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GEOTECHNICAL STUDY REPORT

DUNPHY PARK IMPROVEMENTS
BRIDGEWAY BOULEVARD & NAPA STREET
SAUSALITO, CALIFORNIA

Project Number:

1993.39.04.1

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TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd)

INTRODUCTION

This report presents the results of our geotechnical study for the improvements to be constructed at Dunphy Park in Sausalito, California. In its current condition, the City of Sausalito's Dunphy Park extends to the east from Bridgeway to Richardson's Bay (Bay) and from the extension of Litho Street on the south to Napa Street on the north. Dunphy Park includes a large grass area, scattered mature trees, a sand volleyball court and bocce courts. There is a gravel parking lot along the northern (Napa Street) edge of the park that also extends along a portion of the easterly edge fronting the Cass Gidley Marina and the Cruising Club. An unpaved overflow lot/access roadway exists along the western edge of the Park in former railroad right-of-way. An existing structure is located at Cass/Gidley where the parking lot transitions from west to north. At this corner, a spit extends out into the Bay for a short distance. The Cass/Gidley office building is located on the spit. An asphalt paved bike lane extends along the southerly edge of the park, parallel to Bridgeway. The site location is shown on Plate 1, Appendix A.

We understand the schematic master plan presented to the City Council involves moving the bocce courts to a location adjacent to the volleyball court on the westerly side of the park. A restroom building is planned in the same area as the bocce and volleyball courts. In addition, new fill is proposed within the park to create a bowl-shaped area for use as an amphitheater. The existing parking lot is to be improved and expanded to cover a larger area in the westerly portion of the park and wrap around a portion of the southerly edge of the park. New pathways and walkways are also planned. In addition, improvements are planned by Cass/Gidley and may in the future be anticipated by the Cruising Club. These improvements may include new and/or modified foundations.

SCOPE

The purpose of our study, as outlined in our proposal dated January 27, 2015 and revised February 9, 2015, was to generate geotechnical information for the design and construction of the project. Our scope of services included reviewing selected published geologic data pertinent to the site; evaluating subsurface conditions with borings and laboratory tests; analyzing the field and laboratory data; and presenting this report with the following geotechnical information:

1. A brief description of soil, bedrock and groundwater conditions observed during our study;

2. A discussion of seismic hazards that may affect the proposed improvements; and

3. Conclusions and recommendations regarding:

    a. Primary geotechnical engineering concerns and mitigating measures, as applicable;

    b. Site preparation and grading including remedial grading of weak, porous, compressible and/or expansive surface soils;

    c. Foundation type(s), design criteria, and estimated settlement behavior;

    d. Support of concrete slabs-on-grade;

e. Preliminary pavement thickness based on our experience with similar soils and projects and the results of an R-value test on the anticipated subgrade soils;

f. Utility trench backfill;

g. Geotechnical engineering drainage improvements; and

h. Supplemental geotechnical engineering services.

STUDY

Site Exploration

We reviewed selected geologic references pertinent to the site. The geologic literature reviewed is listed in Appendix B.

On March 12 and 13, 2015, we performed a geotechnical reconnaissance of the site and explored the subsurface conditions by drilling three borings to depths ranging from about 49½ to 63½ feet. These borings were drilled with a track-mounted drill rig capable of rotary wash drilling. In addition, we drilled one boring to a depth of about 5 feet using the 4-inch diameter, solid stem augers from the same track-mounted drill rig. The borings were drilled at the approximate locations shown on the Exploration Plan, Plate 2. The boring locations were determined approximately by pacing their distance from features shown on the Exploration Plan and should be considered accurate only to the degree implied by the method used. Our geologist located and logged the borings and obtained samples of the materials encountered for visual examination, classification and laboratory testing.

Relatively undisturbed samples were obtained from the borings at selected intervals by driving a 2.43-inch inside diameter, split spoon sampler, containing 6-inch long brass liners, using a 140-pound hammer dropping approximately 30 inches. The sampler was driven 12 to 18 inches. The blows required to drive each 6-inch increment were recorded and the blows required to drive the last 12 inches, or portion thereof, were converted to equivalent Standard Penetration Test (SPT) blow counts for correlation with empirical data. Relatively undisturbed samples of Bay Mud were also obtained from the borings at selected depths by hydraulically pushing a 3-inch inside diameter, 30-inch long thin walled Shelby Tube Sampler. Disturbed samples were also obtained at selected depths by driving a 1.375-inch inside diameter (2-inch outside diameter) SPT sampler, without liners or rings, using a 140-pound hammer dropping approximately 30 inches. The sampler was driven 12 to 18 inches, the blows to drive each 6-inch increment were recorded, and the blows required to drive the final 12 inches, or portion thereof, are provided on the boring logs. A Disturbed “bulk” sample of the anticipated subgrade soils was also obtained from boring B-2 and placed in a bucket.

The logs of the borings showing the materials encountered, groundwater conditions, converted blow counts and sample depths are presented on Plates 3 through 6. The soils are described in accordance with the Unified Soil Classification System, outlined on Plate 7. Bedrock is described in accordance with Engineering Geology Rock Terms, shown on Plate 8. An idealized cross section of the conditions encountered in our borings is given on Plate 9.

The boring logs show our interpretation of subsurface soil and bedrock conditions on the date and at the locations indicated. Subsurface conditions may vary at other locations and times. Our interpretation is based on visual inspection of soil and bedrock samples, laboratory test results, and interpretation of drilling and sampling resistance. The location of the soil and bedrock boundaries should be considered approximate. The transition between soil and bedrock types may be gradual.

Laboratory Testing

The samples obtained from the borings were transported to our office and re-examined to verify soil classifications, evaluate characteristics and assign tests pertinent to our analysis. Selected samples were laboratory tested to determine their water content, dry density, classification (Atterberg Limits, percent of silt and clay), shear strength, consolidation characteristics, expansion potential (Expansion Index - EI) and R-value. Results of water content, dry density, classification, shear strength and EI tests are referenced on the boring logs. Results of the classification, triaxial strength, consolidation and R-value tests are presented on Plates 10 through 16.

SITE CONDITIONS

General

Marin County is located within the California Coast Range geomorphic province. This province is a geologically complex and seismically active region characterized by sub-parallel northwest-trending faults, mountain ranges and valleys. The oldest bedrock units are the Jurassic-Cretaceous Franciscan Complex and Great Valley sequence sediments originally deposited in a marine environment. Subsequently, younger rocks such as the Tertiary-age Sonoma Volcanics group, the Plio-Pleistocene-age Clear Lake Volcanics and sedimentary rocks such as the Guinda, Domengine, Petaluma, Wilson Grove, Cache, Huichica and Glen Ellen formations were deposited throughout the province. Extensive folding and thrust faulting during late Cretaceous through early Tertiary geologic time created complex geologic conditions that underlie the highly varied topography of today. In valleys, the bedrock is covered by thick alluvial soils.

Geology

Published geologic maps (Blake, et al., 2000) indicate the property is underlain by Quaternary age artificial fill over marine and marsh deposits (Qmf).

Landslides

Published landslide maps (Rice, et al., 1976) do not indicate large-scale slope instability at the site, and we did not observe active landslides at the site during our study.

Surface

Dunphy Park includes a large grass area, scattered mature trees, a sand volleyball court and bocce courts. There is a gravel parking lot along the northern edge of the park that also extends along a portion of the easterly edge. An unpaved overflow lot/access roadway exists along the western edge of the Park in former railroad right-of-way. An existing structure is located at Cass Gidley where the parking lot transitions from west to north. At this corner, a spit extends out into the Bay for a short distance. The Cass/Gidley office building is located on the spit. An asphalt paved bike lane extends along the southerly edge of the park, parallel to Bridgeway. Natural drainage consists of sheet flow over the ground surface that concentrates in man-made surface drainage elements such as roadside ditches, canals and gutters and natural drainage elements such as swales, ravines and Richardson Bay.

Subsurface

Our borings and laboratory tests indicate that the portion of the site we studied is blanketed by 5½ to 12 feet of heterogeneous fill. Heterogeneous fill is a material with varying density, strength, compressibility and shrink-swell characteristics that often has an unknown origin and placement history. The heterogeneous fill we encountered consists of sand with varying amounts of clay and silt and clay with varying amounts of sand and gravel. These soils exhibit low to medium plasticity (LL = 38-48; PI = 19-26) and moderate expansion potential (EI = 56-60). The heterogeneous fill is underlain by soft silt and clay, referred to locally as Bay Mud, with occasional interbedded layers of gravel or sand. Sandstone bedrock extends from beneath the Bay Mud materials to the maximum depths explored (63½ feet). The sandstone is generally closely fractured, firm to moderately hard, plastic to moderately strong and moderately to highly weathered. A detailed description of subsurface conditions found in our borings is given on Plates 3 through 6, Appendix A. An idealized cross section of the subsurface conditions is presented on Plate 9. Based on Table 20.3-1 of American Society of Civil Engineers (ASCE) Standard 7-10, titled “Minimum Design Loads for Buildings and Other Structures” (2010), we have determined a Site Class of E should be used for the site.

Corrosion Potential

Mapping by the Natural Resources Conservation Service (2015) provides no data for the corrosion potential of the near surface soil for uncoated steel and concrete. Performing corrosivity tests was not part of our requested and/or proposed scope of work. Should the need arise, we would be pleased to provide a proposal to evaluate these characteristics.

Groundwater

We were unable to access the depth to groundwater from our borings because they were drilled using rotary wash techniques. Rotary wash drilling involves circulating fluid as part of the process so it is not possible to measure the groundwater level. Based on our experience, the groundwater level within the park area is likely tidally controlled.

DISCUSSION AND CONCLUSIONS

Seismic Hazards

Seismicity

Data presented by the Working Group on California Earthquake Probabilities (2007) estimates the chance of one or more large earthquakes (Magnitude 6.7 or greater) in the San Francisco Bay region within the next 30 years to be approximately 63 percent. Therefore, future seismic shaking should be anticipated at the site. It will be necessary to design and construct proposed structures in strict adherence with current standards for earthquake-resistant construction.

Faulting

We did not observe landforms within the area that would indicate the presence of active faults and the site is not within a current Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Therefore, we believe the risk of fault rupture at the site is low. However, the site is within an area affected by strong seismic activity. Several northwest-trending Earthquake Fault Zones exist in close proximity to and within several miles of the site (Bortugno, 1982). The shortest distances from the site to the mapped surface expression of these faults are presented in the table below.

ACTIVE FAULT PROXIMITY		
Fault	**Direction**	**Distance-Miles**
San Andreas	WSW	7
Hayward	NE	11

Liquefaction

Liquefaction is a rapid loss of shear strength experienced in saturated, predominantly granular soils below the groundwater level during strong earthquake ground shaking due to an increase in pore water pressure. The occurrence of this phenomenon is dependent on many complex factors including the intensity and duration of ground shaking, particle size distribution and density of the soil.

Granular soils were encountered in our borings for this project. The granular soils we observed included sand and gravel with varying amounts of clay and silt within the heterogeneous fill materials encountered above the Bay Mud. We also encountered layers of native sand and gravel, also with varying amounts of clay and silt, within the Bay Mud. In general, the sand and gravel within the heterogeneous fill is not consistently saturated as the groundwater level fluctuates with the changes in tide. Therefore, the heterogeneous fill was excluded from our liquefaction analysis. The seismic impacts on the heterogeneous fill will be covered in the "Densification" section.

