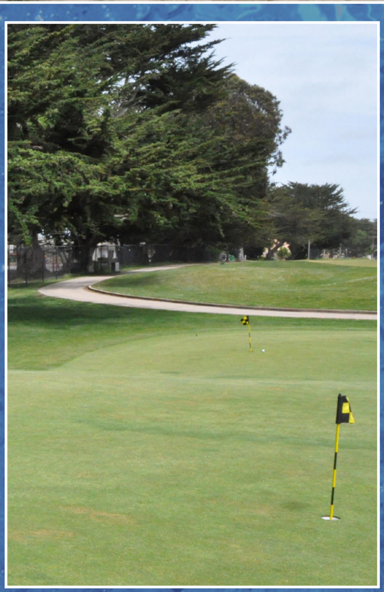
Pacific Grove
Local Water Project



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Brezack & Associates Planning

APPENDIX G

Preliminary Geological Review

GEOTECHNICAL INVESTIGATION
PACIFC GROVE ASBS STORMWATER
MANAGEMENT PROJECT
CITIES OF MONTEREY
AND PACIFIC GROVE, CALIFORNIA

PROJECT 2013.0031

For

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TABLE OF CONTENTS

FIGURES

APPENDIX A

GEOTECHNICAL INVESTIGATION
PACIFC GROVE ASBS STORMWATER MANAGEMENT PROJECT
CITIES OF MONTEREY AND PACIFIC GROVE, CALIFORNIA

1. INTRODUCTION

1.1 GENERAL

This report presents the results of our geotechnical investigation for proposed improvements associated with the Pacific Grove ASBS Stormwater Management Project, in Pacific Grove, California. The location of the project is shown on our Geologic Index Map (Figure 1), and features in the project vicinity are shown on the Treatment Plant Site Plan (Figure 5) of this report. For purposes of this report "site" refers to the Pacific Grove Water Treatment Plant (PGWTP), the Crespi Pond area and the area of the proposed wetland.

1.2 PROJECT DESCRIPTION

As outlined in the Draft Engineering Report prepared by Fall Creek Engineering, the Pacific Grove ASBS stormwater management project is designed to improve stormwater quality prior to discharge to the "Area of Special Biological Significance" (ASBS) designated along the Pacific Grove coastline. The goal of the project is to achieve a 90 percent reduction in the pollutants that are discharged into the bay by seasonal stormwater discharges. The portions of the project that are addressed by this geotechnical investigation consist of the following:

- Reuse or reconstruction of the existing two tanks at the abandoned Pacific Grove Water Treatment Plant (PGWTP): the two existing tanks at this site are being evaluated for their structural integrity and are planned for reuse for stormwater storage/treatment.

- Expansion of Crespi Pond: the existing pond will be expanded by deepening it up to 5 feet (it's present maximum depth is about 5 feet and proposed maximum depth is about 10 feet), and by extending it approximately 120 feet south and 40 feet west of its existing limits.

- Creation of wetlands: a wetland is proposed for construction about 750 south of Crespi Pond. This wetland will extend about 5 feet below existing grade.

- Installation of utility lines: stormwater pipes will be installed to connect the new wetland, Crespi Pond and the PGWTP site. Utility trenches proposed in other areas of the site are beyond our present scope of work.

Originally, our scope also included expansion of a cistern near the intersection of Del Monte Boulevard and Egan Avenue. We understand this portion of the project is no longer being pursued.

1.3 OBJECTIVE AND SCOPE OF INVESTIGATION

The objective of this investigation was to explore subsurface conditions in the above mentioned project areas; provide geotechnical information to assist in the preliminary design and evaluation of the proposed improvements; and to prepare geotechnical recommendations for their design and construction.

The following services were performed for our investigation.

1. Reconnaissance of the project areas to observe surface conditions and mark locations for our subsurface exploration.

2. Coordination of our drilling with Underground Service Alert and the City of Pacific Grove.

3. Research of existing regional geologic information including geologic hazards pertinent to the site.

4. Exploration, sampling, and classification of subsurface soils at selected locations by means of six exploratory drill holes.

5. Laboratory testing of selected soil samples recovered from our drill holes.

6. Engineering analysis of the above field and laboratory data and formulation of conclusions and recommendations for the project.

7. Preparation of this report summarizing our findings, conclusions and recommendations.

1.4 INFORMATION PROVIDED

For this investigation, the following was provided to us and used during our study.

- A 6-page set of plans titled "15% Design Plans, Pacific Grove ASBS," prepared by Fall Creek Engineering and dated June 2013.
- A report titled, "Draft Engineering Report: Preliminary Hydrology Analysis and David Avenue Reservoir Design Alternatives Summary, Pacific Grove Area of Special Biological Significance (ASBS) Stormwater Management Project", prepared by Fall Creek Engineering and dated June 20, 2013.
- Design plans for the existing PGWTP tanks consisting of Sheets 2, 7, 9, 10 and 14 of a 23-page set of plans titled "City of Pacific Grove Sewage Pumping and Treatment Works", prepared by Alfred D. Coons, City Manager and Engineer, dated January 1952.
- Base maps for our Figure 4 and 5 provided by Fall Creek Engineering via email and dated 8/5/13.

2. SITE INVESTIGATION

2.1 SUBSURFACE EXPLORATION

The subsurface exploration program included six drill holes (DH-1 through DH-6). The drill holes were located in the field by referencing to existing site features and pacing; therefore, locations should be considered approximate. The approximate locations of the drill holes are shown on Figure 4, with a more detailed map of the PGWTP tank area in Figure 5.

Drill holes DH-1 through DH-6 were advanced on June 4, 2013, to depths between 9 and 24 feet below the ground surface using a Mobile B-53 drilling rig equipped with an 8-inch diameter hollow stem auger.

In the field, our personnel visually classified the materials encountered in the drill holes and maintained a log of each drill hole. Samples were obtained from the drill holes by driving a 2½-inch inside diameter split spoon or a 2-inch outside diameter (1⅜ inch inside diameter) Standard Penetration Test (SPT) sampler up to a depth of 18 inches into the earth material using a 140-pound hammer falling 30 inches. The number of blows required to drive the samplers was recorded for each 6-inch penetration interval. The number of blows required to drive the sampler the last 12 inches, or the penetration interval indicated on the log where harder material was encountered, was shown as blows per foot on the drill hole logs. The hammer was operated by a hydraulic winch and pulley system.

Soil samples were collected from the drill holes at approximately 5-foot vertical intervals. Soil samples were sealed in the field and transported to our laboratory for further evaluation and testing. Visual classification of soils encountered in our drill holes was made in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). The laboratory test results were used to refine our field classifications. Two Keys to Soil Classification, one for fine grained soils and one for coarse grained soils, and one key for Rock Classification are included in Appendix A together with the logs of the drill holes.

2.2 LABORATORY TESTING

Laboratory tests were performed on selected soil samples. These tests included water content, dry density and percent passing a No. 200 sieve. The laboratory test results are presented on the drill hole logs at the corresponding sample depths.

3. FINDINGS

3.1 GEOLOGIC SETTING

Regional geologic mapping by Clark, Dupre and Rosenberg (1997; Clark and others hereafter for brevity) provides the best available regional-scale geologic mapping for the project area. Our Geologic Index Map (Figure 1) is an excerpt from Clark and others (1997).

The geology of the site vicinity, broadly speaking, is that of an elevated marine terrace cut across granitic bedrock, overlain with a thin mantle of terrace lag deposits and local dune sands.

As mapped by Clark and others (1997), bedrock in the site vicinity is mapped as "porphyritic granodiorite of Monterey of Ross" (map unit Kgdp), which can be thought of as granitic rock.

As relative sea level dropped, a marine (coastal) terrace was planed across the granitic bedrock, with Pleistocene-age (within the last 2 million years) marine terrace deposits deposited across this surface. These deposits ("Peninsula College coastal terrace deposits;" [map unit Qctp] are preserved on high ground about 1500 feet south of the site (see Fig. 1).

As relative sea level dropped further, another marine terrace was planed across the bedrock, leaving the "Ocean View coastal terrace deposits" (map unit Qcto) mapped as fringing the coastline near the site, and underlying the site. Texturally, these deposits are described as consisting of "semi-consolidated, moderately well-sorted marine sand containing thin, discontinuous gravel-rich layers."

Dune sand deposits of Pleistocene age (map unit Qod1) and Holocene age (map unit Qd) have been deposited by the wind in the areas shown on Figure 1 – generally south of the site. The younger (Holocene, map unit Qd) dune sand deposits are described as "unconsolidated, well-sorted, fine- to medium-grained sand." The older (Pleistocene, map unit Qod1) dune sand deposits are described as texturally the same as the younger deposits, and "weakly consolidated."

Approximately 300 to 500 feet east of the PGWTP tanks, Crespi Pond (see Figure 1 and Figure 4) occupies a topographic swale that is mapped as infilled with younger (Holocene age) alluvium. This map unit (Qal) is described as consisting of "unconsolidated, heterogeneous, moderately sorted silt and sand with discontinuous lenses of clay and silty clay". The topographic swale occupied by Crespi Pond likely marks the location of a now-buried stream course that is incised into the top-of-bedrock surface.

Rosenberg (2001) compiled previous and independent geologic mapping for Monterey County that incorporated the geologic mapping of Clark and others (1997), of which he was a co-author. The linework of this compilation is more generalized than that of Clark and others, due to map scale. No significant differences in geologic mapping as it affects the site vicinity are reflected in Rosenberg (2001).

