganic clays of high plasticity, fat clays
		Organic Soil	LL (oven dried)/LL (not dried) < 0.75	OH	Organic silts and organic clays of high plasticity
Peat	Highly Organic	Primarily organic matter, dark in color, and organic odor		PT	Peat, muck and other highly organic soils

*Fines are those soil particles that pass the No. 200 sieve. For gravels and sands with between 5 and 12% fines, use of dual symbols is required (I.e. GW-GM, GW-GC, GP-GM, or GP-GC).

**If the plasticity index is between 4 and 7 and it plots above the "A" line, then dual symbols (I.e. CL-ML) are required.

the "A" line, then dual symbols (I.e. CL-ML) are required.

CONSISTENCY

CLAYS AND PLASTIC SILTS	STRENGTH TON/SQ. FT ++	BLOWS/ FOOT +
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	Over 4	Over 32

RELATIVE DENSITY

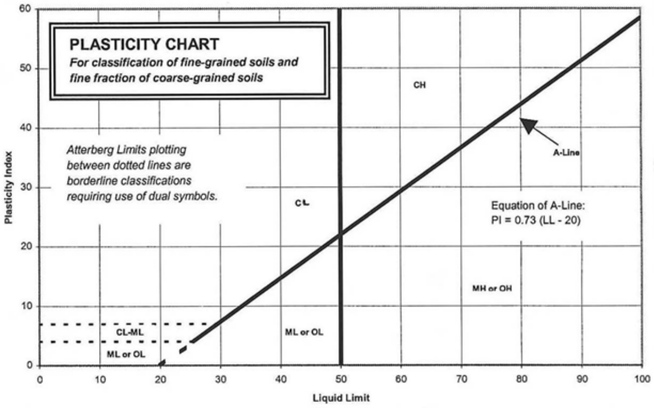
SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/ FOOT +
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	Over 50

+ Number of blows of a 140-pound hammer falling 30-inches to drive a 2-inch O.D. (1-3/8-inch I.D.) split spoon (ASTM D1586).

++ Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D1586), pocket penetrometer, torvane, or visual observation.

CLASSIFICATIONS BASED ON PERCENTAGE OF FINES

Less than 5%, Pass No. 200 (75mm)sieve)	GW, GP, SW, SP
More than 12% Pass N. 200 (75 mm) sieve	GM, GC, SM, SC
5%-12% Pass No. 200 (75 mm) sieve	Borderline Classification requiring use of dual symbols



Drilling Notes:

1. Sampling and blow counts
   a. California Modified – number of blows per foot of a 140 pound hammer falling 30 inches
   b. Standard Penetration Test – number of blows per 12 inches of a 140 pound hammer falling 30 inches

Types of Samples:
X – Sample
SPT - Standard Penetration
CA - California Modified
N - Nuclear Gauge
PO – Pocket Penetrometer (tons/sq.ft.)

GeoSolutions, Inc.

220 High Street, San Luis Obispo, CA 93401
1021 West Tama Lane, Suite 105
Santa Maria, CA 93454

BORING LOG

BORING NO. **S-1**

JOB NO. **SL10070-1**

PROJECT INFORMATION	DRILLING INFORMATION
PROJECT: **Cayucos Sustainable Water Project**	DRILL RIG: **CME 55**
DRILLING LOCATION: **See Figure 2: Exploration Location Plan**	HOLE DIAMETER: **8 Inches**
DATE DRILLED: **March 9, 2017**	SAMPLING METHOD: **CA or SPT**
LOGGED BY: **K. Robinson**	APPROX. ELEVATION: **+85 Feet**

▼ Depth of Groundwater: **28 Feet** Boring Terminated At: **50 Feet** Page 1 of 2

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	(N1) 60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PLASTICITY INDEX (PI)	EXPANSION INDEX (EI)	MAX DRY DENSITY (pcf)	OPT WATER CONTENT (%)	FRICTION ANGLE, PHI (degrees)	COHESION, C (psf)
0	SANDY CLAY: dark brown, slightly moist, fine to medium sand, fat clay, with some gravel, fine to coarse, subangular - ALLUVIUM (Qal)	CL												
-1														
-2				C @ 2-5'				65.0	31	74				
-3														
-4														
-5	5': very stiff, PP = 3 - 3.5 tsf			CA	28	39							24.3	911
-6														
-7														
-8														
-9														
-10	10': very stiff, PP = 3.5 tsf			SPT	17	25								
-11	SANDY CLAY: yellowish brown, very stiff, slightly moist, with light brown mottling, with some fine angular gravel - ALLUVIUM (Qal)	CL		D @ 11-15'				70.8	34	53				
-12														
-13														
-14														
-15	15': PP = 2.25 - 2.5 tsf			CA	29	27	10.3							
-16														
-17														
-18														
-19														
-20	20': PP = 2.5 - 3 tsf			SPT	19	22	16.5							
-21														
-22	22': softer drilling noted at around 22'													
-23														

BORING LOG

BORING NO (cont).**S-1**

JOB NO. **SL10070-1**

▼ Depth of Groundwater: **28 Feet**

Boring Terminated At: **50 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE ID	BLOWS/ 12 IN	(N1) 60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PASTICITY INDEX (PI)	EXPANSION INDEX (EI)	MAX DRY DENSITY (pcf)	OPT WATER CONTENT (%)	FRICTION ANGLE PHI, (degrees)	COHESION, C (psf)
-24														
-25	25': with fine gravel, PP = 3.0 tsf			CA	29	19	20.4							
-28 ▼	- GW measured at 28 feet at end of day													
-30	30': hard, with fine gravel, some charcoal inclusions, moist, PP = 4 - 4.5 tsf			SPT	25	22								
-35	35': less sand and gravel, very stiff to hard, moist, gray brown with rust mottling and charcoal inclusions, PP = 2.75 - 3.25 tsf			CA	40	24								
-40	40': very stiff, wet, sandy clay, PP = 2.5 - 3 tsf			SPT	14	12								
-44	CLAYEY SAND WITH GRAVEL: yellow brown, medium dense, wet, fine to coarse sand and gravel - ALLUVIUM (Qal)			SPT	21	17								
-50	50': increase in gravels, grades to well-graded gravel with sand, some clay			SPT	21	17								
-51														

GeoSolutions, Inc.

220 High Street, San Luis Obispo, CA 93401
1021 West Tama Lane, Suite 105
Santa Maria, CA 93454

BORING LOG

BORING NO. **S-2**

JOB NO. **SL10070-1**

PROJECT INFORMATION

PROJECT: **Cayucos Sanitary District**
DRILLING LOCATION: **Treatment Plant - Recycled Water Tank**
DATE DRILLED: **March 9/10, 2017**
LOGGED BY:**K. Robinson**

DRILLING INFORMATION

DRILL RIG: **CME 55**
HOLE DIAMETER: **8 Inches**
SAMPLING METHOD: **CA or SPT**
APPROX. ELEVATION: **+78 Feet**

▼ Depth of Groundwater: **23 Feet** Boring Terminated At: **40 Feet** Page 1 of 2

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	(N1) 60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PLASTICITY INDEX (PI)	EXPANSION INDEX (EI)	MAX DRY DENSITY (pcf)	OPT WATER CONTENT (%)	FRICTION ANGLE, PHI (degrees)	COHESION, C (psf)
0	SANDY CLAY: dark brown, slightly moist, fine to medium sand, fat clay, with some gravel, fine to coarse, subangular - ALLUVIUM (Qal)	CL												
-1														
-2														
-3														
-4														
-5	5': stiff to very stiff, PP = 3 - 3.25 tsf			CA	18	25								
-6														
-7														
-8														
-9	10': very stiff, trace fine gravel, PP = 3.25 - 3.75 tsf													
-10				CA	28	28								
-11														
-12														
-13														
-14	SANDY CLAY: yellowish brown, very stiff, slightly moist, with trace fine gravel, PP = 2.5 - 3 tsf - ALLUVIUM (Qal)	CL												
-15				SPT	18	25								
-16														
-17														
-18														
-19														
-20	20': stiff, PP = 1 tsf			CA	14	11								
-21														
-22	- GW estimated at 23 feet													
-23 ▼														