As discussed above, sand and gravel layers were encountered within the Bay Mud at the project site. It is not unusual to see sand and gravel layers within Bay Mud. These layers are, by the nature of how they are deposited, typically looser and susceptible to liquefaction. These layers are also typically thin and discontinuous with occasional thicker deposits. For the Dunphy Park site the layers seem to be thicker and extend laterally between the three borings we drilled. Standard Penetration Test (SPT) blow counts from soil borings are used to evaluate liquefaction potential and its impacts. This analysis will be discussed in more details below. We do not have SPT blow counts for all sand and gravel layers encountered because some of these layers were sampled using other techniques. Therefore the analysis described herein focuses on the layers where we have SPT data with the results then applied to the layers for which we do not.

We performed an analysis of the blow count data from our borings using the methods of Seed and Idriss (1982), Seed and others (1985), Youd and Idriss (2001), Idriss and Boulanger (2004) and Idriss and Boulanger (2008). These procedures normalize the blow counts to account for overburden pressure, rod length, hammer energy, and fines (percent of silt and clay) content. Once the blow counts are normalized and adjusted to a clean sand blow count, the cyclic resistance ratio (CRR) for each blow count is then determined using the same procedures referenced above. The CRR is compared to the cyclic stress ratio (CSR) induced by the earthquake. Calculating the CSR requires a peak ground acceleration and design earthquake magnitude.

Peak ground acceleration (PGA) was determined using the methods in the 2013 California Building Code (CBC) and the American Society of Civil Engineers (ASCE) Standard 7-10, titled "Minimum Design Loads for Buildings and Other Structures" (2010). Using the U.S. Seismic Design Maps from the United States Geological Survey (USGS) website (http://geohazards.usgs.gov/designmaps/us/application.php), the site's latitude and longitude of 37.8615°N and 122.4887°W, respectively, and a site soil Class of E, the PGA for the site is 0.56g. Using this information, the CSR for a M_M 7.5 earthquake at the site ranges from 0.56 to 0.62. The San Andreas fault is most likely controlling the ground motions at the site. According to Petersen (1996), the nearby portion of the San Andreas fault is capable of a M_M 7.9 earthquake. Therefore, the CRR values at the site must be scaled to account for the difference between M_M 7.9 and M_M 7.5. When the scaling factor for magnitude and confining stress corrections presented in Idriss and Boulanger (2004) are applied, the CRR values at the site do not exceed the CSR values. Therefore, we judge that the potential for liquefaction of these sand and gravel layers is high.

There are three potential consequences of liquefaction: bearing capacity failure, lateral spreading toward a free face and settlement. Bearing capacity failure is sudden and extreme settlement of foundations that typically occurs when the liquefied layer is relatively close (typically within two times the footing width, depending on the loads) to the bottom of the foundation. Because the liquefiable layers are within the Bay Mud and foundations will be within the upper portion of the fill, we judge that the potential for bearing capacity failure is low.

Lateral spreading can occur where continuous layers of liquefiable soil extend to a free face. The potentially liquefiable layers at the site are continuous, at least within the project site. However, the continuous layers are located within the Bay Mud and within the project site do not extend to a free face. It is possible that farther from the shoreline there is a free face created due to dredging. Therefore, we cannot preclude the possibility of earthquake-lateral spreading into the Bay.

The third potential consequence of liquefaction is settlement due to densification of the liquefied soils. Potential settlements based on the blow count data and cyclic stress ratios were calculated using the methods of Ishihara and Yoshimine (1992). For the layers encountered in our borings we calculated total settlement ranging from 3 to 6 inches. Because liquefaction settlement is typically erratic it is difficult to estimate differential settlement. History suggests that differential settlement can occur over relatively short distances.

Densification

Densification is the settlement of loose, granular soils above the groundwater level due to earthquake shaking. Typically, heterogeneous fill and granular soils that would be susceptible to liquefaction, if saturated, are susceptible to densification if not saturated. As discussed in the "Liquefaction" section, sand and gravel with varying amounts of clay and silt were encountered within the heterogeneous fill. Based on the density of these soils, we judge that there is a moderate potential for densification to impact structures at the site. Given the amount of fine grained material observed in the fill and the increased density of the upper portions of the fill, we estimate that densification settlement will be up to about 1 inch.

Tsunamis and Seiches

Maps published by the California Emergency Management Agency, in conjunction with the California Geological Survey and the University of California (California Emergency Management Agency, 2009), indicates the site is located a tsunamis inundation zone. Site-specific evaluation of tsunamis and seiche impact to the project is beyond the scope of this study.

Geotechnical Issues

General

Based on our study, we judge the proposed improvements can be built as planned, provided the recommendations presented in this report are incorporated into their design and construction. The primary geotechnical concerns during design and construction of the project are:

1. The presence of soft sediments referred to locally as Bay Mud;
2. The presence of soils susceptible to liquefaction and densification;
3. The presence of heterogeneous fill;
4. The detrimental effects of uncontrolled surface runoff; and
5. The strong ground shaking predicted to impact the site during the life of the project.

Soft Bay Sediments

The soft bay sediments encountered at the site are referred to locally as Bay Mud. Improvements constructed on sites underlain by Bay Mud are highly susceptible to settlement. In particular, fills placed over Bay Mud will settle significantly as well as structures constructed on the fill. The Dunphy Park site, as with most of the margin of Richardson Bay and San Francisco Bay, consists of fill placed over the Bay Mud. In order to assess the settlement for new improvements, one must estimate the settlement that occurred due to the original fill placement, assess how much settlement remains from the original fill placement and calculate the settlement due to the new improvements. These analyses are discussed in the subsequent sections.

Settlement to Date and Remaining Settlement - In order to estimate the settlement that has occurred to date from the fill placed at the site and the settlement remaining, we need the following information: the time frame of fill placement, the thickness of fill, the thickness of the sediment and the time rate of consolidation of the sediment. The fill in this area of Sausalito was reportedly placed in the 1960's. For our analysis, we assumed the fill has been there for 50 years or since about 1965. The thickness of the fill varies at the Dunphy site with fill being about 6 feet thick in most areas and thicker (10 to 12 feet) adjacent to Bridgeway. Because the planned improvements are in the area of the thinner fill section, we used 6 feet of fill for our analysis. The thickness of Bay Mud at the site is a little more difficult to determine for a general analysis. As can be seen from the cross section presented on Plate 9, the Bay Mud thickness is a combination of layers interbedded with layers of sand and gravel. For example, the total Bay Mud thickness in boring B-3 is about 8 feet below the fill and another 17 feet below the sand and gravel layers for a total thickness of 25 feet. In boring B-1, the thickness is a combination of about 11 feet below the fill, 5 feet interbedded within the sand layer, and 22 feet below the sand layer for a total thickness of 38 feet. This variability of thickness combined with depth makes the analysis more complicated. The final piece of our analysis is the time rate of consolidation, which comes from the consolidation tests we performed on samples of the Bay Mud.

Using the above information, the first step in our analysis was to estimate the amount of settlement that has occurred due to the placement of the original fill. First, using the engineering characteristics from our consolidation tests presented on Plates 13 through 15 we calculated the anticipated settlement from the fill. In particular we looked at the conditions in borings B-1 and B-4, which represent the area of the planned improvements. Settlement calculations using the conditions in each of those borings yield about 9 and 5 inches of settlement for borings B-1 and B-4, respectively. If we assume that the Bay Mud does not include sand and gravel layers, the estimated settlement due to the fill increases to about 11½ inches in boring B-1 and 8½ inches in boring B-4.

The second step in this process is to estimate how much settlement has occurred to date due to the fill, and thus how much settlement remains. As presented above, we have assumed the fill has been in place for 50 years. The time rate of consolidation from our laboratory tests indicates a rate of 0.04 square feet per day. The Bay Mud is in what is referred to as a double drainage condition because the soil layers above and below the Bay Mud are granular in nature. This means that as the water is squeezed out of the Bay Mud it can travel vertically in both directions. This is important because it essentially doubles the rate of consolidation. Using the layers that we encountered in our borings, we calculated that over 99% of the settlement due to the fill has already occurred, which means that the settlement remaining due to the fill is considered negligible. If the sand and gravel layers are thinner than those encountered in our borings, this percentage reduces to about 94%, which means about 6% of the settlement from the fill is remaining. Using the total settlements from above for this condition, the estimated remaining settlement is less than ¾ inch for boring B-1 and about ½ inch for boring B-4.

Settlement Due to New Improvements - The settlement due to new improvements includes fill placed to create the amphitheater and loads from the foundations for the bathroom and other structural improvements. These settlements are calculated for the current condition with the settlement added to the remaining settlement, if any. For a new fill condition, we calculated settlement of 1½ inches per foot of new fill for the existing conditions in boring B-1 and ½ inch per foot of fill for the existing conditions in boring B-4. When we assume the sand and gravel layers are not present, these values increase to 2 and 1 inches per foot of new fill respectively.

It appears that no new fill will be placed in the areas of planned structural improvements. Therefore, settlement is calculated by adding the loading from foundations to the current condition. The load related to foundations depends on the bearing pressure, the type of footing and the width of the foundation. For example, a strip footing foundation with a bearing pressure of 2250 pounds per square foot (psf) and a footing width of 24 inches yields settlement of less than 1 inch. However, increase the width of the footing to 36 inches and the settlement increases to about 1¼ inches. We calculate settlement for various loading conditions with an emphasis on total settlement being less than 1 inch and found that the ideal bearing pressure is about 1500 psf.

Summary - In summary, it is likely that the remaining settlement from the fill placed in the 1960's is negligible at the project site. For areas along the shoreline where the sand and gravel may be thinner, the settlement remaining is likely less than 1 inch. Therefore, settlements from planned improvements need to be calculated based on the loading condition of the planned improvement. For new fill placed at the site, we recommend you estimate 1 to 2 inches of settlement per foot of new fill added, which represents a range of Bay Mud thickness of 40 to 50 feet. Finally, at a bearing pressure of 1500 psf, a structure without new fill should experience less than 1 inch of total settlement. Based on the above settlement conditions, planned structures need to be designed for 1 inch of differential settlement across the building.

Soils Susceptible to Liquefaction

As discussed previously, sand layers within the soft sediments are susceptible to liquefaction. These layers are thicker than we usually see in the soft sediments and are continuous between the three deeper borings that we drilled for this project. Although a free face is not immediately present, it is possible that these layers may extend to where the channel deepens and thus make the project site susceptible to lateral spreading. The more likely result of liquefaction at the site is settlement related to the densification of the layers due to the seismic shaking. As discussed, we calculated settlements ranging from 3 to 6 inches for the layers encountered in our borings. The risk of lateral spreading and settlement impacting the planned improvements can be reduced by supporting structures on deep foundations that gain support below the susceptible layers or by improving these layers by techniques such as vibro-replacement (stone columns) and deep soil mixing. However, given that the planned improvements include a bathroom, grading to create an amphitheater and parking as well as possible improvements to Cass/Gidley and the Cruising Club, these mitigations do not seem cost effective. Therefore, the risk of liquefaction and its consequences must be accepted by the City of Sausalito and those planning improvements at Cass/Gidley and the Cruising Club. We can provide more detailed information and recommendations regarding deep foundations and ground improvement if requested.

Heterogeneous Fill

Heterogeneous fills of unknown quality and unknown method of placement, such as those found at the site, can settle and/or heave erratically under the load of new fills, structures, slabs, and pavements. Footings, slabs, and pavements supported on heterogeneous fill could also crack as a result of such erratic movements. The detrimental effects of such movements can be reduced by strengthening the soils during grading. This can be achieved on this site by excavating the heterogeneous fill to a depth of 24 inches below existing grade or finished pad grade, whichever is deeper, and replacing it as properly compacted engineered fill.