Wagner and others (2002) prepared a regional geologic compilation map that encompasses the site, also at a more generalized scale than that of Clark and others (1997). Wagner and others (2002) drew on both Clark and others (1997) and Rosenberg (2001), and no significant differences in geologic interpretation are reflected in their mapping.

3.2 GEOHAZARDS MAPPING

Rosenberg (2001) prepared a County-wide map of liquefaction susceptibility, as a derivative map associated with the geologic mapping described above. An excerpt of Rosenberg's Liquefaction Susceptibility Map is presented as our Liquefaction Map (Figure 2). Rosenberg's classification ranged across four liquefaction susceptibility classes (Low, Moderate, and High, with a fourth "variable" class used in areas of significant grading). While this map is necessarily generalized, it maps the older marine terrace deposits (map units Qctp and Qcto underlying the PGWTP tank site and the proposed wetland area) and older dune deposits (map unit Qod1), as having a low liquefaction susceptibility. The younger dune deposits (map unit Qd) mapped south of the PGWTP tank site, and the alluvium filling the topographic swale (underlying Crespi Pond), were considered to have a high liquefaction susceptibility. Bedrock and upland areas are mapped as having a "low" liquefaction susceptibility.

There are no known historic liquefaction sites from the 1906 or 1989 earthquakes in the PGWTP site or vicinity.

The PGWTP site is mapped as lying within a State of California "tsunami inundation area" which fringes the coastline as shown in Figure 3 of this report (State of California Emergency Management Agency, 2009). Both the PGWTP site and Crespi pond are within the designated tsunami inundation area. The shoreward limit of inundation is shown as lying at approximately the southern end of the PGWTP site and Crespi Pond, and runs approximately along contour eastward and westward around the end of Point Pinos.

3.3 EARTHQUAKE FAULTING

No active faults are mapped in the project vicinity (Wagner and others, 2002). Faults associated with the Monterey Bay fault zone are mapped east of the site, and the San Gregorio fault west of the site (see Table 1 of seismic sources below).

The greater San Francisco/Monterey Bay Area is seismically dominated by the active San Andreas Fault system, the tectonic boundary between the northward moving Pacific Plate (west of the fault) and the North American Plate (east of the fault). This movement is distributed across a complex system of generally strike-slip, right-lateral and subparallel faults

Regional faults that have a potential to generate large magnitude earthquakes and significant ground shaking at the site are listed in Table 1. Map distances are derived from the USGS Quaternary Fault and Fold database (http://earthquake.usgs.gov/regional/qfaults/), based on a latitude of 36.636513 and a longtitude of -121.934629.

Table 1. Significant Seismic Sources within Project Vicinity

Fault	Approximate Distance	Direction from Project Site to Fault
Monterey Bay/Tularcitos	2.7 km	Northeast
San Gregorio	10.1 km	West
Reliz	11.7 km	Northeast
Zayante-Vergeles	35.6 km	Northeast
San Andreas	41.5 km	Northeast

3.4 SEISMICITY

The Working Group on California Earthquake Probabilities (WGCEP) estimates the probabilities of major earthquakes are now in their fourth iteration. The greatest changes in approach from the first to the fourth iteration are; 1) the treatment of major faults as either segmented, unsegmented or capable of different rupture scenarios; 2) the progressive consideration of more potential seismic sources, and 3) the use of time-independent versus time-dependent models. Current estimates (WGCEP, 2003, 2008) are most detailed for the greater San Francisco Bay Area; WGCEP (2008) estimated a 63% probability of a large (magnitude 6.7 or greater) earthquake in the San Francisco Bay area as a whole over a 30-year period; this overall probability differed only slightly from the previous (WGCEP, 2003) probability of 62%. The current estimate for the Calaveras fault alone is 7% (revised down from the 11% presented by WGCEP, 2003); for the (northern) San Andreas fault alone, 21%; and for the Hayward fault, 31% (revised upward from the WGCEP (2003) value of 27%).

3.5 SITE COEFFICIENTS AND SEISMIC GROUND MOTION VALUES

The site coefficients and seismic ground motion values in Table 2 were developed using the USGS Seismic Design Maps (http://earthquake.usgs.gov/hazards/designmaps/usdesign.php), incorporating both the ASCE 7-05 and ASCE 7-10 codes, and the project site location (latitude 36.636513N, longitude -121.934629W).

Table 2. Seismic Design Parameters

Parameter	ASCE 7-05 Values	ASCE 7-10 Values
Site Class	B	B
Site Coefficient F_a	1.0	1.0
Site Coefficient F_v	1.0	1.0
S_s	1.473	1.549g
S_1	0.614	0.569g
S_{Ms}	1.473	1.549g
S_{M1}	0.614	0.569g
S_{Ds}	0.982	1.033g
S_{D1}	0.409	0.379g

The design peak ground acceleration (PGA) can be taken as the lesser of the values developed from probabilistic approach and deterministic approach. Using the USGS Seismic Design Maps and incorporating both ASCE 7-05 and ASCE 7-10, the PGA value at the site is 0.62g for the Maximum Considered Earthquake (MCE) and 0.41g for the Design Earthquake (DE). MCE corresponds to a 2% probability of exceedance in a 50-year period. Studies have shown that, for the San Francisco Bay Area, DE roughly corresponds to a 10% probability of exceedance in a 50-year period.

3.6 SURFACE CONDITIONS

The PGWTP and Crespi Pond site is located on Point Pinos, at the northern tip of the City of Pacific Grove. The area includes the Pacific Grove golf Links and is bordered by Sunset Drive and the Pacific Ocean on the north and west, by Asilomar Avenue to the east and by Lighthouse Avenue to the south.

As mentioned above, the proposed development within the area of DH-6 (near the intersection of Del Monte Boulevard and Egan Avenue) was abandoned and is not addressed in this section.

3.6.1 PGWTP Tank Site

The existing ground surface at the tank site slopes gently towards the north at a gradient of about 10:1 (horizontal:vertical), steepening south of the site, and flattening to the north. Past grading for the tank pad appears to have consisted of cuts of 3 to 5 feet in height on the south and west sides of the tanks. There was no evidence of significant fill in the immediate vicinity of the tanks, although the 1952 project plans indicate general fill placement on the east side of the site.

We understand that the existing tanks are also being evaluated for reuse by a structural engineer, Harper and Associates. Based on the information provided in the 1952 design plans, supplemented by information provided to us from Harper and Associates, both tanks are about 57 feet in diameter, 30 to 33 feet in height and are buried 10 to 16 feet below grade. The "clarifier" on the east side, has a sloping base that extends between 13 to 16.5 feet below grade and the "digester" on the west side, has a flat base that extends about 10 feet below existing grade.

3.6.2 Crespi Pond and Proposed Wetland Site

The dune sands that comprise the majority of the golf course form a subtle, roughly north-south trending ridge which forms the eastern border of the topographic swale that terminates at Crespi Pond. The ground surface on the margins of the pond is level to very slightly sloping towards the north. We understand that the existing depth of the pond is about 5 feet, and the entire pond is bordered by the golf course fairway.

The area about 750 feet south of Crespi Pond that is proposed for a new wetland is also located in the topographic swale that borders the dune sand ridge. This area occupies a topographic low with very mild gradients and is also within the golf course.

3.7 SUBSURFACE CONDITIONS

A brief description of the materials encountered in each boring is presented below. For a more detailed description of the soil conditions encountered in our drill holes, refer to the drill hole logs in Appendix A.

DH-1 and DH-2 were located at the PGWTP tank site and extended between 19.5 and 24 feet below ground surface. These boring encountered dune deposits underlain by granite bedrock. The dune deposits consist of Poorly Graded Sand with Clay and are medium dense in the upper two feet, and variably looser in density at variable intervals between about 3 and 12 feet below ground surface. The density increases below about 9 to 12 feet, with granite bedrock located at about 16 feet below ground surface. The granite is severely weathered in the upper portion and increases in density/strength with depth.

DH-3 and DH-4 were located on the northwest and southwest sides of Crespi Pond in the areas of its proposed expansion. Both encountered dune deposits overlying granite. In this topographic swale, the dune deposits have greater fine grained material and consist of medium dense to loose Poorly Graded Sand with Clay to Clayey Sand. Peat rich sand was encountered just above the bedrock in DH-4. Granite bedrock was encountered in both borings at 3.5 feet (DH-3) and 7.5 feet (DH-4) below ground surface. In DH-3 the field measured blow counts were high, indicating dense granite bedrock at 5 feet below ground surface. In DH-4 the granite appeared relatively soft in rock hardness from 7.5 to 12 feet below ground surface and then became denser/stronger at 12 feet below ground surface.

DH- 5 was located in the area of the proposed wetland/pond. It encountered 12 feet of Poorly Graded Sand with Clay underlain by granite at 12.5 feet below ground surface, both of which displayed similar composition and to the other drill holes.

DH-6 was located near the intersection of Del Monte Boulevard and Egan Avenue. This drill hole encountered 2 feet of older dune deposits consisting of a medium dense Clayey Sand. Weathered granite was encountered at 2 feet below ground surface and denser granite at 7 feet below ground surface.

3.8 GROUNDWATER

Groundwater was not encountered in any of our borings (DH-1 through DH-6). These conditions likely do not reflect stabilized groundwater depths, and are likely variable. Based on the site geology we interpret that groundwater will locally pond on the granite bedrock surface after heavy rainfall and then drain outwards towards the ocean. In the area of the PGWTP tanks, subsurface drainage is expected to occur relatively quickly and to flow radially outwards as the water is released along the Point Pinos bluffs. In the area around Crespi Pond, subsurface drainage is likely slower and focused northward by the topographic and bedrock swale.