BORING LOG

BORING NO (cont).**S-2**

JOB NO. **SL10070-1**

Depth of Groundwater: **23 Feet** Boring Terminated At: **40 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE ID	BLOWS/ 12 IN	(N1) 60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PASTICITY INDEX (PI)	EXPANSION INDEX (EI)	MAX DRY DENSITY (pcf)	OPT WATER CONTENT (%)	FRICTION ANGLE PHI, (degrees)	COHESION, C (psf)
-24	25': clay to sandy clay, firm, wet, PP = 0.5 - 1 tsf, with thin lenses of poorly-graded sand and gravel with clay, fine to mediumd sand, fine gravel													
-25				SPT	5	5								
-26														
-27														
-28														
-29	CLAYEY GRAVEL: dark yellow brown, medium dense, fine gravel, some coarse, with fine to coarse sand - ALLUVIUM (Qal)	GC												
-30				CA	33	20								
-31														
-32														
-33														
-34														
-35				SPT	37	23								
-36														
-37														
-38														
-39	40': very dense, thin lens of sandy clay													
-40				SPT	50/6"	50/6"								
-41														
-42														
-43														
-44														
-45														
-46														
-47														
-48														
-49														
-50														
-51														

APPENDIX B

Laboratory Testing

Soil Test Reports

LABORATORY TESTING

This appendix includes a discussion of the test procedures and the laboratory test results performed as part of this investigation. The purpose of the laboratory testing is to assess the engineering properties of the soil materials at the Site. The laboratory tests are performed using the currently accepted test methods, when applicable, of the American Society for Testing and Materials (ASTM).

Undisturbed and disturbed bulk samples used in the laboratory tests are obtained from various locations during the course of the field exploration, as discussed in **Appendix A** of this report. Each sample is identified by sample letter and depth. The Unified Soils Classification System is used to classify soils according to their engineering properties. The various laboratory tests performed are described below:

Expansion Index of Soils (ASTM D4829-08) is conducted in accordance with the ASTM test method and the California Building Code Standard, and are performed on representative bulk and undisturbed soil samples. The purpose of this test is to evaluate expansion potential of the site soils due to fluctuations in moisture content. The sample specimens are placed in a consolidometer, surcharged under a 144-psf vertical confining pressure, and then inundated with water. The amount of expansion is recorded over a 24-hour period with a dial indicator. The expansion index is calculated by determining the difference between final and initial height of the specimen divided by the initial height.

Liquid Limit, Plastic Limit, and Plasticity Index of Soils (ASTM D4318-05) are the water contents at certain limiting or critical stages in cohesive soil behavior. The liquid limit (LL or W_L) is the lower limit of viscous flow, the plastic limit (PL or W_P) is the lower limit of the plastic stage of clay and plastic index (PI or I_P) is a range of water content where the soil is plastic. The Atterberg Limits are performed on samples that have been screened to remove any material retained on a No. 40 sieve. The liquid limit is determined by performing trials in which a portion of the sample is spread in a brass cup, divided in two by a grooving tool, and then allowed to flow together from the shocks caused by repeatedly dropping the cup in a standard mechanical device. To determine the Plastic Limit a small portion of plastic soil is alternately pressed together and rolled into a 1/8-inch diameter thread. This process is continued until the water content of the sample is reduced to a point at which the thread crumbles and can no longer be pressed together and re-rolled. The water content of the soil at this point is reported as the plastic limit. The plasticity index is calculated as the difference between the liquid limit and the plastic limit.

Direct Shear Tests of Soils under Consolidated Drained Conditions (ASTM D3080-04) is performed on undisturbed and remolded samples representative of the foundation material. The samples are loaded with a predetermined normal stress and submerged in water until saturation is achieved. The samples are then sheared horizontally at a controlled strain rate allowing partial drainage. The shear stress on the sample is recorded at regular strain intervals. This test determines the resistance to deformation, which is shear strength, inter-particle attraction or cohesion c, and resistance to interparticle slip called the angle of internal friction ϕ.

Particle Size Analysis of Soils (ASTM D422-63R02) is used to determine the particle-size distribution of fine and coarse aggregates. In the test method the sample is separated through a series of sieves of progressively smaller openings for determination of particle size distribution. The total percentage passing each sieve is reported and used to determine the distribution of fine and coarse aggregates in the sample.

One-Dimensional Consolidation Properties of Soils Using Incremental Loading (ASTM D2435-11) is used to determine the magnitude and rate of consolidation of a soil by applying a series of load increments to an undisturbed soil sample and recording sample deformation at selected time intervals. In this test method, a soil specimen is restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. Each stress increment is maintained until excess pore water pressures are completely dissipated. During the consolidation process, measurements are made of the change in the specimen height and this data is used to determine the relationship between the effective stress and void-ratio or strain, and the rate at which consolidation can occur by evaluating the coefficient of consolidation. The data from the consolidation test is used to estimate the magnitude and rate of both differential and total settlement of a structure or earth-fill.

R-Value Testing (ASTM 2844) was performed by Mid-Coast Geotechnical on representative samples of anticipated subgrade soils to assist in the design of pavement sections at the Site.

GeoSolutions, Inc. **SOILS SUMMARY REPORT** **(805) 543-8539**

Project:	Cayucos Sustainable Water Project			Date Tested:	March 22, 2017
Client:	Cayucos Sanitary District			Project #:	SL10070-1
Sample:	C	Depth:	2.0 to 5.0 Feet	Lab #:	16813
Location:	S-1			Sample Date:	March 13, 2017
				Sampled By:	KR

Soil Classification
ASTM D2487, D2488

Result: Dark Brown Sandy CLAY

Specification: CL

Sieve Analysis
ASTM D422

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	98	
No. 8	93	
No. 16	88	
No. 30	83	
No. 50	78	
No. 100	72	
No. 200	65.0	

Sand Equivalent Cal 217

1		SE
2		
3		
4		

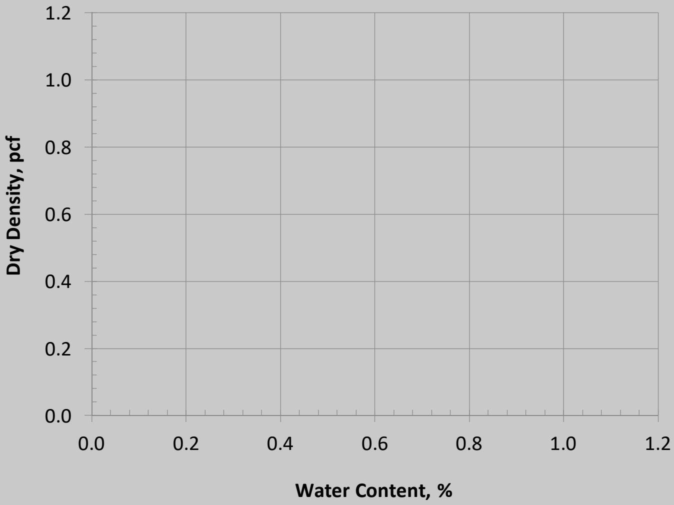
Plasticity Index
ASTM D4318

Liquid Limit:	45
Plastic Limit:	14
Plasticity Index:	**31**

Expansion Index
ASTM D4829

Expansion Index:	**74**
Expansion Potential:	Medium
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve =

Trial #	1	2	3	4
Water Content:				
Dry Density:				

Maximum Dry Density, pcf:	
Optimum Water Content, %:	

Moisture-Density ASTM D2937, **Moisture Content** ASTM D2216

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description
S-1	15.0	10.3	102.4	-	Dark yellowish brown sandy CLAY with gravel (CL)
S-1	20.0	16.5	-	-	Dark yellowish brown sandy CLAY (CL)
S-1	25.0	20.4	102.0		Dark grayish brown sandy CLAY (CL)

Report By: Aaron Eichman

GeoSolutions, Inc. **SOILS SUMMARY REPORT** **(805) 543-8539**

Project:	Cayucos Sustainable Water Project			Date Tested:	March 22, 2017
Client:	Cayucos Sanitary District			Project #:	SL10070-1
Sample:	D	Depth:	11.0 to 15.0 feet	Lab #:	16813
Location:	S-1			Sample Date:	March 13, 2017
				Sampled By:	KR

Soil Classification
ASTM D2487, D2488

Result: Dark Yellowish Brown Sandy CLAY

Specification: CL

Sieve Analysis
ASTM D422

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	98	
No. 8	94	
No. 16	90	
No. 30	86	
No. 50	81	
No. 100	76	
No. 200	70.8	

Sand Equivalent Cal 217

1		SE
2		
3		
4		

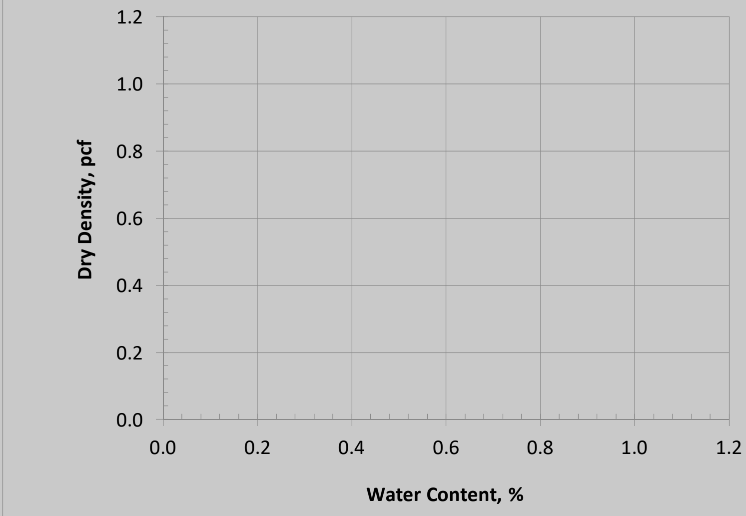
Plasticity Index
ASTM D4318

Liquid Limit:	47
Plastic Limit:	13
Plasticity Index:	**34**

Expansion Index
ASTM D4829

Expansion Index:	**53**
Expansion Potential:	Medium
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve =

Trial #	1	2	3	4
Water Content:				
Dry Density:				

Maximum Dry Density, pcf:	
Optimum Water Content, %:	

Moisture-Density ASTM D2937, **Moisture Content** ASTM D2216

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

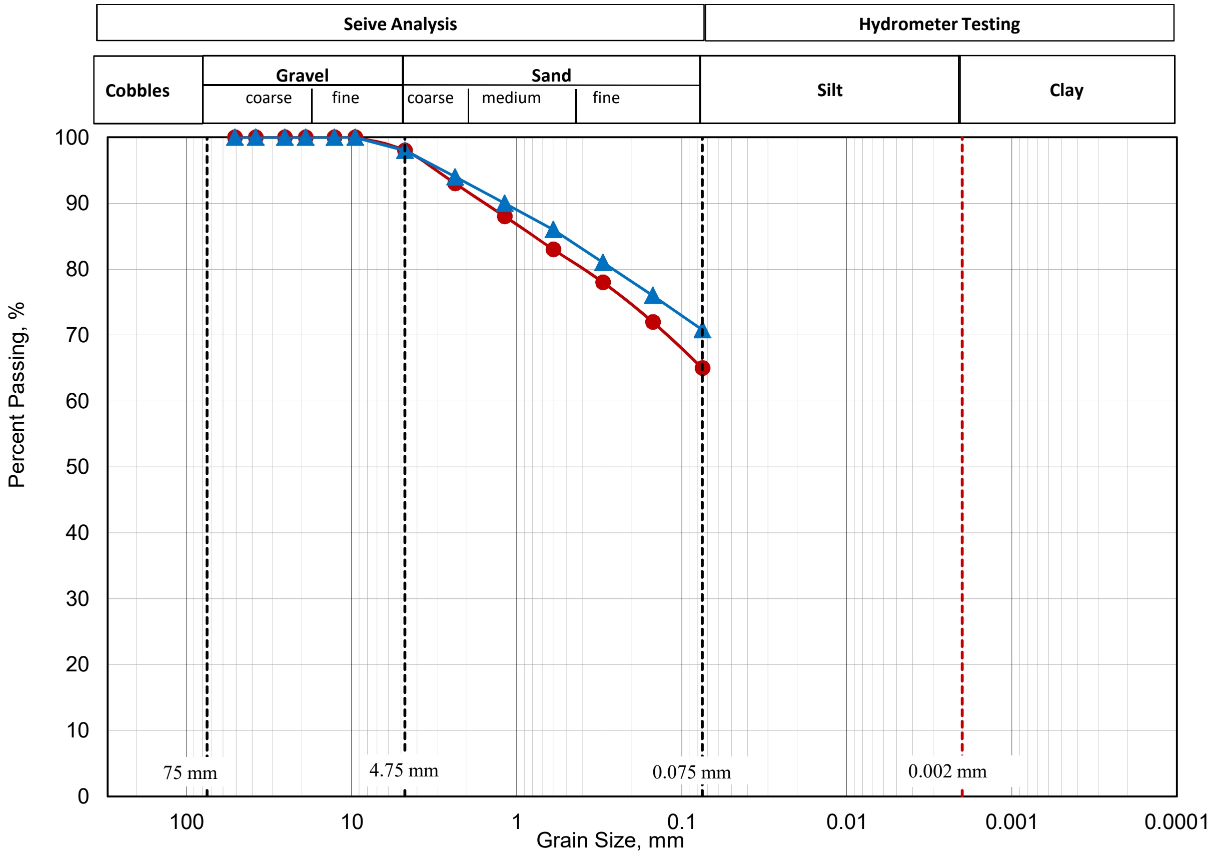
Report By: Aaron Eichman

GeoSolutions, Inc.