Foundation, Slab and Pavement Support - After remedial grading, satisfactory foundation support can be obtained from spread footings bottomed on the engineered fill. Interior slab-on-grade floors, exterior slabs and pavements can also be satisfactorily supported on the engineered fill.

On-Site Soil Quality

All fill materials used in the building area and the upper 12 inches of exterior slab and pavement subgrade must consist of on-site soils or be imported select fill, as subsequently described in "Recommendations." We anticipate that, with the exception of organic matter and of rocks or lumps larger than 6 inches in diameter, the excavated material will be suitable for re-use as fill on the project.

RECOMMENDATIONS

General

As discussed previously, the planned improvements are underlain by heterogeneous fill over soft sediments, referred to locally as Bay Mud, that include significant layers of sand and gravel that are susceptible to liquefaction. The planned improvements will be susceptible to settlement related to the Bay Mud and from liquefaction. In addition, the area may be susceptible to earthquake-induced lateral spreading. The risks associated with these conditions must be accepted by the City of Sausalito and other property owners planning improvements in the area.

Seismic Design

Seismic design parameters presented below are based on Section 1613 titled "Earthquake Loads" of the 2013 California Building Code (CBC). Based on Table 20.3-1 of American Society of Civil Engineers (ASCE) Standard 7-10, titled "Minimum Design Loads for Buildings and Other Structures" (2010), we have determined a Site Class of E should be used for the site. Using a site latitude and longitude of 37.8615°N and 122.4887°W, respectively, and the U.S. Seismic Design Maps from the United States Geological Survey (USGS) website (http://geohazards.usgs.gov/designmaps/us/application.php), we recommend that the following seismic design criteria be used for structures at the site.

2013 CBC Seismic Criteria	
Spectral Response Parameter	**Acceleration (g)**
S_S (0.2 second period)	1.500
S_1 (1 second period)	0.630
S_{MS} (0.2 second period)	1.350
S_{M1} (1 second period)	1.513
S_{DS} (0.2 second period)	0.900
S_{D1} (1 second period)	1.009

Grading

Site Preparation

Areas to be developed should be cleared of vegetation and debris. Trees and shrubs that will not be part of the proposed development should be removed and their primary root systems grubbed. Cleared and grubbed material should be removed from the site and disposed of in accordance with County Health Department guidelines. We did not observe septic tanks, leach lines or underground fuel tanks during our study. Any such appurtenances found during grading should be capped and sealed and/or excavated and removed from the site, respectively, in accordance with established guidelines and requirements of the County Health Department. Voids created during clearing should be backfilled with engineered fill as recommended herein.

Stripping

Areas to be graded should be stripped of the upper few inches of soil containing organic matter. Soil containing more than two percent by weight of organic matter should be considered organic. Actual stripping depth should be determined by a representative of the geotechnical engineer in the field at the time of stripping. The strippings should be removed from the site, or if suitable, stockpiled for re-use as topsoil in landscaping.

Excavations

Following initial site preparation, excavation should be performed as planned or recommended herein. Excavations extending below the proposed finished grade should be backfilled with suitable materials compacted to the requirements given below.

Within fill and interior slab-on-grade areas, the old fill should be excavated to a depth of 24 inches below existing grade or finished pad grade, whichever is deeper. The excavation of old fill should also extend at least 12 inches below exterior slab and pavement subgrade. The excavation of old fill should extend at least 5 feet beyond the outside edge of the exterior footings of the proposed buildings and 3 feet beyond the edge of exterior slabs and pavements and the toe of new fills. The excavated materials should be stockpiled for later use as compacted fill, or removed from the site, as applicable.

At all times, temporary construction excavations should conform to the regulations of the State of California, Department of Industrial Relations, Division of Industrial Safety or other stricter governing regulations. The stability of temporary cut slopes, such as those constructed during the installation of underground utilities, should be the responsibility of the contractor. Depending on the time of year when grading is performed, and the surface conditions exposed, temporary cut slopes may need to be excavated to 1½:1, or flatter. The tops of the temporary cut slopes should be rounded back to 2:1 in weak soil zones.

Fill Quality

All fill materials should be free of perishable matter and rocks or lumps over 6 inches in diameter and must be approved by the geotechnical engineer prior to use. Fill beneath and within 5 feet of the building areas and the upper 12 inches of fill beneath and within 3 feet of exterior slabs and/or pavement edges should be select fill. We judge the on-site soils are generally suitable for use as general and select fill. The suitability of the on-site soils for use as select fill should be verified during grading.

Import Select Fill

Import select fill should be free of organic matter, have a low expansion potential, and conform in general to the following requirements:

SIEVE SIZE	PERCENT PASSING (by dry weight)
6 inch	100
4 inch	90 – 100
No. 200	10 – 60

Liquid Limit – 40 Percent Maximum
Plasticity Index – 15 Percent Maximum
R-value – 10 Minimum (pavement areas only)

Material not conforming to these requirements may be suitable for use as import fill; however, it shall be the contractor’s responsibility to demonstrate that the proposed material will perform in an equivalent manner. The geotechnical engineer should approve imported materials prior to use as compacted fill. The grading contractor is responsible for submitting, at least 72 hours (3 days) in advance of its intended use, samples of the proposed import materials for laboratory testing and approval by the soils engineer.

Fill Placement

The surface exposed by stripping and removal of heterogeneous fill should be scarified to a depth of at least 6 inches, uniformly moisture-conditioned to at least 2 percent above optimum and compacted to at least 90 percent of the maximum dry density of the materials as determined by ASTM Test Method D-1557. Approved fill material should then be spread in thin lifts, uniformly moisture-conditioned to at least 2 percent above optimum and properly compacted. All structural fills, including those placed to establish site surface drainage, should be compacted to at least 90 percent relative compaction. Only approved select materials should be used for fill within building areas and within the upper 12 inches of exterior slabs and pavement subgrades.

SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendation (ASTM D-1557)
Preparation for areas to receive fill	After preparation in accordance with this report, compact upper 6 inches to a minimum of 90 percent relative compaction.
General fill (native or import)	Compact to a minimum of 90 percent relative compaction.
Structural fill beneath buildings, extending outward to 5' beyond building perimeter	Compact to a minimum of 90 percent relative compaction.
Trenches	Compact to a minimum of 90 percent relative compaction. Compact the top 6 inches below vehicle pavement subgrade to a minimum of 95 percent relative compaction.
Pavements, extending outward to 3' beyond edge of pavement	Compact upper 6 inches of subgrade to a minimum of 95 percent relative compaction.
Concrete flatwork and exterior slabs, extending outward to 3' beyond edge of slab	Compact subgrade to a minimum of 90 percent relative compaction. Where subject to vehicle traffic, compact upper 6 inches of subgrade to at least 95 percent relative compaction.
Aggregate Base	Compact aggregate base to at least 95 percent relative compaction.

Wet Weather Grading

Generally, grading is performed more economically during the summer months when on-site soils are usually dry of optimum moisture content. Delays should be anticipated in site grading performed during the rainy season or early spring due to excessive moisture in on-site soils. Special and relatively expensive construction procedures, including dewatering of excavations and importing granular soils, should be anticipated if grading must be completed during the winter and early spring or if localized areas of soft saturated soils are found during grading in the summer and fall.

Open excavations also tend to be more unstable during wet weather as groundwater seeps towards the exposed cut slope. Severe sloughing and occasional slope failures should be anticipated. The occurrence of these events will require extensive clean up and the installation of slope protection measures, thus delaying projects. The general contractor is responsible for the performance, maintenance and repair of temporary cut slopes.

Foundation Support

Provided the heterogeneous fill is strengthened by remedial grading as recommended herein, proposed structures and structural improvements can be supported on continuous and isolated spread footings that bottom on select engineered fill.

Spread Footings

Spread footings should be at least 12 inches wide and should bottom on select engineered fill at least 12 inches below pad subgrade. Additional embedment or width may be needed to satisfy code and/or structural requirements. Because of the potential for uneven soil support, continuous footings should have sufficient reinforcement to span, as a simple beam, an unsupported distance of approximately 10 feet. In addition, the foundation system should be designed to withstand 1-inch of differential settlement across planned structures.

The bottoms of all footing excavations should be thoroughly cleaned out or wetted and compacted using hand-operated tamping equipment prior to placing steel and concrete. This will remove the soils disturbed during footing excavations, or restore their adequate bearing capacity, and reduce post-construction settlements. Footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in soils exposed in the footing excavations, the soil should be thoroughly moistened to close all cracks prior to concrete placement. The moisture condition of the foundation excavations should be checked by the geotechnical engineer no more than 24 hours prior to placing concrete.

Bearing Pressures - Footings installed in accordance with these recommendations may be designed using allowable bearing pressures of 1000, 1500 and 2000 pounds per square foot (psf), for dead loads, dead plus code live loads, and total loads (including wind and seismic), respectively.

Lateral Pressures - The portion of spread footing foundations extending into select engineered fill may impose a passive equivalent fluid pressure and a friction factor of 350 pcf and 0.35, respectively, to resist sliding. Passive pressure should be neglected within the upper 6 inches, unless the soils are confined by concrete slabs or pavements.

Slab-On-Grade

Provided grading is performed in accordance with the recommendations presented herein, interior and exterior slabs should be underlain by select engineered fill. Slab-on-grade subgrade should be rolled to produce a dense, uniform surface. The future expansion potential of the subgrade soils should be reduced by thoroughly presoaking the slab subgrade prior to concrete placement. The moisture condition of the subgrade soils should be checked by the geotechnical engineer no more than 24 hours prior to placing the capillary moisture break. The slabs should be underlain with a capillary moisture break consisting of at least 4 inches of clean, free-draining crushed rock or gravel (excluding pea gravel) at least ¼-inch and no larger than ¾-inch in size. Class 2 aggregate base can be used for slab rock under exterior slabs.

Slabs should be designed by the project civil or structural engineer to support the anticipated loads, reduce cracking and provide protection against the infiltration of moisture vapor. A vapor barrier should be placed under all slabs-on-grade that are likely to receive an impermeable floor finish or be used for any purpose where the passage of water vapor through the floor is undesirable. RGH does not practice in the field of moisture vapor transmission evaluation or mitigation. Therefore, we recommend that a qualified person be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. This person should provide recommendations for mitigation of the potential adverse impact of moisture vapor transmission on various components of the structure as deemed appropriate.

Utility Trenches

The shoring and safety of trench excavations is solely the responsibility of the contractor. Attention is drawn to the State of California Safety Orders dealing with “Excavations and Trenches.”

Unless otherwise specified by the City of Sausalito, on-site, inorganic soil may be used as general utility trench backfill. Where utility trenches support pavements, slabs and foundations, trench backfill should consist of aggregate baserock. The baserock should comply with the minimum requirements in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base. Trench backfill should be moisture-conditioned as necessary, and placed in horizontal layers not exceeding 8 inches in thickness, before compaction. Each layer should be compacted to at least 90 percent relative compaction as determined by ASTM Test Method D-1557. The top 6 inches of trench backfill below vehicle pavement subgrades should be moisture-conditioned as necessary and compacted to at least 95 percent relative compaction. Jetting or ponding of trench backfill to aid in achieving the recommended degree of compaction should not be attempted.

Pavements

An R-value of 12 was measured on a bulk sample of near-surface soil obtained in the planned parking lot. Because of potential variation in the on-site soils, we selected an R-value of 10 for use in pavement design calculations. Based on the selected R-value, we have computed pavement sections for Traffic Indices (TI) ranging from 5.0 to 7.0 in the table below. The project engineer, in consultation with City officials, should choose the pertinent (TI) for this project.