Golf course maintenance personnel report that even in the topographic low areas the fairway is drivable relatively quickly after rains, indicating that drainage of perched water is fairly rapid.

Groundwater depth is subject to fluctuations depending on rainfall, golf course irrigation, pumping in local wells, or other factors that may not be evident at the time of our investigation.

3.9 TOP OF BEDROCK SURFACE

The site is located on a marine terrace surface. Geomorphically, these surfaces are cut by wave action and therefore tend to be quite planar, and nearly level when formed. Our borings are consistent with this, encountering top-of-bedrock at an approximate elevation of 7 feet in DH-2, and an elevation of 5 feet in DH-1, a difference in elevation of 2 feet across a distance of over 100 feet. In general, we expect that most of the top-of-bedrock surface would be similarly planar. Locally incised and buried drainageways may exist (such as that occupied by Crespi Pond), and bedrock would be encountered at greater depths in these areas.

Bedrock encountered at the maximum depth of our drill holes was soft in rock hardness and crumbled to sand size material in the samplers, even when the SPT blow counts were very high. Based on this we infer that material will be excavatable with conventional equipment, but extra time and horsepower will likely be required. Generally speaking, rock quality is expected to improve with depth.

Based on our borings and the topographic maps that were provided, the depth to bedrock, dense bedrock and the elevation of dense bedrock is estimated Table 3. Elevations are based on the datum provided to us by Fall Creek Engineering as shown in Figure 5:

Table 3. Summary of Bedrock Depth and Elevation

Location	Depth Below Ground Surface to Top of Bedrock (feet)	Depth Below Ground Surface to Dense Bedrock (feet)	Elevation of Dense Bedrock (feet)
PGWTP Site (DH-1 and DH-2)	16 to 16.5	16 to 16.5	Elev. 5 to 7
Crespi Pond (DH-3 and DH-4)	3.5 to 7.5	5 to 12	Elev. 10 to 12
Proposed Wetland (DH-5)	12.5	12.5	Elev. 26

3.10 VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil, bedrock and groundwater conditions, as described in this report, are based on data obtained from subsurface exploration and laboratory testing for this study, and from subsurface data obtained by others. Our conclusions and recommendations are based on these interpretations. The project area has undergone different phases of land usage, with associated grading. Therefore, it is likely that undisclosed variations in subsurface conditions exist within the project area, such as old foundations, abandoned utilities and localized fill deposits of unknown character. Additionally, the hardness of granite bedrock will be locally variable.

We recommend that careful observations be made during construction to verify our interpretations. Should variations from our interpretations be found, we should be notified to evaluate whether any revisions should be made to our recommendations.

4. DISCUSSION AND CONCLUSIONS

4.1 GENERAL

Based on the results of our investigation, we conclude that the site is geotechnically suitable for the proposed improvements, provided the recommendations presented in this report are followed. A review of our conclusions with respect to various hazards is presented below. Detailed recommendations are presented in Section 5.

4.2 SURFACE RUPTURE AND SEISMIC GROUND SHAKING

Because the project area is not located within a State of California Earthquake Fault Zone and no mapped active faults are known to cross the project area, the probability of ground surface rupture at the project area due to displacement along a fault is remote.

The project area is in a region of high seismicity. Based on general knowledge of the local seismicity, it should be anticipated that, during its useful life, the project area will be subject to strong ground shaking. It is also anticipated that the project area will periodically experience small to moderate magnitude earthquakes. Proposed improvements should be designed accordingly.

4.3 SHALLOW BEDROCK AND EXCAVATABILTY

Shallow granite bedrock exists at variable elevations across the site. Granite bedrock, depending on the degree of weathering, may be very difficult to excavate. Approximate depth to dense bedrock as encountered in our borings is summarized in Section 3.9.

Based on DH-3 and DH-4, it appears that hard rock could be encountered at shallow depths below Crespi Pond water elevation and that excavating up to 10 feet below the pond water elevation may be difficult. However, the bedrock surface may dive deeper as one approaches the centerline of the topographic swale in this area, allowing greater excavatability.

Underground contractors should be aware of the presence of shallow bedrock and employ suitable equipment for these conditions. Heavy ripping, jack hammering, and other appropriate means may be required.

4.4 LIQUEFACTION AND LATERAL SPREADING

Soil liquefaction is a phenomenon in which saturated granular soils, and certain fine-grained soils, lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and non-plastic silts. Certain gravels, plastic silts, and clays are also susceptible to liquefaction. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type; 3) relative density of granular soils; 4) moisture content and plasticity of fine-grained soils; 5) overburden pressure; and 6) depth to groundwater.

The regional liquefaction susceptibility mapping reviewed in Section 4.4 considers the marine terrace deposits underlying the PGWTP tank site and the wetland area to have a low liquefaction susceptibility. The alluvial deposits that are mapped to underlay Crespi Pond are mapped as having a high liquefaction potential. Our borings encountered materials we classified as dune deposits, because they are sand-sizes or finer, with fairly uniform grain size, and no large clasts. The dune sand deposits are relatively clean with low fines content and are potentially liquefiable depending on their density and the depth of groundwater. At the PGWTP tank site potentially liquefiable soils were encountered in isolated zones between about 3 and 12 feet below ground surface.

The setting of the PGWTP tank site is such that groundwater generated from rainfall reaches the top of bedrock, and rather than perch on top of the bedrock, it tends to drain away relatively rapidly. This is due to the site being located near the northern end of a bedrock peninsula with virtually no contributing watershed, and the existence of free-draining faces along the bluff margins. None of our borings at the site encountered groundwater during drilling. From the groundwater data we were able to obtain in this investigation (see Section 3.8), we infer that for the majority of the year groundwater does not saturate the 16 feet of soil that mantles the bedrock at the PGWTP site. Therefore, the probability that temporarily perched groundwater would occur at the same time as a major earthquake is low. With this condition, in our opinion the hazard of liquefaction at the site is low.

Because there is a low potential for liquefaction at the PGWTP tanks site, there is a correspondingly a low potential of lateral spreading in this area of the site.

Crespi Pond and the site for a future wetlands are located in a topographic low. Our borings did not encounter ground water in these areas either, but based on the site geology there is a greater likelihood that perched groundwater may remain trapped in these areas for longer periods of time. Also, use of the area as wetlands may result in raising the groundwater elevation. We analyzed the liquefaction potential in these areas based on a PGA value of 0.41g (see Section 3.3), an earthquake moment magnitude of 7.3, and a perched groundwater depth of 5 feet below ground surface.

The results of our liquefaction analysis suggest that the sand layers from 5 to 7 feet in DH-3 and from 5 to 8 feet in DH-4 are potentially liquefiable. Estimated liquefaction-induced ground settlements for these layers are 1/4 inch to 1/3 inch respectively. Case histories have shown that actual liquefaction-induced settlements could be 50 to 200 percent of the estimated values.

4.5 SETTLEMENT OF WATER TANK FOUNDATIONS

Seismic and static settlement of the 0 to 3.5 foot thick soil layer between the bottom of the tanks and the underlying bedrock is a potential issue. During construction of the existing tanks, it is possible that the soil layer that exists between the bottom of the tanks and the bedrock surface was subexcavated and recompacted in place, thus reducing the magnitude of seismic and static settlement of this layer.

The information from our surrounding drill holes may not be representative of the soils beneath the tank foundations, but we judge that they represent a "worst case" condition. Assuming this worst case condition the seismically induced settlement of this area is judged to be less than about ½ inch.

As long as the loads within the tanks (the maximum water level) are similar to the loads they experienced when they were in service, additional static settlement should be minor. Due to the age of the tank it is likely that static settlement occurred years ago.

4.6 EXCAVATIONS AND DEWATERING

Proposed excavations within the Crespi Pond area are planned to be 5 to 10 feet below existing ground surface. Excavation depths for utilities are not presently known. Excavations within the dune sand deposits will encounter cohesionless materials that are subject to collapse and will require shoring or sloping the excavation sidewalls. Detailed recommendations are provided in Section 5.2.

Depending on the time of year of construction, if groundwater is encountered during construction, dewatering may also be required to allow construction to proceed in a "dry" condition.

4.7 EXPANSION POTENTIAL OF NEAR-SURFACE SOILS

The near-surface soils are generally sands with a low percentage of fines. These types of soil generally have low expansion potential.

5. RECOMMENDATIONS

5.1 GENERAL

Recommendations are provided in this section for expansion of the Crespi Pond and the wetland area, for assessment of the existing PGWTP tanks and for construction of utility trenches. General recommendations are provided for other improvements in the area. If new tanks or other improvements are proposed, we request the opportunity to review them to evaluate if our recommendations are suitable.

5.2 EARTHWORK

5.2.1 Clearing and Grubbing

Clearing and grubbing should be performed in areas proposed for earthwork, concrete slabs-on-grade or other development. Clearing and grubbing should include clearing of existing structures, utility lines to be abandoned, deleterious materials, debris, obstructions, and stumps and primary roots of trees and brush (roots over 1 inch in diameter or longer than about 3 feet in length). Depressions, voids and holes that extend below the proposed finish grade should be cleaned and backfilled with engineered fill.

Surface vegetation and organic laden soils should be stripped. Organic laden soils are defined as soils with more than 3 percent by weight of organic content. The required stripping depth should be determined in the field by the Engineer at the time of construction. Stripped material may be stockpiled for use in landscape areas if approved by the project landscape architect, or otherwise removed from the site.