PARTICLE SIZE ANALYSIS (ASTM D422)
SUMMARY REPORT

(805) 543-8539

Project: Cayucos Sustainable Water Project
Location: Toro Creek Road, Cayucos, CA
Project #: SL10070-1
Date: 4/2/2017
Checked By: Aeron Eichman



LEGEND symbol	 sample ID	SAMPLE DESCRIPTION	PLASTICITY (ASTM D4318) Liquid Limit (LL)	 Plastic Limit (PL)	 Plasticity Index (PI)
●	C: S-1@2-5'	Dark Brown Sandy CLAY (CL)	45	14	31
▲	D: S-1 @ 11-15'	Dark Yellowish Brown Sandy CLAY (CL)	47	13	34

PARTICLE SIZE ANALYSIS SUMMARY

LEGEND symbol	 sample ID	D_{100}	D_{60}	D_{30}	D_{10}	C_u	C_c	% Gravel	% Sand	% Passing No. 200	% Silt	% Clay
●	C: S-1@2-5'	9.5	-	-	-	-	-	2.0	33.0	65.0	-	-
▲	D: S-1 @ 11-15'	9.5	-	-	-	-	-	2.0	27.2	70.8	-	-

Remarks: Testing was performed in accordance with ASTM D422 and D4318 (where applicable)

NP - non-plastic
NA - not available (could not be calculated from data)

D_{100} - grain size diameter corresponding to 100% passing (mm)
D_{60} - grain size diameter corresponding to 60% passing (mm)
D_{30} - grain size diameter corresponding to 30% passing (mm)
D_{10} - grain size diameter corresponding to 30% passing (mm)

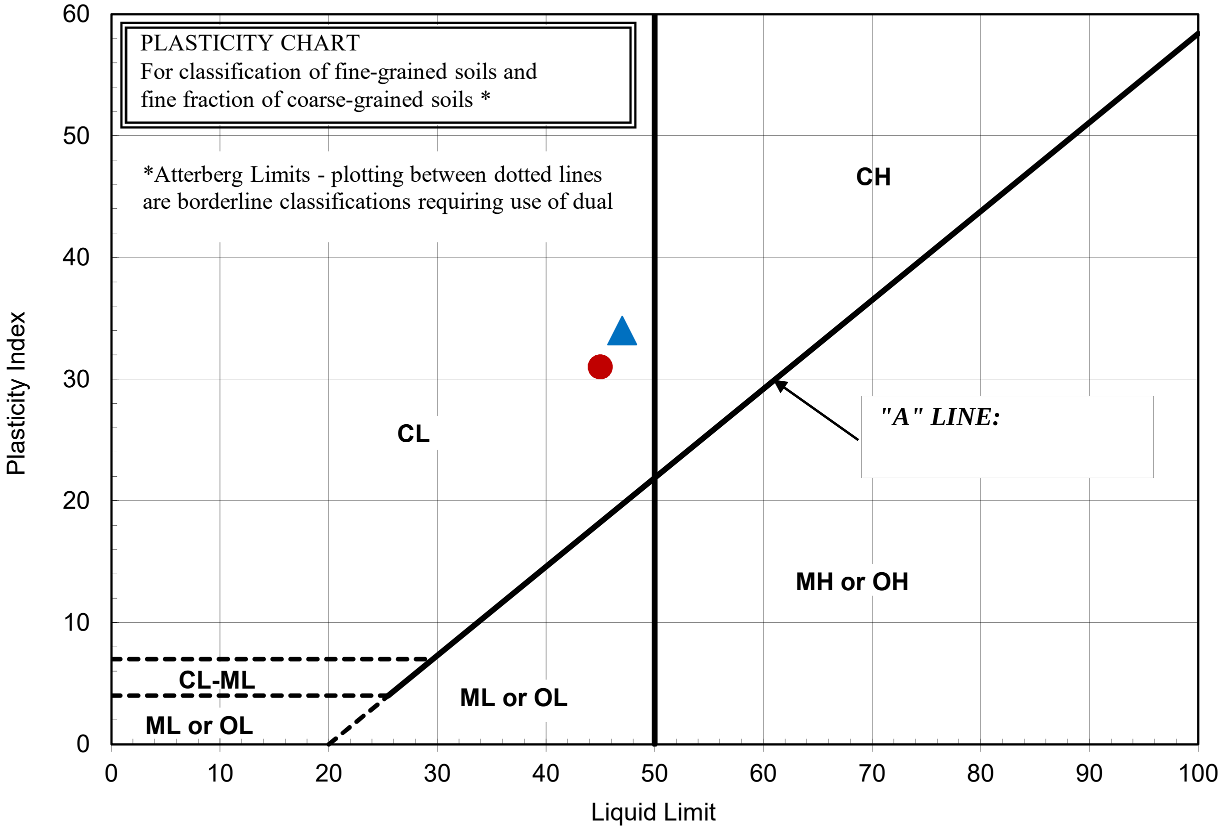
C_c - coefficient of curvature: $C_c = (D_{30})^2 / (D_{60}*D_{10})$
Cu - coefficient of uniformity: $C_u = D_{60} / D_{10}$

GeoSolutions, Inc.

PLASTICITY INDEX TEST SUMMARY REPORT
(ASTM D4318)

(805) 543-8539

Project: Cayucos Sanitary District
Location: B-5
Project #: SL10070-1
Date: 3/23/2017
Checked by: AE



LEGEND		CLASSIFICATION	TEST RESULTS		
symbol	sample ID		Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
●	C: S-1@2-5'	Dark Brown Sandy CLAY (CL)	45	14	31
▲	D: S-1 @ 11-15'	Dark Yellowish Brown Sandy CLAY (CL)	47	13	34

Remarks:

Testing was performed in accordance with ASTM D4318

NP - material tested is nonplastic (liquid or plastic limit tests could not be performed)

Report By: Aaron Eichman

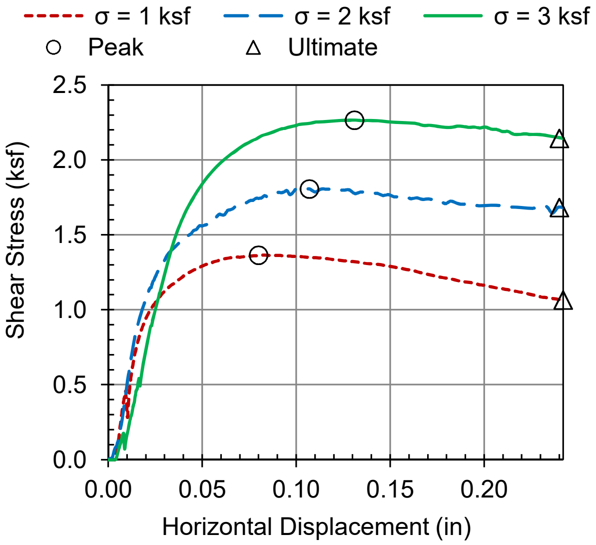
GeoSolutions, Inc.

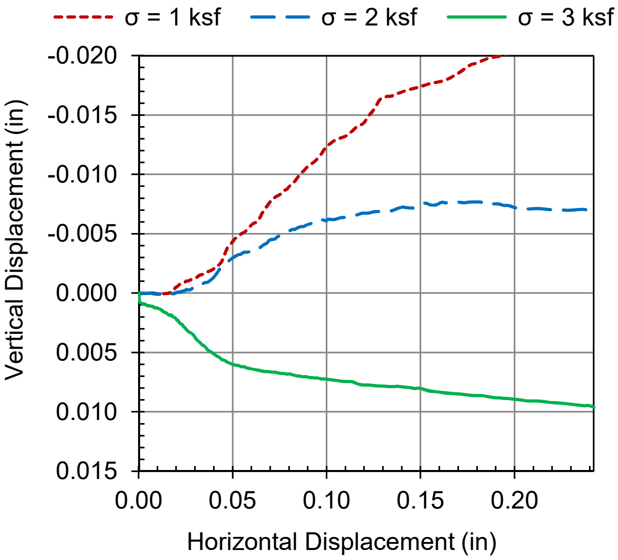
DIRECT SHEAR TEST
SUMMARY REPORT (ASTM D3080)