PAVEMENT SECTIONS		
TI	ASPHALT CONCRETE (feet)	CLASS 2 AGGREGATE BASE (feet)
7.0	0.35	1.15
6.0	0.25	1.05
5.0	0.20	0.85

* If required

Pavement thicknesses were computed using Caltrans CalFP v1.1 design software and are based on a pavement life of 20 years. These recommendations are intended to provide support for the traffic represented by the indicated Traffic Indices. They are not intended to provide pavement sections for heavy concentrated construction storage or wheel loads such as forklifts, parked truck-trailers and concrete trucks or for post-construction concentrated wheel loads such as self-loading dumpster trucks.

In areas where heavy construction storage and wheel loads are anticipated, the pavements should be designed to support these loads. Support could be provided by increasing pavement sections or by providing reinforced concrete slabs. Alternatively, paving can be deferred until heavy construction storage and wheel loads are no longer present. Loading areas for self-loading dumpster trucks should be provided with reinforced concrete slabs at least 6 inches thick, and reinforced with No. 4 bars at 12-inch centers each way. Alternatively, the asphalt concrete section should be increased to at least 8 inches in these areas.

Prior to placement of aggregate base, the upper 6 inches of the pavement subgrade soils should be scarified, uniformly moisture-conditioned to near optimum, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface. Aggregate base materials should be spread in thin layers, uniformly moisture-conditioned, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface. The materials and methods used should conform to the requirements of the City of Sausalito and the current edition of the Caltrans Standard Specifications, except that compaction requirements should be based on ASTM Test Method D-1557. Aggregate used for the base course should comply with the minimum requirements specified in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base.

Parking Lot Drainage

Water tends to migrate under pavements and collect in the aggregate courses at low areas on parking lot subgrade soils, such as around storm drain inlets and the thread of paved swales leading to inlets. The ponded water will soften subgrade soils and, under repetitive heavy-wheel loads, will induce inordinately high stresses on the subgrade and pavement components that could result in untimely maintenance. Under-pavement drainage can be improved and maintenance reduced by replacing a 12-inch wide strip (extending at least 15 feet on either side of the inlet) of the select subbase layer or subgrade soils with a subdrain consisting of ¾-inch or 1½-inch free-draining Class 1 Permeable Material. The drain rock should be outletted into the storm drain inlet. Storm drain trenches can be made to serve as pavement subdrains. We should be consulted to verify the suitability of storm drain trenches as pavement subdrains in a case-specific basis.

Where pavements will abut landscaped areas, the pavement baserock layer and subgrade soils should be protected against saturation from irrigation and rainwater with a subdrain, similar to that previously discussed. The subdrain should extend to a depth of at least 6 inches below the bottom of the baserock layer. Alternatively, a grouted moisture cut-off that extends 12 inches below the bottom of the baserock layer should be provided below or immediately behind the curb and gutter.

Wet Weather Paving

In general, the pavements should be constructed during the dry season to avoid the saturation of the subgrade and base materials, which often occurs during the wet winter months. If pavements are constructed during the winter, a cost increase relative to drier weather construction should be anticipated. Unstable areas may have to be overexcavated to remove soft soils. The excavations will probably require backfilling with imported crushed (ballast) rock. The geotechnical engineer should be consulted for recommendations at the time of construction.

Geotechnical Drainage

Surface water should be diverted away from slopes, foundations and edges of pavements. Surface drainage gradients should slope away from building foundations in accordance with the requirements of the CBC or local governing agency. Where a gradient flatter than 2 percent for paved areas and 4 percent for unpaved areas is required to satisfy design constraints, area drains should be installed with spacing no greater than about 20 feet. Roofs should be provided with gutters and the downspouts should be connected to closed (glued Schedule 40 PVC or ABS with SDR of 35 or better) conduits discharging well away from foundations, onto paved areas or into the site's surface drainage system.

Water seepage or the spread of extensive root systems into the soil subgrade of footings, slabs or pavements could cause differential movements and consequent distress in these structural elements. Landscaping should be planned with consideration for these potential problems.

Maintenance

Periodic land maintenance will be required. Surface and subsurface drainage facilities should be checked frequently, and cleaned and maintained as necessary or at least annually. A dense growth of deep-rooted ground cover must be maintained on all slopes to reduce sloughing and erosion. Sloughing and erosion that occurs must be repaired promptly before it can enlarge.

Supplemental Services

Pre-Bid Meeting

It has been our experience that contractors bidding on the project often contact us to discuss the geotechnical aspects. Informal contacts between RGH and an individual contractor could result in incomplete or misinterpreted information being provided to the contractor. Therefore, we recommend a pre-bid meeting be held to answer any questions about the report prior to submittal of bids. If this is not possible, questions or clarifications regarding this report should be directed to the project owner or their designated representative. After consultation with RGH, the project owner or their representative should provide clarifications or additional information to all contractors bidding the job.

Plan and Specifications Review

Coordination between the design team and the geotechnical engineer is recommended to assure that the design is compatible with the soil, geologic and groundwater conditions encountered during our study. RGH Consultants (RGH) recommends that we be retained to review the project plans and specifications to determine if they are consistent with our recommendations. In the event we are not retained to perform this recommended review, we will assume no responsibility for misinterpretation of our recommendations.

Construction Observation and Testing

Prior to construction, a meeting should be held at the site that includes, but is not limited to, the owner or owner's representative, the general contractor, the grading contractor, the foundation contractor, the underground contractor, any specialty contractors, the project civil engineer, other members of the project design team and RGH. This meeting should serve as a time to discuss and answer questions regarding the recommendations presented herein and to establish the coordination procedure between the contractors and RGH.

In addition, we should be retained to monitor all soils related work during construction, including:

- Site stripping, over-excavation, grading, and compaction of near surface soils;
- Placement of all engineered fill and trench backfill with verification field and laboratory testing;
- Observation of all foundation excavations; and
- Observation of foundation and subdrain installations.

If, during construction, we observe subsurface conditions different from those encountered during the explorations, we should be allowed to amend our recommendations accordingly. If different conditions are observed by others, or appear to be present beneath excavations, RGH should be advised at once so that these conditions may be evaluated and our recommendations reviewed and updated, if warranted. The validity of recommendations made in this report is contingent upon our being notified and retained to review the changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at, or adjacent to, the site, the recommendations made in this report may no longer be valid or appropriate. In such case, we recommend that we be retained to review this report and verify the applicability of the conclusions and recommendations or modify the same considering the time lapsed or changed conditions. The validity of recommendations made in this report is contingent upon such review.

These supplemental services are performed on an as-requested basis and are in addition to this geotechnical study. We cannot accept responsibility for items that we are not notified to observe or for changed conditions we are not allowed to review.

LIMITATIONS

This report has been prepared by RGH for the exclusive use of the City of Sausalito and their consultants as an aid in the design and construction of the proposed Dunphy Park improvements described in this report.

The validity of the recommendations contained in this report depends upon an adequate testing and monitoring program during the construction phase. Unless the construction monitoring and testing program is provided by our firm, we will not be held responsible for compliance with design recommendations presented in this report and other addendum submitted as part of this report.

Our services consist of professional opinions and conclusions developed in accordance with generally accepted geotechnical engineering principles and practices. We provide no warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided to us regarding the proposed construction, the results of our field exploration, laboratory testing program, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

The borings represent subsurface conditions at the locations and on the date indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration on March 12 and 13, 2015, and may not necessarily be the same or comparable at other times.

The scope of our services did not include an environmental assessment or a study of the presence or absence of toxic mold and/or hazardous, toxic or corrosive materials in the soil, surface water, groundwater or air (on, below or around this site), nor did it include an evaluation or study for the presence or absence of wetlands. These studies should be conducted under separate cover, scope and fee and should be provided by a qualified expert in those fields.

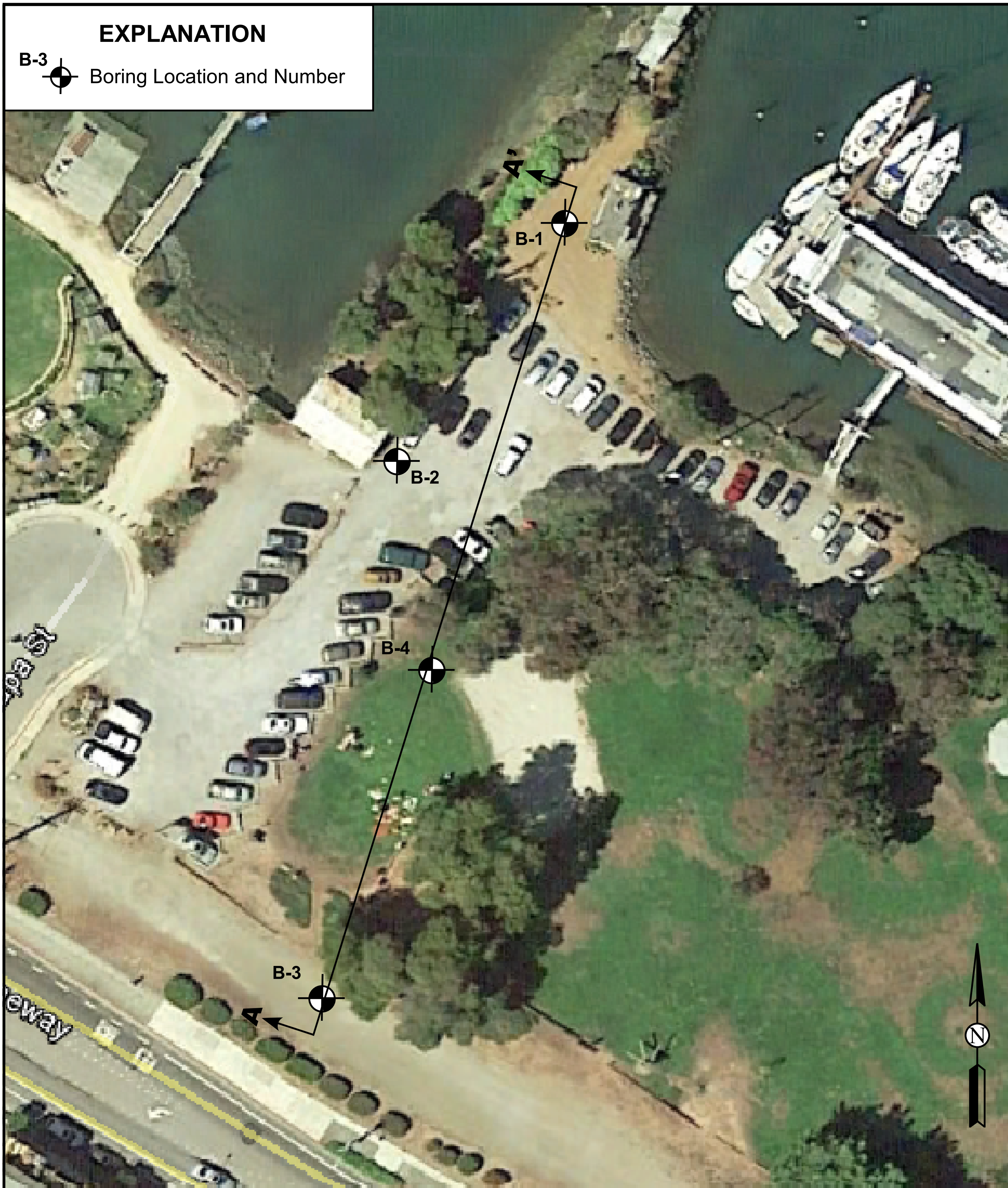
APPENDIX A - PLATES

LIST OF PLATES

Reference: Maptech Topoquad, San Francisco North, California Quadrangle

Scale: 1" = 2000'