5.2.2 Excavations, Shoring and Dewatering

Excavations up to 10 feet are anticipated for expansion of the Crespi Pond and the new wetland area. Excavations of unknown depth are anticipated for utility lines. Excavations may encounter hard granite bedrock conditions (see Section 3.9 for approximate depth to dense rock encountered in our drill holes). Based on the materials encountered in our borings we infer that excavations can be accomplished with conventional equipment, supplemented by heavy rippers. However, jack hammers in hard granite may be necessary. All underground contractors should be prepared for hard shallow rock conditions.

The contractor is responsible for the design, installation, maintenance and removal of temporary shoring and bracing systems. The presence of nearby existing structures, pavements, and underground utilities must be incorporated in the design of the shoring and bracing systems. The presence of relatively clean sandy soils that are subject to sudden collapse should be taken into consideration in design and construction.

Groundwater was not encountered in any of our drill holes but is expected to seasonally pond on the bedrock surface for short periods of time. Dewatering systems may be necessary. The design, installation, permitting, maintenance and removal of dewatering systems are the responsibility of the contractor.

Excavations adjacent to existing or proposed foundations should be above an imaginary plane having an inclination of 1½:1 (horizontal to vertical) extending down from the top of the foundations. Otherwise, the effect of the adjacent foundations should be incorporated in the design and construction of the excavations and improvements.

5.2.3 Subgrade Preparation

Subgrade preparation is recommended in areas to receive engineered fill, or to support improvements such as pavements, concrete slabs, etc. Where subgrade preparation is required, the subgrade should be compacted to the recommendations given under Section 5.2.5. "Engineered Fill Placement and Compaction."

Soil with moisture content above optimum value should be anticipated during and shortly after rainy seasons, or for soils below the groundwater level. Where unstable, wet or soft soil is encountered, the soil will require processing before compaction can be achieved. When construction schedule does not allow air-drying, other means such as lime treatment of the soil or excavation and replacement may be considered. Geotextile fabrics may also be used to help stabilize the subgrade. The method to be used should be determined at the time of construction based on the actual site conditions. We recommend obtaining unit prices for subgrade stabilization during the construction bid process.

5.2.4 Material for Engineered Fill

In general, on-site soils with an organic content of less than 3 percent by weight, free of any hazardous or deleterious materials, and meeting the gradation requirements below may be used as general engineered fill to achieve project grades, except when special material is required.

In general, engineered fill material should not contain rocks or lumps larger than 3 inches in greatest dimension, should not contain more than 15 percent of the material larger than 1½ inches, and should contain at least 10 percent passing the No. 200 sieve. In addition to these requirements, import fill should have a low expansion potential as indicated by Plasticity Index of 12 or less, or Expansion Index of less than 20.

All import fills should be approved by the project geotechnical engineer prior to delivery to the site. At least five (5) working days prior to importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

5.2.5 Engineered Fill Placement and Compaction

Engineered fill should be placed in horizontal lifts each not exceeding 8 inches in thickness, moisture conditioned to the required moisture content, and mechanically compacted. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory maximum dry density as determined by ASTM Test Method D1557, latest edition, expressed as a percentage.

Moisture conditioning of soils should consist of adding water to the soils if they are too dry and allowing the soils to dry if they are too wet.

Engineered fills consisting of on-site or imported soils should be compacted to a minimum of 90 percent relative compaction. The moisture content of the material should be brought to between 1 and 3 percent above the laboratory optimum value before compaction is performed. In pavement areas, the upper 8 inches of soil should be compacted to a minimum of 95 percent relative compaction.

5.2.6 Cut and Fill Slopes

Generally, cut and fill slopes in sandy soil should be constructed at inclinations no steeper than 2.5:1 (horizontal:vertical). Permanent cut slopes within the metamorphic rock may be constructed at inclinations as steep as 1:1 (horizontal:vertical).

All pavements and concrete slabs-on-grade should be set back at least 5 feet horizontally from the crests of cut or fill slopes.

It may be desirable to lay back cut slopes in sand to as flat as 3:1 on the margins of Crespi Pond and the proposed wetland area. The stability of saturated slopes on the pond margins will be dependent on the percentage of fines within the material and to what degree vegetation is established.

5.2.7 Utility Trench Excavation and Backfill

Trench excavation, bedding and backfill should conform to the City of Pacific Grove Standard Specifications. Construction, shoring, and bracing of excavations should comply with the current CAL-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor.

5.2.8 Wet Weather Construction

If site grading and construction is to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads or subgrades, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors. The contractor is solely responsible to protect his work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

5.3 WATER TANK FOUNDATIONS

Based on the information provided to us, we understand that the eastern tank (the clarifier) has a base foundation consisting of a 14-inch thick concrete mat that slopes from 13 feet at the edges to 16.5 feet below ground surface at the center. The western tank (the digester) has a flat, 18-inch thick concrete mat that is founded 10 feet below ground surface.

For evaluation of these two tanks we recommend a net allowable bearing capacity of 2500 pounds per square foot on the underlying soils when considering dead plus normal live loading. This allowable foundation soil pressure may be increased by one-third when considering short-term wind or seismic loading. This assumes the existing embedment depths as noted above. Static settlement of the tanks is expected to have already occurred. Total settlement due to seismic shaking may be on the order of ½ inch .

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of the mat foundations and underlying soils and by passive pressures acting against the embedded sides of the tanks. For frictional resistance at the base of the tanks, an ultimate coefficient of friction of 0.3 may be used for design. In addition, an allowable passive lateral bearing pressure equal to an equivalent fluid pressure of 300 pounds per cubic foot (pcf) may be used. These values may be used in combination without reduction. This passive pressure can be assumed to act from 2 feet below grade and downward.

The side walls of the tanks should be designed to retain the surrounding soil. We infer that at-rest soil pressures are applicable for the tank walls as they are restrained from deflecting at the top. Assuming drained backfill conditions, the tank walls should be designed to resist an equivalent fluid pressure of 55 pcf.

If the structural engineer wishes to include seismic forces in the design of tank walls, the walls may be evaluated using the above at-rest soil pressure plus a horizontal seismic line force of $10H^2$ pounds per lineal foot (where H is the height of the vertical design plane from the wall base to the ground surface above). The resultant of the seismic force should be applied at 2/3H above the wall base. A reduced factor of safety for overturning and sliding may be used in seismic design as determined by the structural designer.

5.4 CONCRETE SLABS-ON-GRADE

No specific exterior concrete slabs-on-grade are presently proposed. The following recommendations are for exterior slabs in general. Preparation of subgrade soil and placement and compaction of engineered fill for concrete slabs-on-grade should be as outlined in Section 5.2, the "Earthwork" section of this report.

Exterior concrete slabs that are not sensitive to moisture transmission through the slabs, such as exterior flatwork may be constructed directly on properly prepared soil subgrades. Design of reinforcement, joint spacing, etc. is the responsibility of the design engineer.

Exterior concrete slabs-on-grade should be cast free from adjacent foundations or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure.

5.5 SURFACE DRAINAGE

Engineering design of grading and drainage at the site is the responsibility of the project Civil Engineer. We recommend the following be considered by the project Civil Engineer and incorporated into the project plans where appropriate.

Sufficient surface drainage should be provided to direct runoff away from building foundations, concrete slabs-on-grade and pavements, and towards suitable collection and discharge facilities. Ponding of surface water should be avoided by establishing positive drainage away from all improvements. Water collected from roof downspouts should be discharged into a closed pipe or towards drainage structures, and the water carried to a suitable discharge point.

The dune sand deposits are highly erodible and care should be taken to provide erosion protection where water is discharged and to plant and mulch all disturbed surfaces, establishing vegetation as appropriate, prior to the winter rains.

6. PLAN REVIEW, EARTHWORK AND FOUNDATION OBSERVATION

Post-report geotechnical services by Pacific Geotechnical Engineering (PGE), typically consisting of pre-construction design consultations and reviews, construction observation and testing services, are necessary for PGE to confirm the recommendations contained in this report. This report is based on limited sampling and investigation, and by those constraints may not have discovered local anomalies or other varying conditions that may exist on the project site. Therefore, this report is only preliminary until PGE can confirm that actual conditions in the ground conform to those anticipated in the report. Accordingly, as an integral part of this report, PGE recommends post-report geotechnical services to assist the project team during design and construction of the project. PGE requires that it perform these services if it is to remain as the project geotechnical engineer-of-record.

During design, PGE can provide consultation and supplemental recommendations to assist the project team in design and value engineering, especially if the project design has been modified after completion of our report. It is impossible for us to anticipate every design scenario and use of construction materials during preparation of our report. Therefore, retaining PGE to provide post-report consultation will help address design changes, answer questions and evaluate alternatives proposed by the project designers and contractors.

Prior to issuing project plans and specifications for construction bidding purposes, PGE should review the grading, drainage and foundation plans and the project specifications to determine if the intent of our recommendations has been incorporated in these documents. We have found that such a review process will help reduce the likelihood of misinterpretation of our recommendations which may cause construction delay and additional cost.

Construction phase services can include, among other things, the observation and testing during site clearing, stripping, excavation, mass grading, subgrade preparation, fill placement and compaction, backfill compaction, foundation construction and pavement construction activities.

Pacific Geotechnical Engineering would be pleased to provide cost proposals for follow-up geotechnical services. Post-report geotechnical services may include additional field and laboratory services.