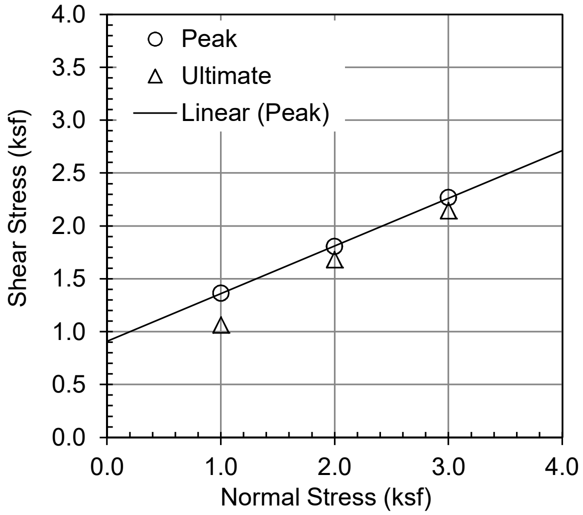
(805) 543-8539

Project: Cayucos Sustainable Water Project — Project No.: SL10070-1
Client: Cayucos Sanitary District — Date Tested: 3/16/2017
Sample No.: S-1 @ 5' Depth: 5.0 Feet — Lab No.: 16813
Location: S-1 — Checked By: AE

MATERIAL DESCRIPTION	LL	PL	PI	% passing No. 200	Gs *	Sample Type
Very Dark Brown Sandy CLAY with Gravel	nm	nm	nm	nm	-	in-situ (rings)







Initial Conditions	Specimen No.		
	1	2	3
Dry Density	106.2	110.5	104.5
Water Content (%)	18.3	18.3	18.3
Diameter (in)	2.42	2.42	2.42
Sample Height (in)	1.00	1.00	1.00

Test Data	Specimen No.		
	1	2	3
Normal Stress (ksf)	1.00	2.00	3.00
Peak Shear Stress (ksf)	1.37	1.81	2.27
Horiz. Displacent at Peak Shear (in)	0.08	0.11	0.13
Ultimate Shear Stress (ksf)	1.07	1.68	2.15
Horiz. Displ. at Ult. Shear (in)	0.24	0.24	0.24
Rate of Deformation (in/min)	0.004	0.004	0.004

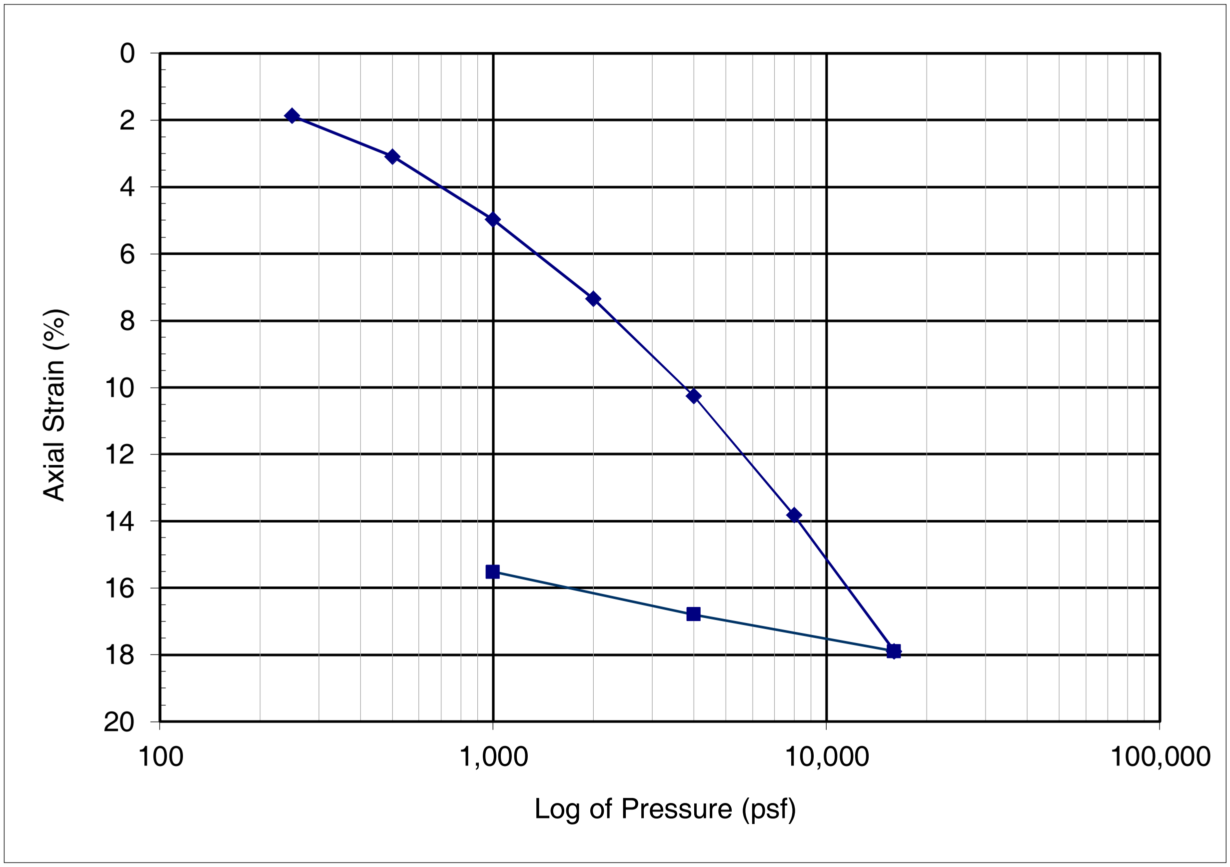
Angle of Internal Friction, ø$_{peak}$ (degrees):	**24.3**
Cohesion, C$_{peak}$ (psf)	**911**

Remarks:

Samples were not saturated prior to shearing

GeoSolutions, Inc.	**CONSOLIDATION REPORT** **ASTM D2435-11**	**(805) 543-8539**

Project:	Cayucos Sustainable Water Project	Date Tested:	3/17/2017
Client:	Cayucos Sanitary District	Project #:	SL10070-1
Sample:	S-2 @ 20' Depth: 20.0 Feet	Lab #:	16813
Location:	S-2	Sample Date:	3/13/2017
Material:	Dark Yellowish Brown Sandy CLAY (CL)	Sampled By:	KR