RGH CONSULTANTS	**SITE LOCATION MAP** Dunphy Park Improvements Bridgeway Boulevard Sausalito, California	PLATE 1
Job No: 1993.39.04.1 — Date: JUNE 2015		



Reference: 2015 Google Earth

Scale: 1" = 50'

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Job No: 1993.39.04.1 | Date: JUNE 2015

EXPLORATION PLAN
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE
2

Date(s) Drilled **3/12/15**	Logged By **BPC/REP**	Checked By **EGC**
Drilling Method **Rotary Wash**	Drill Bit Size/Type **3-1/8 inch Drag Bit &Tricone Bit, Casing to 3 feet**	Total Depth of Borehole **63-1/2 feet**
Drill Rig Type **Fraste Multi Drill XL Track Mounted**	Drilling Contractor **Pitcher**	Approximate Surface Elevation **Existing Ground Surface**
Groundwater Level and Date Measured **N/A**	Sampling Method(s) **Modified California, SPT, Tube**	Hammer Data **140lb, 30" autotrip**

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance, blows/ft	Graphic Log	MATERIAL DESCRIPTION	Dry Density (pcf)	Water Content (%)	% <#200 Sieve	PI, %	LL, %	Expansion Index (EI)	UC, psf	REMARKS AND OTHER TESTS
	0				MOTTLED BROWN-RED SAND w/CLAY (SC). Medium dense, moist, fine sand. (FILL)								
			14		BROWN SANDY CLAY (CL). Stiff, moist, fine to coarse sand. (FILL)	112	12.4	32.4					
			3		GRAY-BROWN CLAYEY SAND W/GRAVEL (SC). Medium dense, moist, fine to coarse sand. (FILL)								
	5		1		MOTTLED GRAY & ORANGE CLAYEY SAND w/SILT & GRAVEL (SC). Very loose, wet, fine to coarse sand. (FILL)	82	41.2						
			50 psi to 9 feet		GRAY-RED SANDY CLAY w/SILT (CL). Very soft, wet, fine sand. (BAY MUD)								
					GRAY SILT (MH). Very soft, wet. (BAY MUD)								
	10												
			1										
	15												
					RED & GRAY SAND W/CLAY & GRAVEL (SP-SC). Medium dense, wet, coarse sand.								
			12					9.1					
	20												
					GRAY CLAYEY SILT (MH). Soft, wet. (BAY MUD)								
			4										
	25												
					GRAY CLAYEY SAND (SC). Loose, wet, medium to coarse sand.								
			400 psi to 29-1/2 feet,										
	30												
					GRAY SILT w/CLAY (MH). Soft to medium-stiff, wet. (BAY MUD)								
	35												

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Job No: 1993.39.04.1 | Date: JUNE 2015

LOG OF BORING B-1

Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE **3A**

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance, blows/ft	Graphic Log	MATERIAL DESCRIPTION	Dry Density (pcf)	Water Content (%)	% <#200 Sieve	PI, %	LL, %	Expansion Index (EI)	UC, psf	REMARKS AND OTHER TESTS
	35				GRAY SILT w/CLAY (MH). Soft to medium-stiff, wet. (BAY MUD)								
	40		o psi to 40 feet, 50 psi to 40-1/2										
	45												
			3		Occasional shells								
	50												
					BROWN SAND W/GRAVEL (SP). Dense, wet.								
	55												
			50/3"		LIGHT BROWN & RED BROWN SANDSTONE, Very closely spaced fractures, firm to moderately hard, weak, moderately to highly weathered.								
	60												
			50/4"										
					Boring Terminated at 63-1/2 feet								
	65												
	70												

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Job No: 1993.39.04.1 | Date: JUNE 2015

LOG OF BORING B-1
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE
3 B

Date(s) Drilled	3/12/15	Logged By	REP	Checked By	EGC
Drilling Method	Rotary Wash	Drill Bit Size/Type	4 inch S.S.A	Total Depth of Borehole	5 feet
Drill Rig Type	Fraste Multi Drill XL Track Mounted	Drilling Contractor	Pitcher	Approximate Surface Elevation	Existing Ground Surface
Groundwater Level and Date Measured	NFWE	Sampling Method(s)	Bulk	Hammer Data	140lb, 30" autotrip

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance, blows/ft	Graphic Log	MATERIAL DESCRIPTION	Dry Density (pcf)	Water Content (%)	% <#200 Sieve	PI, %	LL, %	Expansion Index (EI)	UC, psf	REMARKS AND OTHER TESTS
	0				GRAY GRAVEL W/SAND (GP). Dense, dry, fine to coarse sand, fine angular gravel to 3/4 inch diameter. (Aggregate Base)								
					BROWN & YELLOW BROWN CLAYEY SAND W/GRAVEL (SC). Loose, moist, fine to coarse sand.			46.7					
	5				Boring terminated at 5 feet. No free water encountered.								
	10												
	15												
	20												
	25												
	30												
	35												

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LOG OF BORING B-2

Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE

4

Date(s) Drilled **3/12/15**	Logged By **REP**	Checked By **EGC**	
Drilling Method **Rotary Wash**	Drill Bit Size/Type **3-1/8 inch Drag Bit &Tricone Bit, Casing to 8 feet**	Total Depth of Borehole **54-1/2 feet**	
Drill Rig Type **Fraste Multi Drill XL Track Mounted**	Drilling Contractor **Pitcher**	Approximate Surface Elevation **Existing Ground Surface**	
Groundwater Level and Date Measured **N/A**	Sampling Method(s) **Modified California, Tube**	Hammer Data **140lb, 30" autotrip**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance, blows/ft	Graphic Log	MATERIAL DESCRIPTION	Dry Density (pcf)	Water Content (%)	% <#200 Sieve	PI, %	LL, %	Expansion Index (EI)	UC, psf	REMARKS AND OTHER TESTS
	0		14		BROWN SILTY GRAVEL w/SAND (GM). Medium dense, dry, fine to coarse sand, fine to coarse subangular gravel to 1" diameter. (FILL)								
			22		GRAY GRAVEL (GP). Medium dense, dry, fine rounded gravel (Pea Gravel FILL)								
	5		33		DARK BROWN, YELLOW BROWN & OLIVE SANDY CLAY w/GRAVEL (CL). Very stiff, dry, fine to coarse sand, fine to coarse subangular gravel to 1-inch diameter. (FILL)								
					BROWN, DARK GRAY & YELLOW BROWN CLAYEY SAND w/GRAVEL (SC). Dense, dry, fine to coarse sand, fine subrounded gravel to 3/4-inch diameter. (FILL)								
	10		8		DARK GRAY & OLIVE CLAYEY GRAVEL w/SAND (GC). Loose, wet, fine to coarse sand, fine to coarse subrounded gravel to 1-1/2-inch diameter. (FILL)								
	15		20		OLIVE & RED BROWN SAND (SP). Medium dense, wet, fine to coarse sand.								
			100 psi		OLIVE GREY SILT (MH). Very soft, wet. (BAY MUD)								
	20		100 psi		OLIVE GRAY SILTY SAND w/GRAVEL (SM). Loose, wet, fine to coarse sand, fine subangular gravel to 1/2-inch diameter.								
	25												
	30		3		OLIVE GRAY SILT (MH). Soft, wet, some organics. (BAY MUD)								
	35		75 psi to 34 feet,										

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Job No: 1993.39.04.1 | Date: JUNE 2015

LOG OF BORING B-3
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE **5A**

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance, blows/ft	Graphic Log	MATERIAL DESCRIPTION	Dry Density (pcf)	Water Content (%)	% <#200 Sieve	PI, %	LL, %	Expansion Index (EI)	UC, psf	REMARKS AND OTHER TESTS
	35		75 psi		OLIVE GRAY SILT (MH). Soft, wet, some organics. (BAY MUD)								
	40												
			3										
	45				GRAY SANDY SILT (MH). Very soft, wet, fine sand. (BAY MUD)								
	50												
					RED & YELLOW BROWN CLAYEY GRAVEL w/SAND (GC). Dense, wet, fine to coarse sand.								
			43		BROWN & YELLOW BROWN SANDSTONE. Closely spaced fractures, firm, friable to moderately strong, moderately to highly weathered.								
	55				Boring terminated at 54-1/2 feet.								
	60												
	65												
	70												

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Job No: 1993.39.04.1 | Date: JUNE 2015

LOG OF BORING B-3
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE
5B

Date(s) Drilled **3/12/15**	Logged By **REP**	Checked By **EGC**
Drilling Method **Rotary Wash**	Drill Bit Size/Type **3-1/8 inch Drag Bit &Tricone Bit, Casing to 8-1/2 feet**	Total Depth of Borehole **49-1/2 feet**
Drill Rig Type **Fraste Multi Drill XL Track Mounted**	Drilling Contractor **Pitcher**	Approximate Surface Elevation **Existing Ground Surface**
Groundwater Level and Date Measured **N/A**	Sampling Method(s) **Modified California, Tube**	Hammer Data **140lb, 30" autotrip**

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance, blows/ft	Graphic Log	MATERIAL DESCRIPTION	Dry Density (pcf)	Water Content (%)	% <#200 Sieve	PI, %	LL, %	Expansion Index (EI)	UC, psf	REMARKS AND OTHER TESTS
	0				BROWN CLAYEY SAND (SC). Medium dense, moist, fine to coarse sand, fine to coarse angular gravel to 1" diameter, porous. (FILL)								
			13					38.6					
			3			103	17.6						
					BROWN CLAYEY SAND (SC). Very loose, wet, fine to coarse sand, porous. (FILL)								
	5		10			115	19	20					
					OLIVE GRAY & DARK BROWN SANDY SILT (MH). Stiff, wet, fine to coarse sand. (FILL)								
					OLIVE GRAY SANDY SILT (MH). Very soft, wet. (BAY MUD)								
	10		1		OLIVE GRAY SILT (MH). Very soft, wet, some shell fragments. (BAY MUD)	72	51.1						
	15				OLIVE GRAY SILTY SAND (SM). Very loose, wet, fine sand								
			2										
	20		12										
					OLIVE GRAY & RED SANDY GRAVEL (GP). Medium dense, wet, fine to coarse sand, fine angular gravel to 3/4 inch diameter.								
	25												
			300 psi to 39 feet, 125 psi		OLIVE GRAY SILT (MH). Soft, wet. (BAY MUD)								
	30												
	35												

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Job No: 1993.39.04.1 | Date: JUNE 2015

LOG OF BORING B-4
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE
6 A

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance, blows/ft	Graphic Log	MATERIAL DESCRIPTION	Dry Density (pcf)	Water Content (%)	% <#200 Sieve	PI, %	LL, %	Expansion Index (EI)	UC, psf	REMARKS AND OTHER TESTS
	35				OLIVE GRAY SILT (MH). Soft, wet. (BAY MUD)								
	40												
	45												
			33/4"		LIGHT BROWN & OLIVE SANDSTONE. Very closely fractured, firm, plastic to weak, highly weathered								
	50				Boring terminated at 49-1/2 feet.								
	55												
	60												
	65												
	70												

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Job No: 1993.39.04.1 | Date: JUNE 2015

LOG OF BORING B-4

Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE

6 B

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance, blows/ft	Graphic Log	MATERIAL DESCRIPTION	Dry Density (pcf)	Water Content (%)	% <#200 Sieve	PI, %	LL, %	Expansion Index (EI)	UC, psf	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11	12	13	14

COLUMN DESCRIPTIONS

1 Elevation (feet): Elevation (MSL, feet).
2 Depth (feet): Depth in feet below the ground surface.
3 Sample Type: Type of soil sample collected at the depth interval shown.
4 Sampling Resistance, blows/ft: Number of blows to advance driven sampler one foot (or distance shown) beyond seating interval using the hammer identified on the boring log.
5 Graphic Log: Graphic depiction of the subsurface material encountered.
6 MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.
7 Dry Density (pcf): Dry density, in pcf.
8 Water Content (%): Water content, percent.
9 % <#200 Sieve: % <#200 Sieve
10 PI, %: Plasticity Index, expressed as a water content.
11 LL, %: Liquid Limit, expressed as a water content.
12 Expansion Index (EI): Expansion Index (EI)
13 UC, psf: Unconfined compressive strength, in pounds per square foot.
14 REMARKS AND OTHER TESTS: Comments and observations regarding drilling or sampling made by driller or field personnel.