7. LIMITATIONS

In preparing the feasibility-level findings and professional opinions presented in this report, we have endeavored to follow generally accepted principles and practices of the engineering geologic and geotechnical engineering professions in the area and at the time our services were provided. No warranty, express or implied, is provided.

The preliminary recommendations contained in this report are based, in part, on information that has been provided to us. In the event that the general development concept or general location and type of structures are modified, our preliminary conclusions and recommendations shall not be considered valid unless we are retained to review such changes and to make any necessary additions or changes to our recommendations. For Pacific Geotechnical Engineering to remain the geotechnical consultant of record for the proposed project, we must provide supplemental geotechnical services during final design phase, plan review and construction observation services, as outlined above under the Plan Review, Earthwork and Foundation Observation section of this report.

Subsurface exploration is necessarily confined to selected locations and conditions may, and often do, vary between these locations. Should conditions different from those assumed in this report be encountered during project development, additional exploration, testing, and analysis may be required.

Should persons concerned with this project observe geotechnical features or conditions at the site or surrounding areas which are different from those described in this report, those observations should be reported immediately to Pacific Geotechnical Engineering for evaluation.

It is important for project performance that the preliminary recommendations given in this report are made known to the design professionals involved with the project, that they be incorporated into project drawings and documents, and that the preliminary recommendations be validated and/or supplemented by a design level geotechnical investigation.

Report prepared by,

PACIFIC GEOTECHNICAL ENGINEERING

Soma B. Goresky
GE 2252

G. Reid Fisher PhD
CEG 1858

8. REFERENCES

California Geological Survey, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California: CGS Special Publication 117A.

Clark, J.C., Dupre, W.R., and Rosenberg, Lewis I., 1997, Geologic Map of the Monterey and Seaside 7.5-minute Quadrangles, Monterey County, California: A Digital Database: US Geological Survey OFR 97-030.

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Wagner , D.L, Greene, H. Gary, Saucedo, G.J., and Pridmore, C.L., 2002, Geologic Map of the Monterey 30'X60' Quadrangle and Adjacent Areas, California, California Geological Survey Regional Geologic Map No. 1, 1:100,000 scale.

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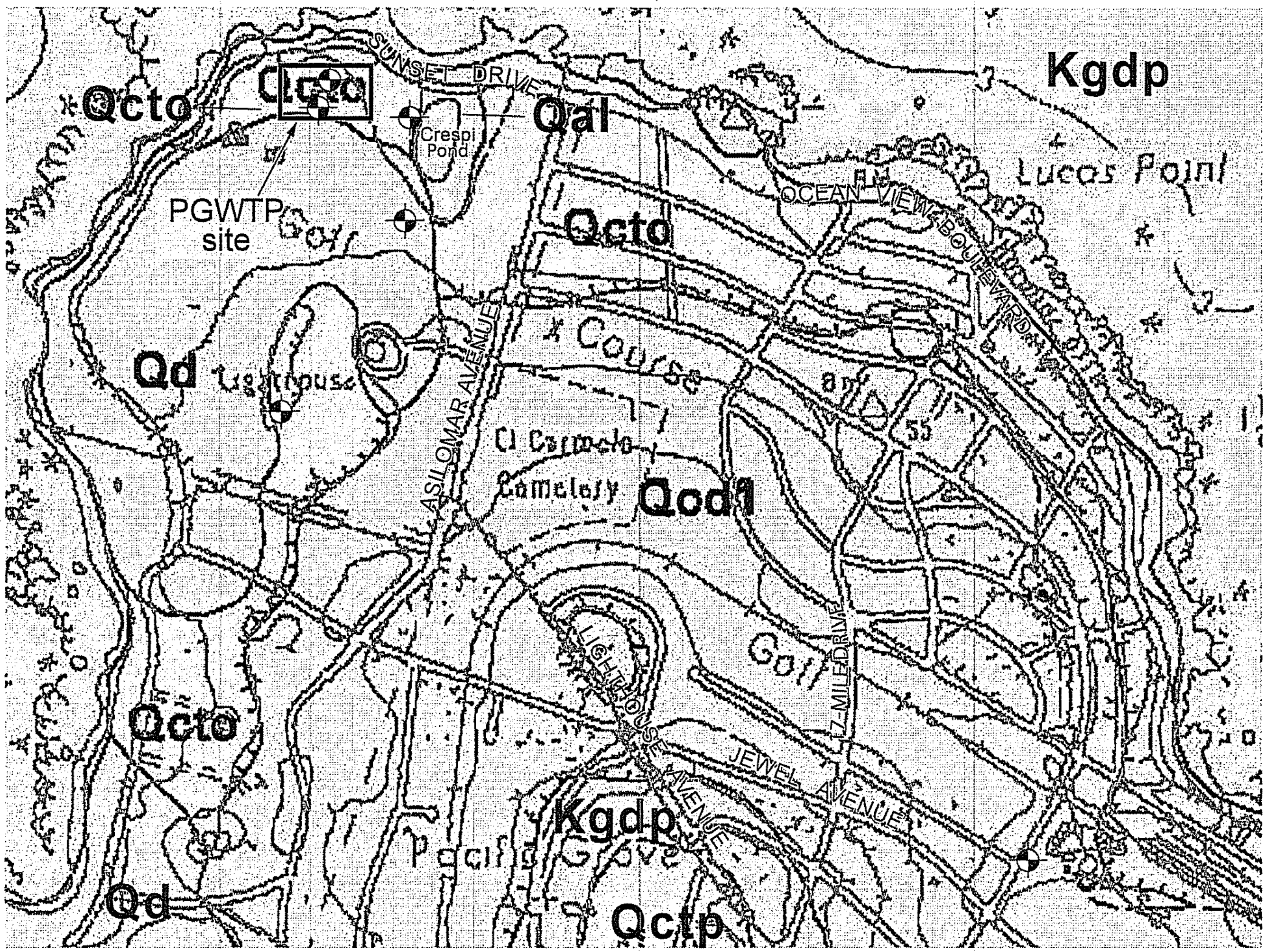
The Working Group on California Earthquake Probabilities, 1996, Database of potential sources for earthquakes larger than magnitude 6 in northern California: U.S. Geological Survey Open-File Report 96-705.

FIGURES



EXPLANATION

Qd	Dune sand deposits (Holocene)
Qal	Alluvial deposits (Holocene)
Qod1	Younger coastal dune deposits (Pleistocene)
Qcto	Ocean view coastal terrace (Pleistocene)
Qctl	Lighthouse coastal terrace (Pleistocene)
Qctp	Peninsula College coastal terrace (Pleistocene)
Kgdp	Porphyritic granodiorite of Monterey of Ross (Cretaceous)
	Drill hole location

N

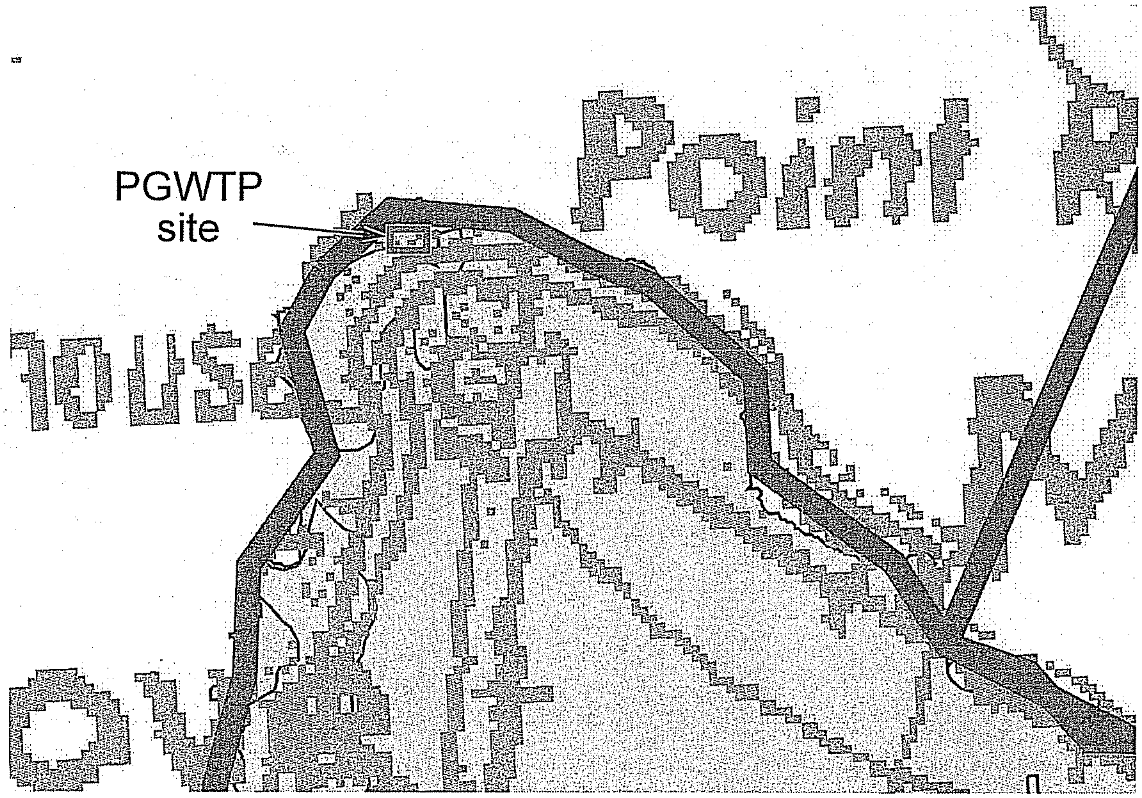
0 500 ft

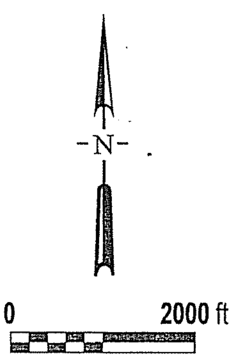
NOTES:

1. BASE MAP: Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles (J.C. Clark and others, 1997).
2. Drill hole locations are shown on this figure to illustrate the overall distribution of exploration. See Figures 4 and 5 for specific locations of drill holes.