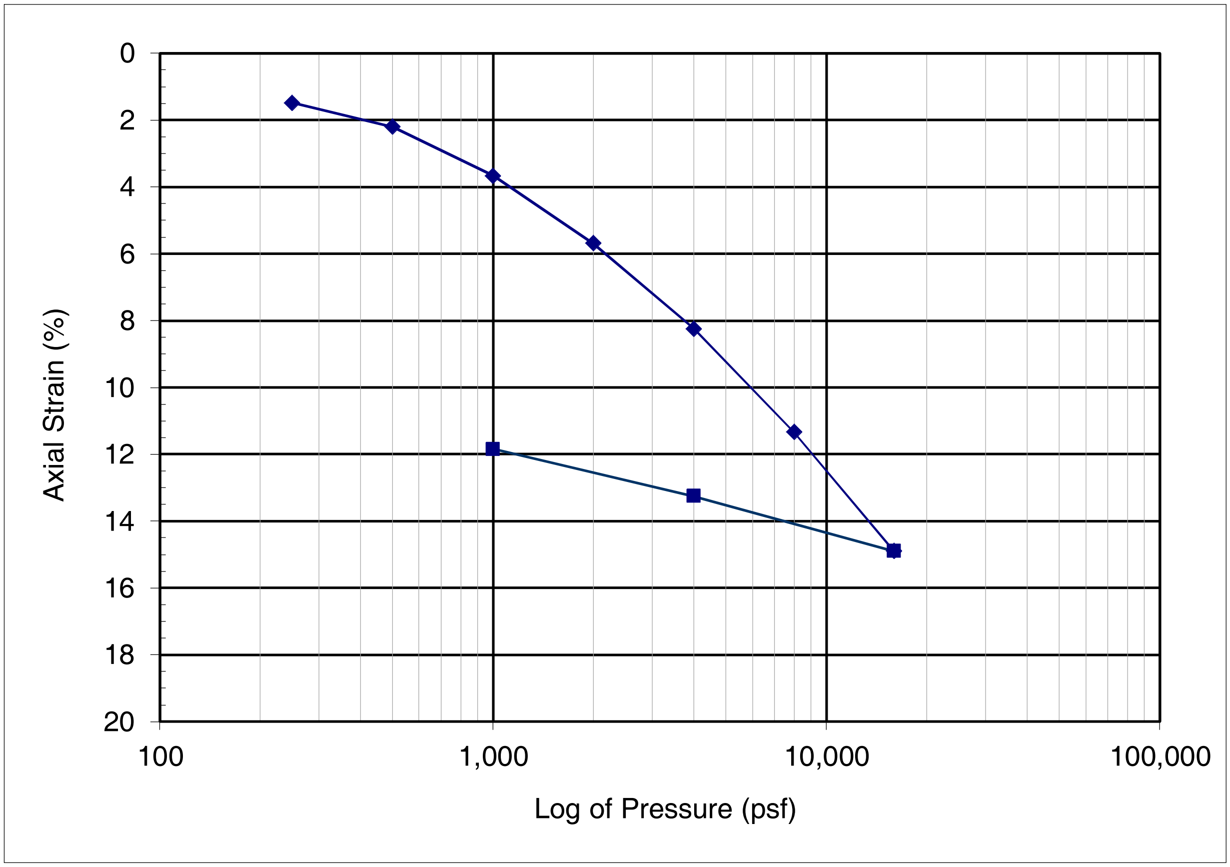
Applied Pressure (psf)	Axial Strain (%)
---	---
250	1.87
500	3.09
1000	4.98
2000	7.34
4000	10.25
8000	13.82
16000	17.89
4000	16.79
1000	15.52

Compression Index, Cc
0.152
Recompression Index, Cr
0.015

Report By: Aaron Eichman

CONSOLIDATION REPORT
ASTM D2435-11

Project:	Cayucos Sustainable Water Project			Date Tested:	3/17/2017
Client:	Cayucos Sanitary District			Project #:	SL10070-1
Sample:	S-2 @ 5'	Depth:	5.0 Feet	Lab #:	16813
Location:	S-2			Sample Date:	3/13/2017
Material:	Dark Brown Sandy CLAY (CL)			Sampled By:	KR



Applied Pressure (psf)	Axial Strain (%)
---	---
250	1.48
500	2.20
1000	3.67
2000	5.69
4000	8.24
8000	11.34
16000	14.90
4000	13.25
1000	11.84

Compression Index, Cc
0.129
Recompression Index, Cr
0.013

Report By: Aaron Eichman

APPENDIX C

Seismic Hazard Analysis

USGS Design Map Summary Report

USGS Design Map Detailed Report

SEISMIC HAZARD ANALYSIS

According to section 1613 of the 2016 CBC (CBSC, 2016), all structures and portions of structures should be designed to resist the effects of seismic loadings caused by earthquake ground motions in accordance with the *ASCE 7 2010 Minimum Design Loads for Buildings and Other Structures*, hereafter referred to as ASCE7-10 (ASCE, 2013). Estimating the design ground motions at the Site depends on many factors including the distance from the Site to known active faults; the expected magnitude and rate of recurrence of seismic events produced on such faults; the source-to-site ground motion attenuation characteristics; and the Site soil profile characteristics. As per section 1613.3.2 of the 2016 CBC, the Site soil profile classification is determined by the average soil properties in the upper 100 feet of the Site profile and can be determined based on the criteria provided in Table 20.3-1 of ASCE7-10.

ASCE7-10 provides recommendations for estimating site-specific ground motion parameters for seismic design considering a Risk-targeted Maximum Considered Earthquake (MCE_R) in order to determine *design spectral response accelerations* and a Maximum Considered Earthquake Geometric Mean (MCE_G) in order to determine probabilistic geometric mean *peak ground accelerations*.

Spectral accelerations from the MCE_R are based on a 5% damped acceleration response spectrum and a 1% exceedance in 50 years (4975-year return period). *Maximum* short period (S_s) and 1-second period (S_1) spectral accelerations are interpolated from the MCE_R-based ground motion parameter maps for bedrock, provided in ASCE7-10. These spectral accelerations are then multiplied by site-specific coefficients (F_a, F_v), based on the Site soil profile classification and the maximum spectral accelerations determined for bedrock, to yield the *maximum* short period (S_{MS}) and 1-second period (S_{M1}) spectral response accelerations at the Site. According to section 11.2 of ASCE7-10 and section 1613 of the 2016 CBC, buildings and structures should be specifically proportioned to resist *design* earthquake ground motions. Section 1613.3.4 of the 2016 CBC indicates the site-specific *design* spectral response accelerations for short (S_{DS}) and 1-second (S_{D1}) periods can be taken as two-thirds of *maximum* ($S_{DS} = 2/3{*}S_{MS}$ and $S_{D1} = 2/3{*}S_{M1}$).

Per ASCE7-10, Section 21.5, the probabilistic maximum mean peak ground acceleration (PGA) corresponding to the MCE_G can be computed assuming a 2% probability of exceedance in 50 years (2475-year return period) and is initially determined from mapped ground accelerations for bedrock conditions. The site-specific peak ground acceleration (PGA_M) is then determined by multiplying the PGA by the site-specific coefficient F_h (where F_h is a function of Site Class and PGA).

Spectral response accelerations, peak ground accelerations, and site coefficients provided in this report were obtained using the web-based U.S. Seismic Design Map tool available from the United States Geological Survey website (USGS, 2013). This program utilizes the methods developed in the 1997, 2000, 2003, 2008 and 2013 errata editions of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures in conjunction with user-inputted Site latitude and longitude coordinates to calculate seismic design parameters and response spectra (both for period and displacement) for soil profile Site Classifications A through E. Output from the web-based program are included in this Appendix.

USGS Design Maps Summary Report

User–Specified Input

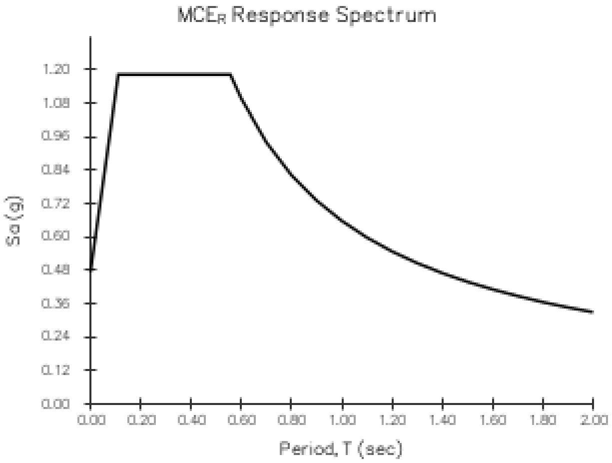
Report Title	Cayucos Sustainable Water Project Mon April 17, 2017 22:54:17 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	35.42°N, 120.8632°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

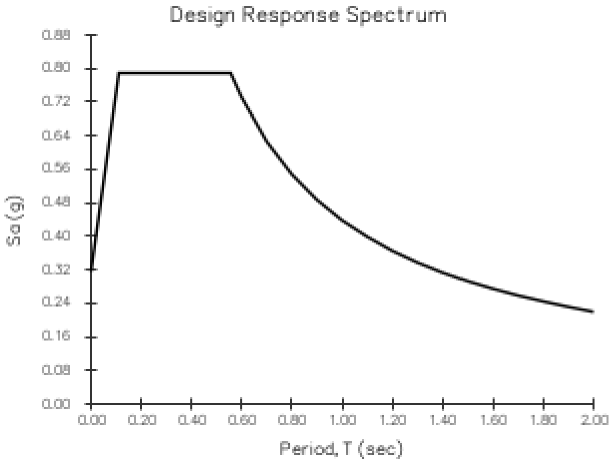
USGS–Provided Output

S_S = 1.128 g **S_{MS} =** 1.183 g **S_{DS} =** 0.789 g

S_1 = 0.415 g **S_{M1} =** 0.658 g **S_{D1} =** 0.438 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA$_M$, T$_L$, C$_{RS}$, and C$_{R1}$ values, please view the detailed report.