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity
COMP: Compaction test
CONS: One-dimensional consolidation test
LL: Liquid Limit, percent
PI: Plasticity Index, percent
SA: Sieve analysis (percent passing No. 200 Sieve)
UC: Unconfined compressive strength test, Qu, in psf
WA: Wash sieve (percent passing No. 200 Sieve)

MATERIAL GRAPHIC SYMBOLS

Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
Lean-Fat CLAY, CLAY w/SAND, SANDY CLAY (CL-CH)
Clayey GRAVEL (GC)
Silty GRAVEL (GM)
Poorly graded GRAVEL (GP)
SILT, SILT w/SAND, SANDY SILT (MH)
Sandstone
Clayey SAND (SC)
Silty SAND (SM)
Poorly graded SAND (SP)

TYPICAL SAMPLER GRAPHIC SYMBOLS

Bulk Sample
2.5-inch-OD Modified California w/ brass liners
2-inch-OD unlined split spoon (SPT)
Shelby Tube (Thin-walled, fixed head)

OTHER GRAPHIC SYMBOLS

Water level (at time of drilling, ATD)
Water level (after waiting)
Minor change in material properties within a stratum
Inferred/gradational contact between strata
? Queried contact between strata

GENERAL NOTES

1: Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2: Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

LAYERING

MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 inches
THINLY BEDDED	2½ to 8 inches
VERY THINLY BEDDED	¾ to 2½ inches
CLOSELY LAMINATED	¼ to ¾ inches
VERY CLOSELY LAMINATED	Less than ¼ inch

JOINT, FRACTURE, OR SHEAR SPACING

VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 inches
CLOSELY SPACED	2½ to 8 inches
VERY CLOSELY SPACED	¾ to 2½ inches
EXTREMELY CLOSELY SPACED	Less than ¼ inch

HARDNESS

Soft - pliable; can be dug by hand

Firm - can be gouged deeply or carved with a pocket knife

Moderately Hard - can be readily scratched by a knife blade; scratch leaves heavy trace of dust and is readily visible after the powder has been blown away

Hard - can be scratched with difficulty; scratch produces little powder and is often faintly visible

Very Hard - cannot be scratched with pocket knife, leaves a metallic streak

STRENGTH

Plastic - capable of being molded by hand

Friable - crumbles by rubbing with fingers

Weak - an unfractured specimen of such material will crumble under light hammer blows

Moderately Strong - specimen will withstand a few heavy hammer blows before breaking

Strong - specimen will withstand a few heavy ringing hammer blows and usually yields large fragments

Very Strong - rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

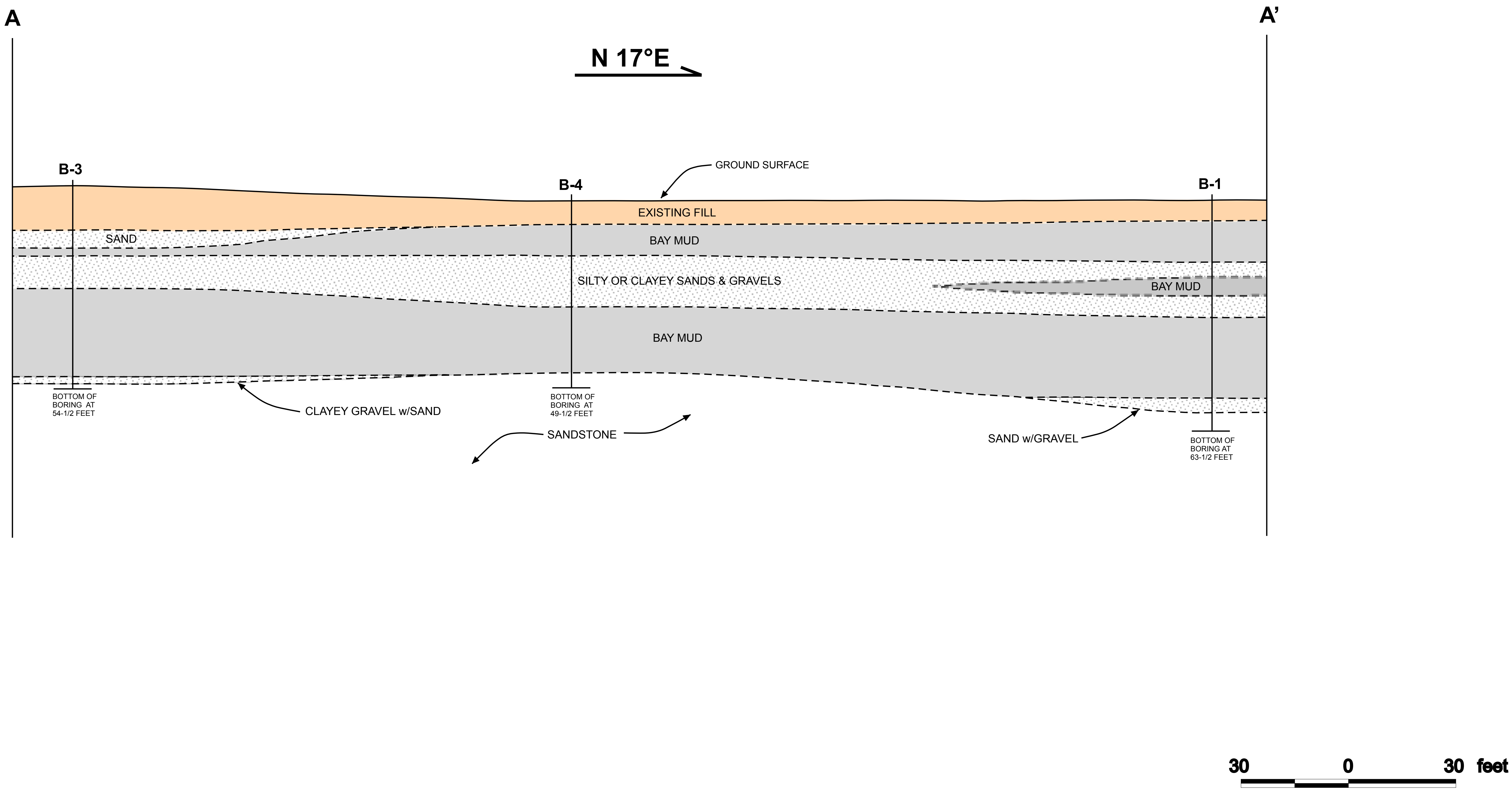
DEGREE OF WEATHERING

Highly Weathered - abundant fractures coated with oxides, carbonates, sulphates, mud, etc., thorough discoloration, rock disintegration, mineral decomposition

Moderately Weathered - some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

Slightly Weathered - a few stained fractures, slight discoloration, little or no effect on cementation, no mineral composition

Fresh - unaffected by weathering agents; no appreciable change with depth



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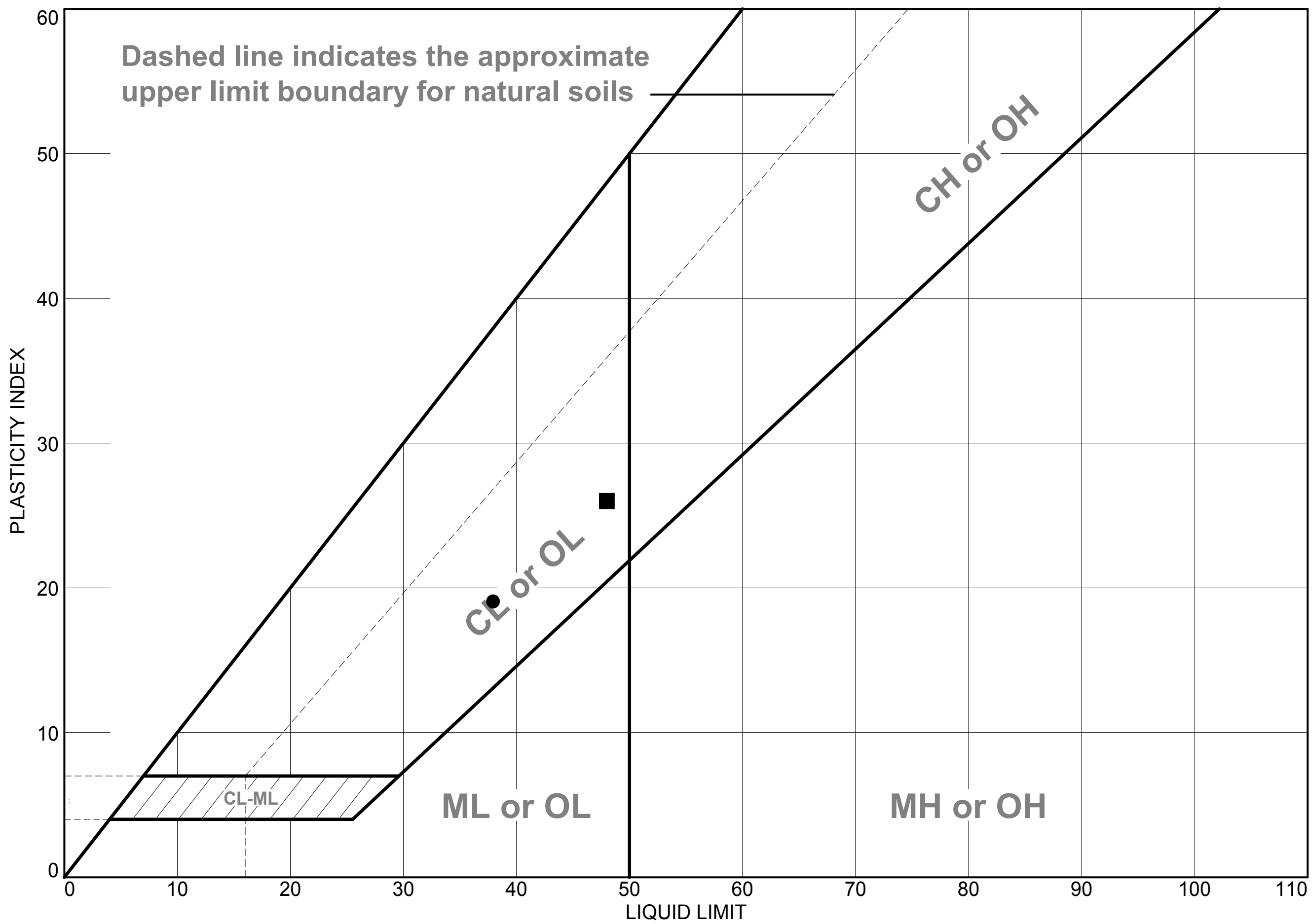
Job No: 1993.39.04.1 | Date: JUNE 2015