PACIFIC GEOTECHNICAL ENGINEERING

DRAFTED BY: CSS	**GEOLOGIC INDEX MAP**	FIGURE 1
DATE: AUGUST 2013	PACIFIC GROVE ASBS STORMWATER MANAGEMENT PROJECT MONTEREY and PACIFIC GROVE, CALIFORNIA	PROJECT 2013.0031
CHECED BY:		
REVISION:		



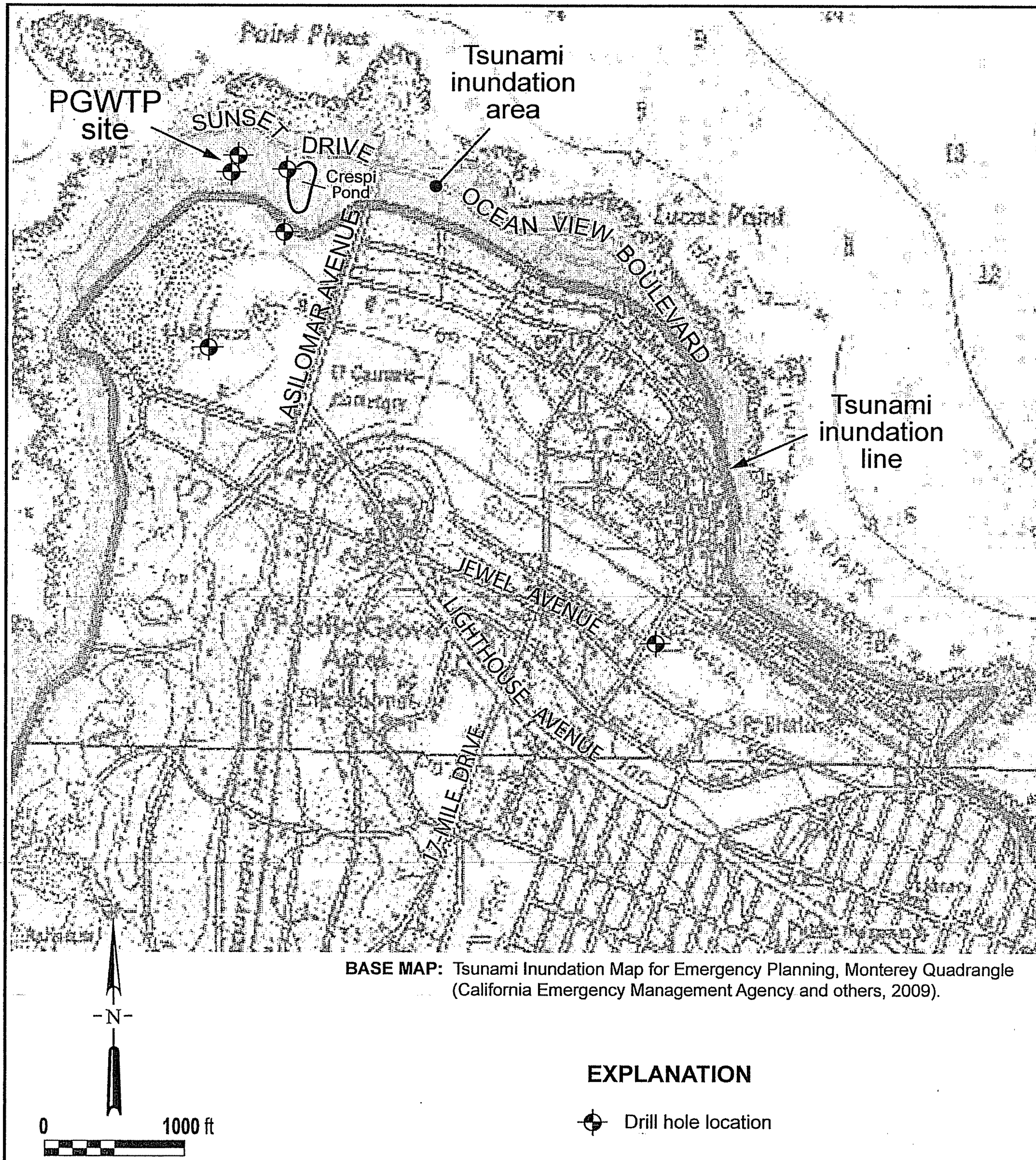


EXPLANATION

- High liquefaction susceptibility
- Low liquefaction susceptibility

BASE MAP: Digital Map Showing Relative Liquefaction Susceptibility of Monterey County, California (L.I. Rosenberg, 2001).

PACIFIC GEOTECHNICAL ENGINEERING	DRAFTED BY: CSS	LIQUEFACTION MAP	FIGURE 2
	DATE: AUGUST 2013	PACIFIC GROVE ASBS STORMWATER MANAGEMENT PROJECT MONTEREY and PACIFIC GROVE, CALIFORNIA	
	CHECED BY:		PROJECT 2013.0031
	REVISION:		



BASE MAP: Tsunami Inundation Map for Emergency Planning, Monterey Quadrangle (California Emergency Management Agency and others, 2009).

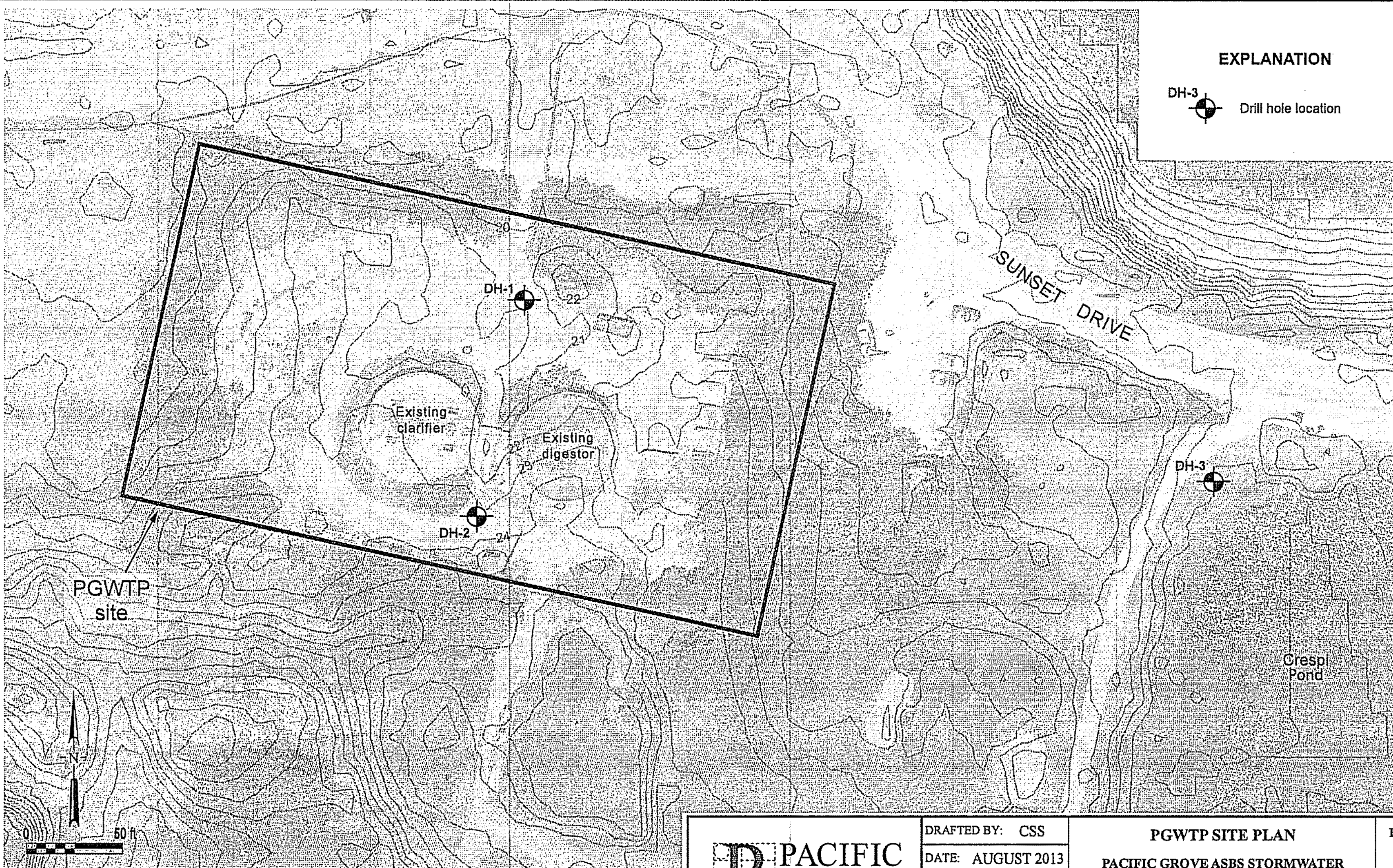
EXPLANATION

Drill hole location

PACIFIC GEOTECHNICAL ENGINEERING	DRAFTED BY: CSS	**TSUNAMI INUNDATION MAP**	**FIGURE** 3
	DATE: AUGUST 2013	**PACIFIC GROVE ASBS STORMWATER MANAGEMENT PROJECT MONTEREY and PACIFIC GROVE, CALIFORNIA**	
	CHECED BY:		**PROJECT** 2013.0031
	REVISION:		

BASE MAP: "Pacific Grove Water Treatment Plant Vicinity, Pacific Grove ASBS Stormwater Management Plan," by Fall Creek Engineering, Inc., received electronically on 8/5/13.