Design Maps Detailed Report

ASCE 7-10 Standard (35.42°N, 120.8632°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 1.128 g
From Figure 22-2 [2]	S_1 = 0.415 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.128 g, F_a = 1.049

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.415 g, F_v = 1.585

Equation (11.4–1): $S_{MS} = F_aS_S = 1.049 \times 1.128 = 1.183$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.585 \times 0.415 = 0.658$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

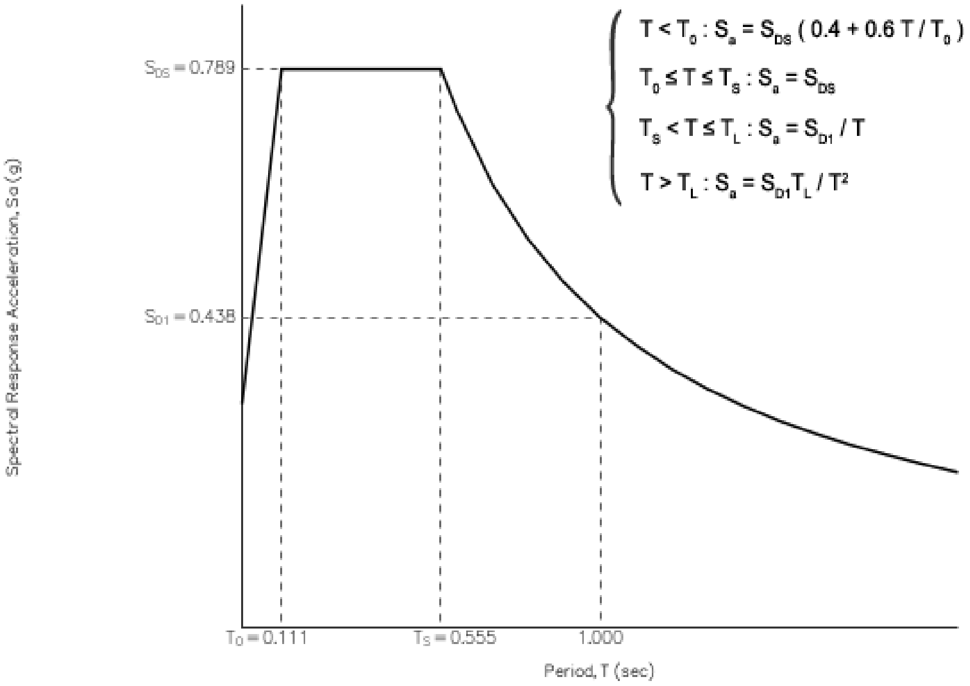
Equation (11.4–3): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.183 = 0.789$ g

Equation (11.4–4): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.658 = 0.438$ g

Section 11.4.5 — Design Response Spectrum

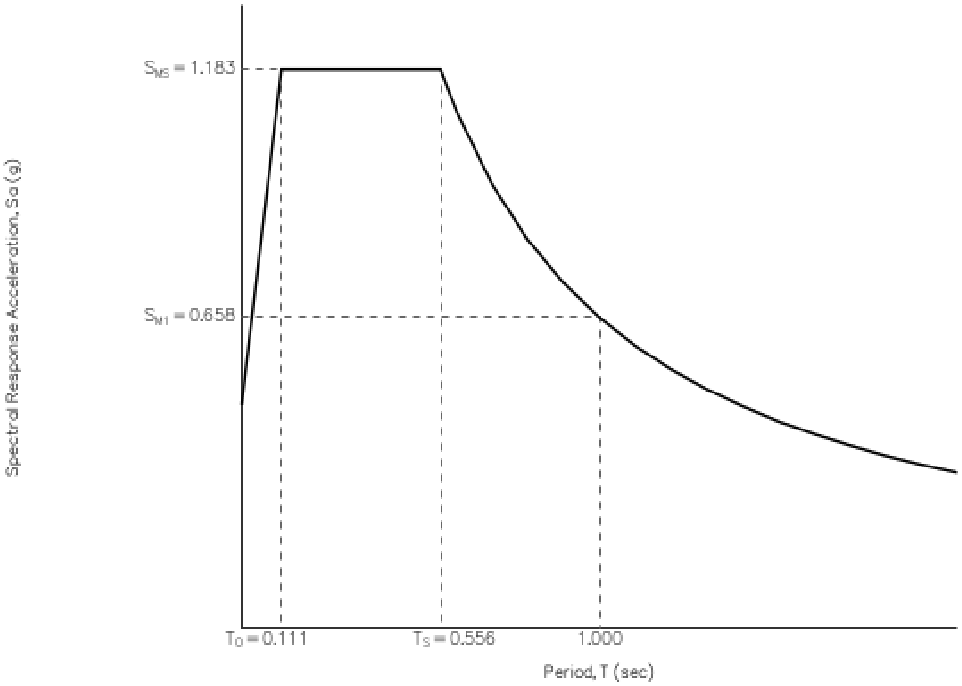
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4]	PGA = 0.440
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.060 \times 0.440 = 0.466$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.440 g, F_{PGA} = 1.060

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5]	C_{RS} = 0.966
From Figure 22-18 [6]	C_{R1} = 0.982

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	**RISK CATEGORY**		
	I or II	**III**	**IV**
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.789 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	**RISK CATEGORY**		
	I or II	**III**	**IV**
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.438 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX D

Preliminary Grading Specifications

PRELIMINARY GRADING SPECIFICATIONS

A. General

1. These preliminary specifications have been prepared for the subject site; GeoSolutions, Inc. should be consulted prior to the commencement of site work associated with site development to ensure compliance with these specifications.

2. GeoSolutions, Inc. should be notified at least 72 hours prior to site clearing or grading operations on the property in order to observe the stripping of surface materials and to coordinate the work with the grading contractor in the field.

3. These grading specifications may be modified and/or superseded by recommendations contained in the text of this report and/or subsequent reports.

4. If disputes arise out of the interpretation of these grading specifications, the Soils Engineer shall provide the governing interpretation.

B. Obligation of Parties

1. The Soils Engineer should provide observation and testing services and should make evaluations to advise the client on geotechnical matters. The Soils Engineer should report the findings and recommendations to the client or the authorized representative.

2. The client should be chiefly responsible for all aspects of the project. The client or authorized representative has the responsibility of reviewing the findings and recommendations of the Soils Engineer. During grading the client or the authorized representative should remain on-site or should remain reasonably accessible to all concerned parties in order to make decisions necessary to maintain the flow of the project.

3. The contractor is responsible for the safety of the project and satisfactory completion of all grading and other operations on construction projects, including, but not limited to, earthwork in accordance with project plans, specifications, and controlling agency requirements.