CROSS SECTION A-A'
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE 9

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brn Clayey Sand W/ Gravel (SC)	38	19	19		46.7	SC
■	Brn Clayey Sand W/ Gravel (SC)	48	22	26		38.6	SC

Project No. 1993.39.04.1 **Client:** RGH Consultants

Project: Dunphy Park Improvements

●**Source of Sample:** B-2 **Sample Number:** Bulk

■**Source of Sample:** B-4 **Depth:** 1.5’ & 2.0’

Remarks:

●Expansion Index=60

■Expansion Index=56

Tested By: SW

Checked By: GEF

RGH CONSULTANTS

Job No: 1993.39.04.1 Date: JUNE 2015

CLASSIFICATION TEST DATA

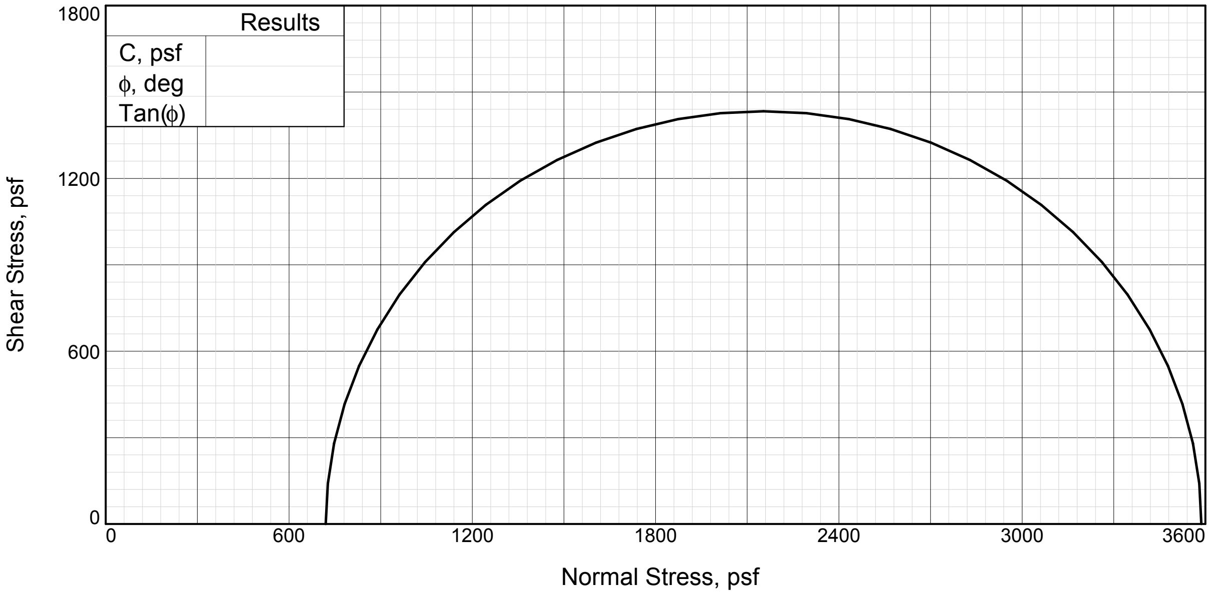
Dunphy Park Improvements

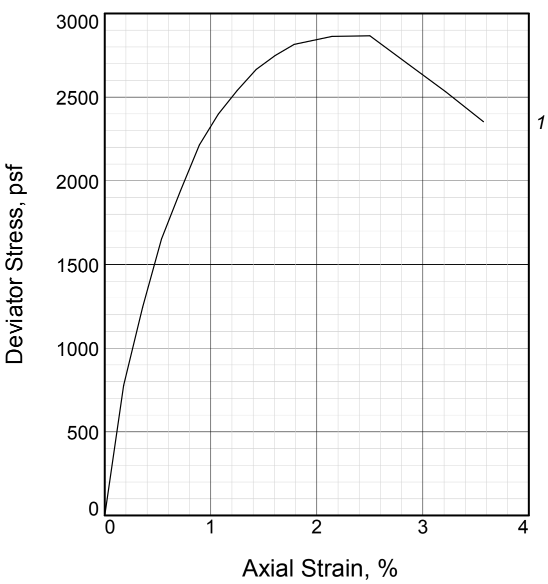
Bridgeway Boulevard

Sausalito, California

PLATE

10





Sample No.		1
Initial	Water Content, %	15.2
	Dry Density, pcf	100.5
	Saturation, %	60.5
	Void Ratio	0.6775
	Diameter, in.	2.390
	Height, in.	5.600
At Test	Water Content, %	15.2
	Dry Density, pcf	100.5
	Saturation, %	60.5
	Void Ratio	0.6775
	Diameter, in.	2.390
	Height, in.	5.600
Strain rate, in./min.		0.060
Back Pressure, psf		0
Cell Pressure, psf		720
Fail. Stress, psf		2867
Ult. Stress, psf		2867
σ_1 Failure, psf		3587
σ_3 Failure, psf		720

Type of Test:
Unconsolidated Undrained
Sample Type: Undisturbed
Description: Brn Clayey Sand (SC)

Specific Gravity= 2.70
Remarks:

Tested By: SW
Checked By: GEF

Client: RGH Consultants

Project: Dunphy Park Improvements

Source of Sample: B-4 **Depth:** 2.0’

Proj. No.: 1993.39.04.1 **Date Sampled:** 4/6/15

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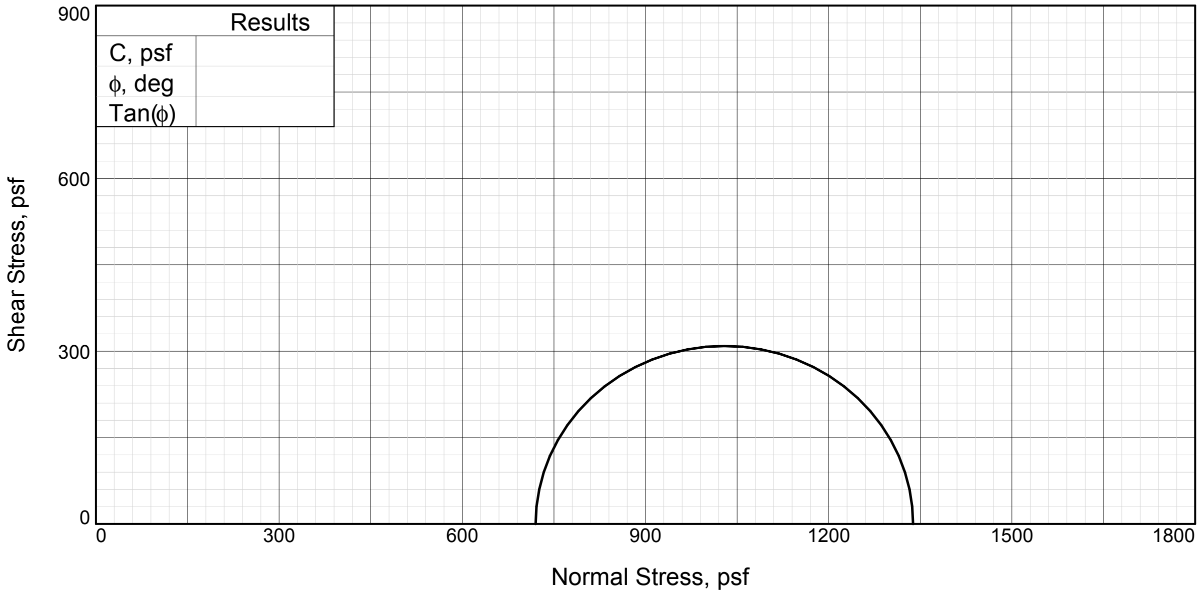
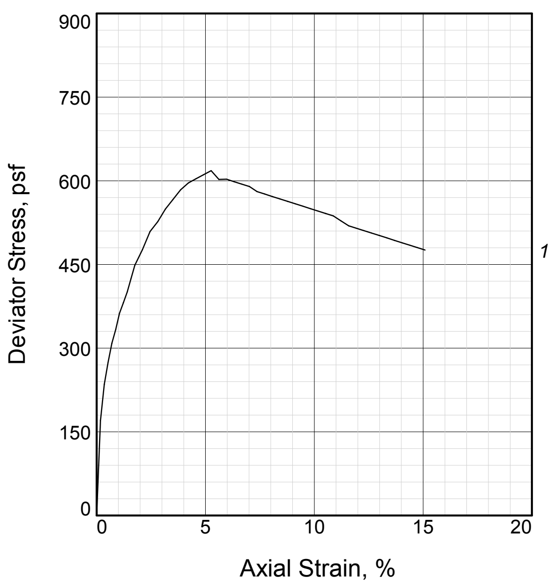
Job No: 1993.39.04.1 | Date: JUNE 2015

TRIAXIAL TEST DATA
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE
11





Sample No.		1
Initial	Water Content, %	30.9
	Dry Density, pcf	90.0
	Saturation, %	95.5
	Void Ratio	0.8733
	Diameter, in.	2.410
	Height, in.	5.700
At Test	Water Content, %	30.9
	Dry Density, pcf	90.0
	Saturation, %	95.5
	Void Ratio	0.8733
	Diameter, in.	2.410
	Height, in.	5.700
Strain rate, in./min.		0.060
Back Pressure, psf		0
Cell Pressure, psf		720
Fail. Stress, psf		618
Ult. Stress, psf		618
σ_1 Failure, psf		1338
σ_3 Failure, psf		720

Type of Test:
Unconsolidated Undrained
Sample Type: Undisturbed
Description: Grey Clayey Sand (SC)

Specific Gravity= 2.70
Remarks:

Tested By: SW
Checked By: GEF

Client: RGH Consultants

Project: Dunphy Park Improvements

Source of Sample: B-1 **Depth:** 4.0’