PACIFIC GEOTECHNICAL ENGINEERING	DRAFTED BY: CSS	OVERALL SITE PLAN	FIGURE 4
	DATE: AUGUST 2013	PACIFIC GROVE ASBS STORMWATER MANAGEMENT PROJECT MONTEREY and PACIFIC GROVE, CALIFORNIA	
	CHECED BY:		PROJECT 2013.0031
	REVISION:		



BASE MAP: "Pacific Grove Water Treatment Plant Contour Data, Pacific Grove ASBS Stormwater Management Plan," by Fall Creek Engineering, Inc., received electronically on 8/5/13.

PACIFIC GEOTECHNICAL ENGINEERING	DRAFTED BY: CSS	PGWTP SITE PLAN	FIGURE 5
	DATE: AUGUST 2013	PACIFIC GROVE ASBS STORMWATER MANAGEMENT PROJECT MONTEREY and PACIFIC GROVE, CALIFORNIA	
	CHECED BY:		PROJECT 2013.0031
	REVISION:		

APPENDIX A

KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS
(50% OR MORE IS SMALLER THAN NO. 200 SIEVE SIZE)
(modified from ASTM D2487 to include fine grained soils with intermediate plasticity)

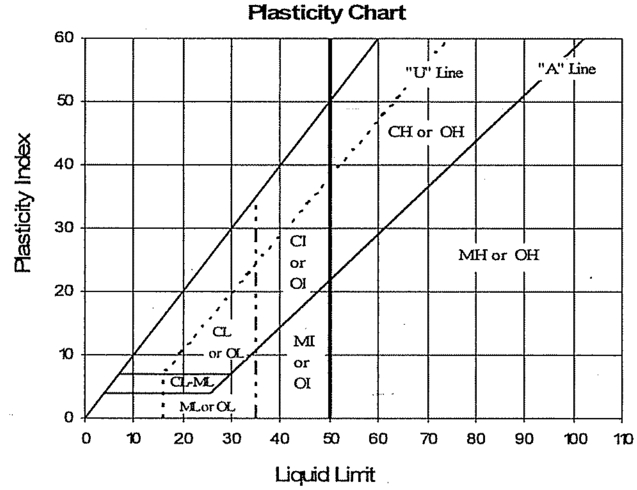
MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES
SILTS AND CLAYS (Liquid Limit less than 35) Low Plasticity	Inorganic	PI < 4 or plots below "A" line	ML	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above "A" line	CL	Lean Clay, Lean Clay with Sand or Gravel, Sandy or Gravelly Lean Clay, Sandy or Gravelly Lean Clay with Sand or Gravel
	Inorganic	PI between 4 and 7	CL-ML	Silty Clay, Silty Clay with Sand or Gravel, Sandy or Gravelly Silty Clay, Sandy or Gravelly Silty Clay with Sand or Gravel
	Organic	See footnote 3	OL	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)
SILTS AND CLAYS (35 ≤ Liquid Limit < 50) Intermediate Plasticity	Inorganic	PI < 4 or plots below "A" line	MI	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above "A" line	CI	Clay, Clay with Sand or Gravel, Sandy or Gravelly Clay, Sandy or Gravelly Clay with Sand or Gravel
	Organic	See footnote 3	OI	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)
SILTS AND CLAYS (Liquid Limit 50 or greater) High Plasticity	Inorganic	PI plots below "A" line	MH	Elastic Silt, Elastic Silt with Sand or Gravel, Sandy or Gravelly Elastic Silt, Sandy or Gravelly Elastic Silt with Sand or Gravel
	Inorganic	PI plots on or above "A" line	CH	Fat Clay, Fat Clay with Sand or Gravel, Sandy or Gravelly Fat Clay, Sandy or Gravelly Fat Clay with Sand or Gravel
	Organic	See note 3 below	OH	Organic Silt (below "A" Line) or Organic Clay (on or above "A" Line) (1,2)

1. If soil contains 15% to 29% plus No. 200 material, include "with sand" or "with gravel" to group name, whichever is predominant.
2. If soil contains ≥30% plus No. 200 material, include "sandy" or "gravelly" to group name, whichever is predominant. If soil contains ≥15% of sand or gravel sized material, add "with sand" or "with gravel" to group name.
3. Ratio of liquid limit of oven dried sample to liquid limit of not dried sample is less than 0.75.

CONSISTENCY	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 – 0.5	2 – 4
FIRM	0.5 – 1.0	5 – 8
STIFF	1.0 – 2.0	9 – 15
VERY STIFF	2.0 – 4.0	16 – 30
HARD	> 4.0	> 30

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soil is below the water table

Plasticity Chart



PACIFIC GEOTECHNICAL ENGINEERING

KEY TO SOIL CLASSIFICATION – COARSE GRAINED SOILS
(MORE THAN 50% IS LARGER THAN NO. 200 SIEVE SIZE)
(modified from ASTM D2487 to include fines with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES[1]
GRAVELS (more than 50% of coarse fraction is larger than No. 4 sieve size)	Gravels with less than 5% fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3	GW	Well Graded Gravel, Well Graded Gravel with Sand
		Cu < 4 and/or 1 > Cc > 3	GP	Poorly Graded Gravel, Poorly Graded Gravel with Sand
	Gravels with 5% to 12% fines	ML, MI or MH fines	GW-GM	Well Graded Gravel with Silt, Well Graded Gravel with Silt and Sand
			GP-GM	Poorly Graded Gravel with Silt, Poorly Graded Gravel with Silt and Sand
		CL, CI or CH fines	GW-GC	Well Graded Gravel with Clay, Well Graded Gravel with Clay and Sand
			GP-GC	Poorly Graded Gravel with Clay, Poorly Graded Gravel with Clay and Sand
	Gravels with more than 12% fines	ML, MI or MH fines	GM	Silty Gravel, Silty Gravel with Sand
		CL, CI or CH fines	GC	Clayey Gravel, Clayey Gravel with Sand
		CL-ML fines	GC-GM	Silty Clayey Gravel; Silty, Clayey Gravel with Sand
SANDS (50% or more of coarse fraction is smaller than No. 4 sieve size)	Sands with less than 5% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3	SW	Well Graded Sand, Well Graded Sand with Gravel
		Cu < 6 and/or 1 > Cc > 3	SP	Poorly Graded Sand, Poorly Graded Sand with Gravel
	Sands with 5% to 12% fines	ML, MI or MH fines	SW-SM	Well Graded Sand with Silt, Well Graded Sand with Silt and Gravel
			SP-SM	Poorly Graded Sand with Silt, Poorly Graded Sand with Silt and Gravel
		CL, CI or CH fines	SW-SC	Well Graded Sand with Clay, Well Graded Sand with Clay and Gravel
			SP-SC	Poorly Graded Sand with Clay, Poorly Graded Sand with Clay and Gravel
	Sands with more than 12% fines	ML, MI or MH fines	SM	Silty Sand, Silty Sand with Gravel
		CL, CI or CH fines	SC	Clayey Sand, Clayey Sand with Gravel
		CL-ML fines	SC-SM	Silty, Clayey Sand; Silty, Clayey Sand with Gravel

US STANDARD SIEVES	3 Inch		¾ Inch	No. 4	No. 10	No. 40	No. 200
	COARSE	FINE		COARSE	MEDIUM	FINE	
COBBLES & BOULDERS	GRAVELS			SANDS			SILTS AND CLAYS

RELATIVE DENSITY (SANDS AND GRAVELS)	STANDARD PENETRATION (BLOWS/FOOT)
Very Loose	0 - 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 - 50
Very Dense	50+

1. Add "with sand" to group name if material contains 15% or greater of sand-sized particle. Add "with gravel" to group name if material contains 15% or greater of gravel-sized particle.