C. Site Preparation

1. The client, prior to any site preparation or grading, should arrange and attend a meeting which includes the grading contractor, the design Structural Engineer, the Soils Engineer, representatives of the local building department, as well as any other concerned parties. All parties should be given at least 72 hours notice.

2. All surface and sub-surface deleterious materials should be removed from the proposed building and pavement areas and disposed of off-site or as approved by the Soils Engineer. This includes, but is not limited to, any debris, organic materials, construction spoils, buried utility line, septic systems, building materials, and any other surface and subsurface structures within the proposed building areas. Trees designated for removal on the construction plans should be removed and their primary root systems grubbed under the observations of a representative of GeoSolutions, Inc. Voids left from site clearing should be cleaned and backfilled as recommended for structural fill.

3. Once the Site has been cleared, the exposed ground surface should be stripped to remove surface vegetation and organic soil. A representative of GeoSolutions, Inc. should determine the required depth of stripping at the time of work being completed. Strippings may either be disposed of off-site or stockpiled for future use in landscape areas, if approved by the landscape architect.

D. Site Protection

1. Protection of the Site during the period of grading and construction should be the responsibility of the contractor.

2. The contractor should be responsible for the stability of all temporary excavations.

3. During periods of rainfall, plastic sheeting should be kept reasonably accessible to prevent unprotected slopes from becoming saturated. Where necessary during periods of rainfall, the contractor should install check-dams, de-silting basins, sand bags, or other devices or methods necessary to control erosion and provide safe conditions.

E. Excavations

1. Materials that are unsuitable should be excavated under the observation and recommendations of the Soils Engineer. Unsuitable materials include, but may not be limited to: 1) dry, loose, soft, wet, organic, or compressible natural soils; 2) fractured, weathered, or soft bedrock; 3) non-engineered fill; 4) other deleterious materials; and 5) materials identified by the Soils Engineer or Engineering Geologist.

2. Unless otherwise recommended by the Soils Engineer and approved by the local building official, permanent cut slopes should not be steeper than 2:1 (horizontal to vertical). Final slope configurations should conform to section 1804 of the 2016 California Building Code unless specifically modified by the Soil Engineer/Engineering Geologist.

3. The Soil Engineer/Engineer Geologist should review cut slopes during excavations. The contractor should notify the Soils Engineer/Engineer Geologist prior to beginning slope excavations.

F. Structural Fill

1. Structural fill should not contain rocks larger than 3 inches in greatest dimension, and should have no more than 15 percent larger than 2.5 inches in greatest dimension.

2. Imported fill should be free of organic and other deleterious material and should have very low expansion potential, with a plasticity index of 12 or less. Before delivery to the Site, a sample of the proposed import should be tested in our laboratory to determine its suitability for use as structural fill.

G. Compacted Fill

1. Structural fill using approved import or native should be placed in horizontal layers, each approximately 8 inches in thickness before compaction. On-site inorganic soil or approved imported fill should be conditioned with water to produce a soil water content near optimum moisture and compacted to a minimum relative density of 90 percent based on ASTM D1557-12_{e1}.

2. Fill slopes should not be constructed at gradients greater than 2-to-1 (horizontal to vertical). The contractor should notify the Soils Engineer/Engineer Geologist prior to beginning slope excavations.

3. If fill areas are constructed on slopes greater than 10-to-1 (horizontal to vertical), we recommend that benches be cut every 4 feet as fill is placed. Each bench shall be a minimum of 10 feet wide with a minimum of 2 percent gradient into the slope.

4. If fill areas are constructed on slopes greater than 5-to-1, we recommend that the toe of all areas to receive fill be keyed a minimum of 24 inches into underlying dense material. Key depths are to be observed and approved by a representative of GeoSolutions, Inc. Sub-drains shall be placed in the keyway and benches as required.

H. Drainage

1. During grading, a representative of GeoSolutions, Inc. should evaluate the need for a sub-drain or back-drain system. Areas of observed seepage should be provided with sub-surface drains to release the hydrostatic pressures. Sub-surface drainage facilities may include gravel blankets, rock filled trenches or Multi-Flow systems or equal. The drain system should discharge in a non-erosive manner into an approved drainage area.

2. All final grades should be provided with a positive drainage gradient away from foundations. Final grades should provide for rapid removal of surface water runoff. Ponding of water should not be allowed on building pads or adjacent to foundations. Final grading should be the responsibility of the contractor, general Civil Engineer, or architect.

3. Concentrated surface water runoff within or immediately adjacent to the Site should be conveyed in pipes or in lined channels to discharge areas that are relatively level or that are adequately protected against erosion.

4. Water from roof downspouts should be conveyed in solid pipes that discharge in controlled drainage localities. Surface drainage gradients should be planned to prevent ponding and promote drainage of surface water away from building foundations, edges of pavements and sidewalks. For soil areas we recommend that a minimum of 2 percent gradient be maintained.

5. Attention should be paid by the contractor to erosion protection of soil surfaces adjacent to the edges of roads, curbs and sidewalks, and in other areas where hard edges of structures may cause concentrated flow of surface water runoff. Erosion resistant matting such as Miramat, or other similar products, may be considered for lining drainage channels.

6. Sub-drains should be placed in established drainage courses and potential seepage areas. The location of sub-drains should be determined after a review of the grading plan. The sub-drain outlets should extend into suitable facilities or connect to the proposed storm drain system or existing drainage control facilities. The outlet pipe should consist of a non-perforated pipe the same diameter as the perforated pipe.

I. Maintenance

1. Maintenance of slopes is important to their long-term performance. Precautions that can be taken include planting with appropriate drought-resistant vegetation as recommended by a landscape architect, and not over-irrigating, a primary source of surficial failures.

2. Property owners should be made aware that over-watering of slopes is detrimental to long term stability of slopes.

J. Underground Facilities Construction

1. The attention of contractors, particularly the underground contractors, should be drawn to the State of California Construction Safety Orders for “Excavations, Trenches, Earthwork.” Trenches or excavations greater than 5 feet in depth should be shored or sloped back in accordance with OSHA Regulations prior to entry.

2. Bedding is defined as material placed in a trench up to 1 foot above a utility pipe and backfill is all material placed in the trench above the bedding. Unless concrete bedding is required around utility pipes, free-draining sand should be used as bedding. Sand to be used as bedding should be tested in our laboratory to verify its suitability and to measure its compaction characteristics. Sand bedding should be compacted by mechanical means to achieve at least 90 percent relative density based on ASTM D1557-12$_{e1}$.

3. On-site inorganic soils, or approved import, may be used as utility trench backfill. Proper compaction of trench backfill will be necessary under and adjacent to structural fill, building foundations, concrete slabs, and vehicle pavements. In these areas, backfill should be conditioned with water (or allowed to dry), to produce a soil water content of about 2 to 3 percent above the optimum value and placed in horizontal layers, each not exceeding 8 inches in thickness before compaction. Each layer should be compacted to at least 90 percent relative density based on ASTM D1557-12$_{e1}$. The top lift of trench backfill under vehicle pavements should be compacted to the requirements given in report under Preparation of Paved Areas for vehicle pavement sub-grades. Trench walls must be kept moist prior to and during backfill placement.

K. Completion of Work

1. After the completion of work, a report should be prepared by the Soils Engineer retained to provide such services. The report should including locations and elevations of field density tests, summaries of field and laboratory tests, other substantiating data, and comments on any changes made during grading and their effect on the recommendations made in the approved Soils Engineering Report.

2. Soils Engineers shall submit a statement that, to the best of their knowledge, the work within their area of responsibilities is in accordance with the approved soils engineering report and applicable provisions within Chapter 18 of the 2016 CBC.