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soi is below the water table

ROCK QUALITY DESCRIPTIONS

HARDNESS**		WEATHERING**	
Very Hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of the geologist's pick	**Fresh or Unweathered**	Rock fresh, crystals bright, few joints and fractures may show slight staining. Rock rings under hammer if crystalline.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow with hammer required to break sample.	**Very Slight**	Rock generally fresh, fractures and joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Moderately Hard	Can be scratched with knife or pick. Gouges or grooves to ½ inch can be excavated by hard blow of point of a geologist's pick. Hand specimens broken with moderate blow.	**Slight**	Rock generally fresh, joints and fractures stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitic rock, some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Medium	Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips about 1 inch maximum in dimension by hard blows of the point of a geologist's pick.	**Moderate**	Significant portions of rock show discoloration and weathering effects. In granitic rock, most feldspars are dull and discolored; some show clay. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Soft	Can be grooved or gouged readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small pieces can be broken by finger pressure,	**Moderately Severe**	All rock except quartz discolored or stained. In granitic rock, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
Very Soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces one inch or more thickness can be broken with finger pressure. Can be scratched readily by finger nail.	**Severe**	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitic rock, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
		Very Severe	All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
		Complete	Rock reduced to "soil." Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

FRACTURE DIMENSIONS*

Fracture	Block Size (or Spacing[1])
Crushed	~5 microns to 0.1 ft
Intensely	0.05 to 0.1 ft
Closely	0.1 to 0.5 ft
Moderately	0.5 to 1.0 ft
Slightly	1.0 to 3.0 ft
Massive	3.0 ft and larger

1 Average distance between adjacent fractures

* Source of data unknown

** Source of data: "Subsurface Investigaiton for Design and Constructio of Foundation Buildings," (1976) American Society of Civil Engineers, Manuals and Reports on Engineering Practice – No. 5

DATE: 6/4/2013	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: PGWTP **PROJECT NUMBER:** 2013.0031

DRILL RIG: Mobile B53, 140# downhole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** ± 21

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: — Final: —

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
DUNE DEPOSITS: POORLY GRADED SAND WITH CLAY: Very dark brown (10YR 2/2), dry to moist, dense upper, medium dense to loose below; fine sand;	SP-SC	1	S D D	58				3		113		
		2										
		3	S D D	23				4		103		
		4										
		5										
		6	S I I	8		10		3				
		7										
POORLY GRADED SAND WITH CLAY: Brown (7.5YR 4/4), wet, medium dense; fine sand	SP-SC	8										
		9	S I I	17		5		18				
		10										
		11										
POORLY GRADED SAND WITH CLAY: Light brownish grey (10YR 6/2), wet, very dense; fine sand	SP-SC	12										
		13										
		14	S I I	72/10"				22				
- drilling gets abruptly hard at 16 ft.		15										
BEDROCK: GRANITE: Variable brownish yellow, pale brown, light grey, wet, rock mass is soft due to weathering, individual crystal and fragments are very hard, severely weathered; fracture cannot be determined in samples		16										
		17										
		18										
		19	S I I	49								
		20										

DATE: 6/4/2013	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: PGWTP **PROJECT NUMBER:** 2013.0031

DRILL RIG: Mobile B53, 140# downhole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** ± 21

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
BEDROCK: GRANITE: as above - by 23 ft. weathered such that rock separates into individual crystals; no oxidation color in sample, light grey, black, white quartz-plagioclase biotite granitoid		21										
		22										
		23										
		24	I	50/4"								
BOTTOM OF HOLE = 24 Feet No Groundwater encountered		25										
		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

DATE: 6/4/2013	LOG OF EXPLORATORY DRILL HOLE	DH- 2
PROJECT NAME: PGWTP		PROJECT NUMBER: 2013.0031
DRILL RIG: Mobile B53, 140# downhole hammer & wire winch		LOGGED BY: CSS
HOLE DIAMETER: 8" hollow stem auger		HOLE ELEVATION: ± 23
SAMPLER: D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	GROUND WATER DEPTH:	Initial: — Final: —

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
<u>PAVEMENT SECTION</u> (4" AC, no Baserock)												
<u>DUNE DEPOSITS</u>: POORLY GRADED SAND WITH CLAY to CLAYEY SAND: Dark brown (10YR 3/3), dry to moist, medium dense; fine sand	SP-SC to SC	1	S									
		2	D D	40				4		113		
		3	S I									
POORLY GRADED SAND WITH CLAY: Black (10YR 2/1), dry, medium dense; fine sand	SP-SC	4	I	12								
		5	S									
		6	I I	17				20		83		
POORLY GRADED SAND with CLAY and GRAVEL: Dark brown (7.5YR 3/3), moist, medium dense; fine to coarse mostly subangular to angular sand; with fine gravel	SP-SC	7										
		8										
-by 9 ft. dark brown (7.5YR 3/2)		9	S I									
- by 9.75 ft. grades to light brownish grey		10	I	12		10		16				
		11										
		12										
		13										
-by 14.5 ft. greenish grey (Gley1 6/10Y)		14	S I									
		15	I	24		8		20				
<u>BEDROCK</u>: GRANITE: Variably colored, brown overall; wet; rock mass soft due to weathering, crystals are hard; severely weathered to clayey sand, fractures cannot be determined in samples		16										
		17										
		18										
		19	S I	50/5"								
BOTTOM OF HOLE = 19.5 Feet No Groundwater Encountered		20										

DATE: 6/4/2013	LOG OF EXPLORATORY DRILL HOLE	DH- 3

PROJECT NAME: PGWTP **PROJECT NUMBER:** 2013.0031

DRILL RIG: Mobile B53, 140# downhole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** ± 15

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: — Final: —

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
DUNE DEPOSITS: CLAYEY SAND: Very dark brown (10YR 2/2), dry to moist, loose; fine sand	SC	1	S									
		2	D	13								
CLAYEY SAND: Very dark brown (10YR 2/2), moist, medium dense; with fine sand	SC		D			31		18		110		
		3	S									
BEDROCK: (upper 6 in. completely weathered to clayey sand) GRANITE: multi colored grey, white, black with yellowish brown areas from oxidation; dry; rock mass soft due to weathering, crushes into individual crystals			D	15								
		4	D									
		5	S									
		6	D	50/4"		9		9		127		
		7										
		8										
		9	D	50/4"								
BOTTOM OF HOLE = 9 Feet No Groundwater Encountered		10										
		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

DATE: 6/4/2013	LOG OF EXPLORATORY DRILL HOLE	DH- 4
PROJECT NAME: PGWTP		PROJECT NUMBER: 2013.0031
DRILL RIG: Mobile B53, 140# downhole hammer & wire winch		LOGGED BY: CSS
HOLE DIAMETER: 8" hollow stem auger		HOLE ELEVATION: ± 23
SAMPLER: D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	GROUND WATER DEPTH:	Initial: — Final: —

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
DUNE DEPOSITS: POORLY GRADED SAND WITH CLAY: Greyish brown (10YR 5/2), moist, medium dense; fine sand	SP-SC	1	S									
		2	D	30								
CLAYEY SAND: Very dark grayish brown (10YR 3/2), moist, medium dense; fine sand	SC		D		>4.5	15		6		109		
		3	S									
			I	4								
POORLY GRADED SAND WITH CLAY: Grayish brown (10YR 5/2), dry to moist, loose; fine sand	SP-SC	4	I									
		5	S									
-peat rich layer begins at ≈ 6.2 ft. ends of sample 6.5 ft.		6	I I	5								
		7										
WEATHERED BEDROCK: Weathered to CLAYEY SAND: Dark greenish grey (Gley1 4/5GY), moist, medium dense; mostly fine sand.	SC	8										
		9	S									
			I I	10		22		14				
		10										
		11										
BEDROCK: GRANITE: Variable colors but dark yellowish brown over all; moist, rock mass soft due to weathering; crystals are severely weathered; crumbles		12										
		13										
		14	S I	50/6"								
BOTTOM OF HOLE = 14.5 Feet		15										
		16										
		17										
		18										
		19										
		20										

DATE: 6/4/2013	**LOG OF EXPLORATORY DRILL HOLE**	**DH- 5**
PROJECT NAME: PGWTP		**PROJECT NUMBER:** 2013.0031
DRILL RIG: Mobile B53, 140# downhole hammer & wire winch		**LOGGED BY:** CSS
HOLE DIAMETER: 8" hollow stem auger		**HOLE ELEVATION:** ± 38

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: — Final: —

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
DUNE DEPOSITS: CLAYEY SAND: Greyish brown (10YR 5/2), moist to wet, loose; fine sand	SC	1	S									
		2	D D	11								
-by 3 ft. wet with water in sample liner, medium dense		3	S D	25								
? ? ? ? ? ?		4	D			13		16		114		
-No recovery at 5.0-6.5 ft.	SP-SC	5	S									
POORLY GRADED SAND WITH CLAY: Light brown light grey (10YR 6/2), wet, medium dense		6	I I	12								
		7										
		8										
		9	S I	26		6		23				
		10	I									
		11										
-abruptly drilling gets harder		12										
BEDROCK: GRANITE: Variable colored, grey white, black yellowish brown where oxidized; wet; rock mass soft due to weathering; severely weathered rock crumbles into individual rock crystals		13										
		14	S I	50/6"								
BOTTOM OF HOLE = 14.5 Feet No Groundwater Encountered		15										
		16										
		17										
		18										
		19										
		20										

DATE: 6/4/2013	LOG OF EXPLORATORY DRILL HOLE	DH- 6

PROJECT NAME: PGWTP **PROJECT NUMBER:** 2013.0031

DRILL RIG: Mobile B53, 140# downhole hammer & wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** ± 21

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: — Final: —

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
OLDER DUNE DEPOSITS: CLAYEY SAND: Very dark brown (10YR 2/2), dry to moist, medium dense; fine sand; Grades slightly coarser with depth to fine sand with some medium sand	SC	1	S D D	28				4		109		
		2										
WEATHERED BEDROCK: WEATHERED to a CLAYEY SAND: Very dark grey (10YR 3/1), mottled with strong brown (7.5Y 4/6), wet, medium dense to very dense; mostly fine sand		3	S I I	16				15				
		4										
		5	S I	50/5"								
		6										
BEDROCK: GRANITE: Variably colored white, grey, black with yellowish brown oxidation; moist; rock mass is soft due to weathering; severely weathered, crumbles to individual hard minerals		7										
		8	S I	50/4"								
		9										
		10										
		11										
		12										
		13	I	50/4"								
BOTTOM OF HOLE = 14 Feet No Groundwater Encountered		14										
		15										
		16										
		17										
		18										
		19										
		20										