Proj. No.: 1993.39.04.1 **Date Sampled:** 4/6/15

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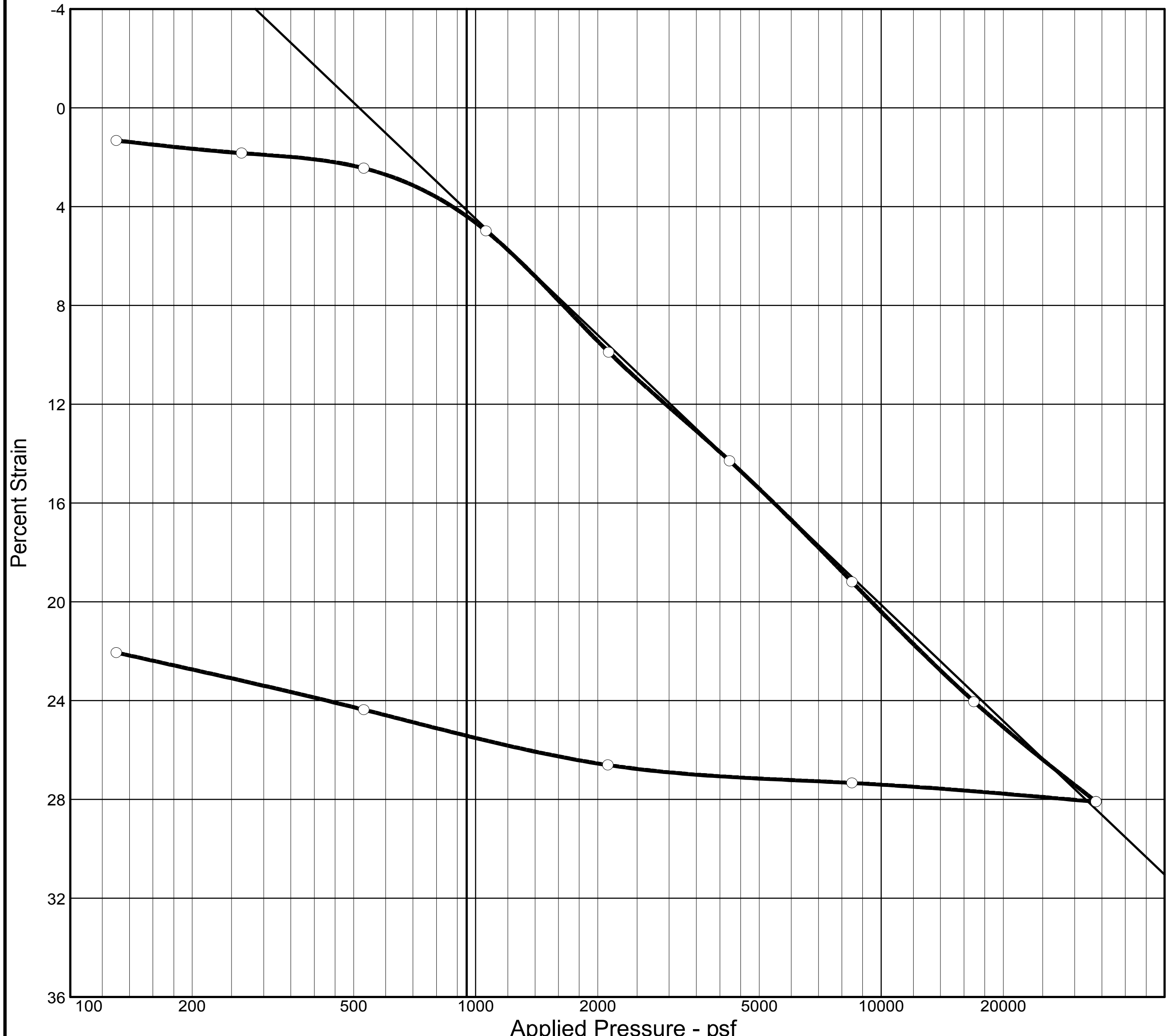
Job No: 1993.39.04.1 | Date: JUNE 2015

TRIAXIAL TEST DATA
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE
12

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Heave %	e_o
Sat.	Moist.											
99.7 %	44.1 %	76.8			2.70		1007	0.34	0.05			1.195

MATERIAL DESCRIPTION	USCS	AASHTO
Grey Clay (CH)		

Project No. 1993.39.04.1 **Client:** RGH Consultants

Project: Dunphy Park Improvements

Source: B-1 **Elev./Depth:** 7.0’-9.0’

Remarks:

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CONSOLIDATION TEST DATA

Dunphy Park Improvements

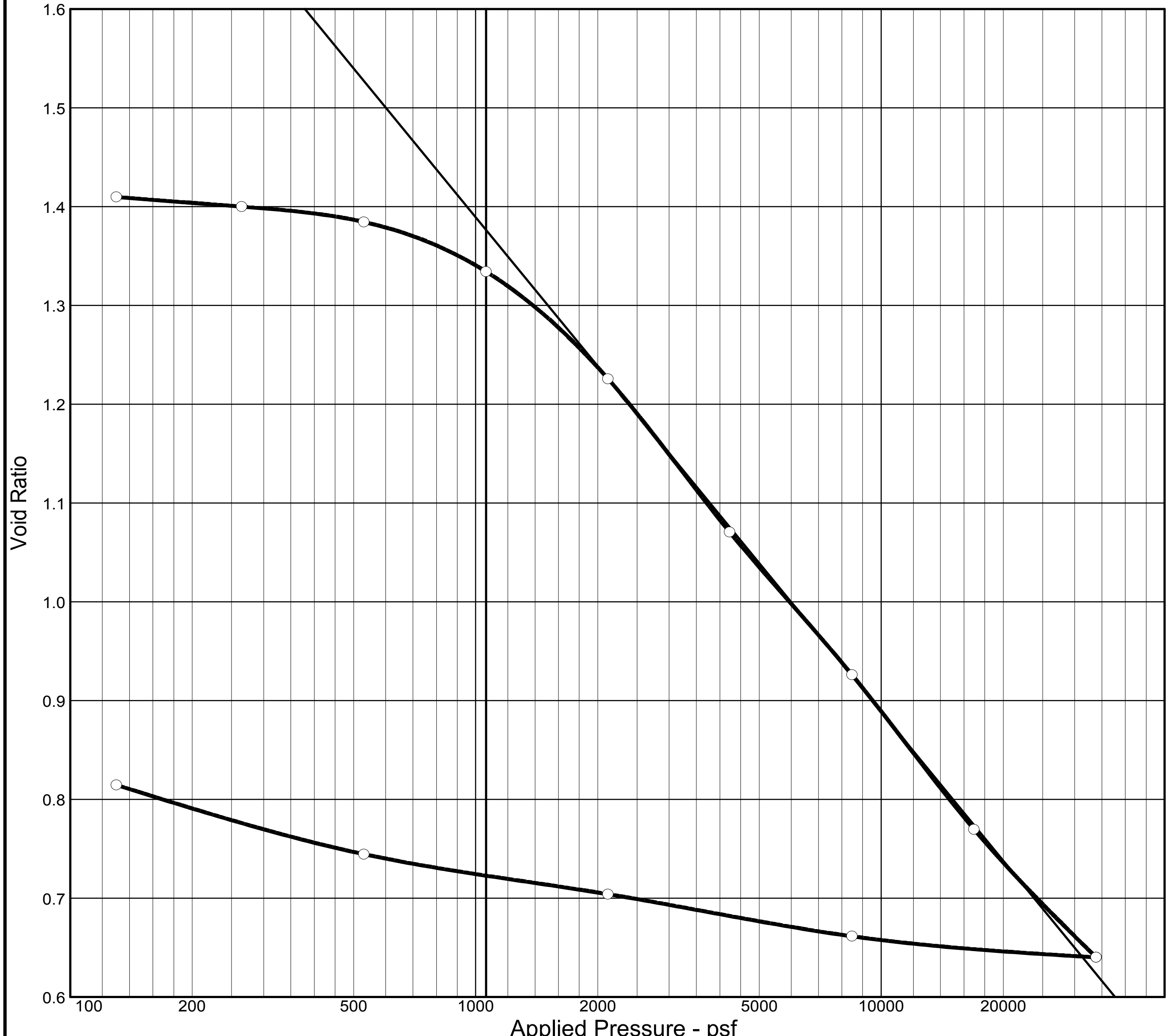
Bridgeway Boulevard

Sausalito, California

PLATE

13

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Heave %	e_o
Sat.	Moist.											
99.9 %	53.1 %	69.2			2.70		1376	0.50	0.07			1.436

MATERIAL DESCRIPTION	USCS	AASHTO
Grey Clay (CH)	CH	

Project No. 1993.39.04.1 **Client:** RGH Consultants

Project: Dunphy Park Improvements

Source: B-3 **Elev./Depth:** 23.0'

Remarks:

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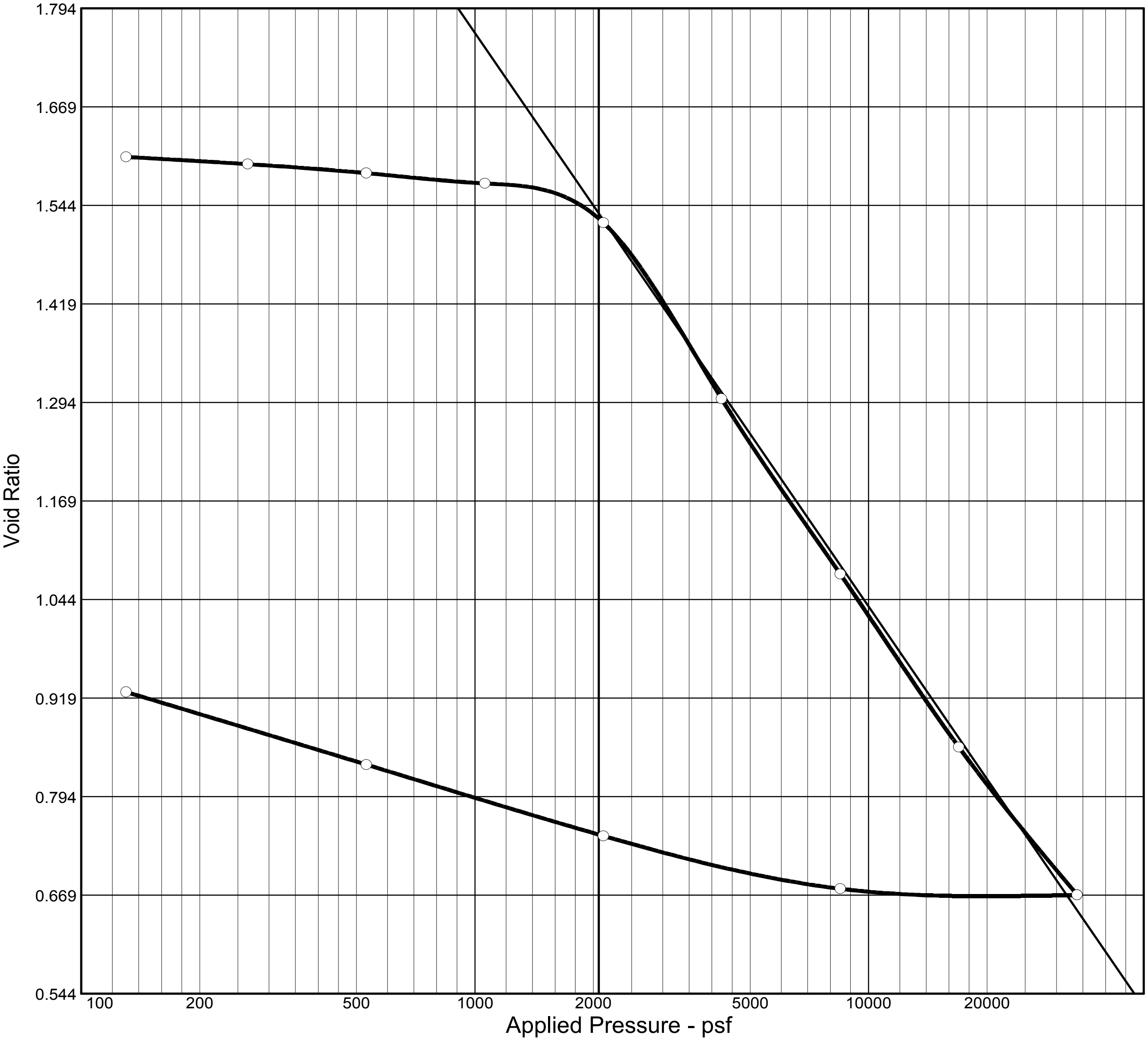
Job No: 1993.39.04.1 | Date: JUNE 2015

CONSOLIDATION TEST DATA
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE **14**

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Heave %	e_o
Sat.	Moist.											
99.9 %	59.7 %	64.5			2.70		2128	0.73	0.11			1.614

MATERIAL DESCRIPTION	USCS	AASHTO
Grey Clay (CH)	SC	

Project No. 1993.39.04.1 **Client:** RGH Consultants

Project: Dunphy Park Improvements

Source: B-3 **Elev./Depth:** 33.0'

Remarks:

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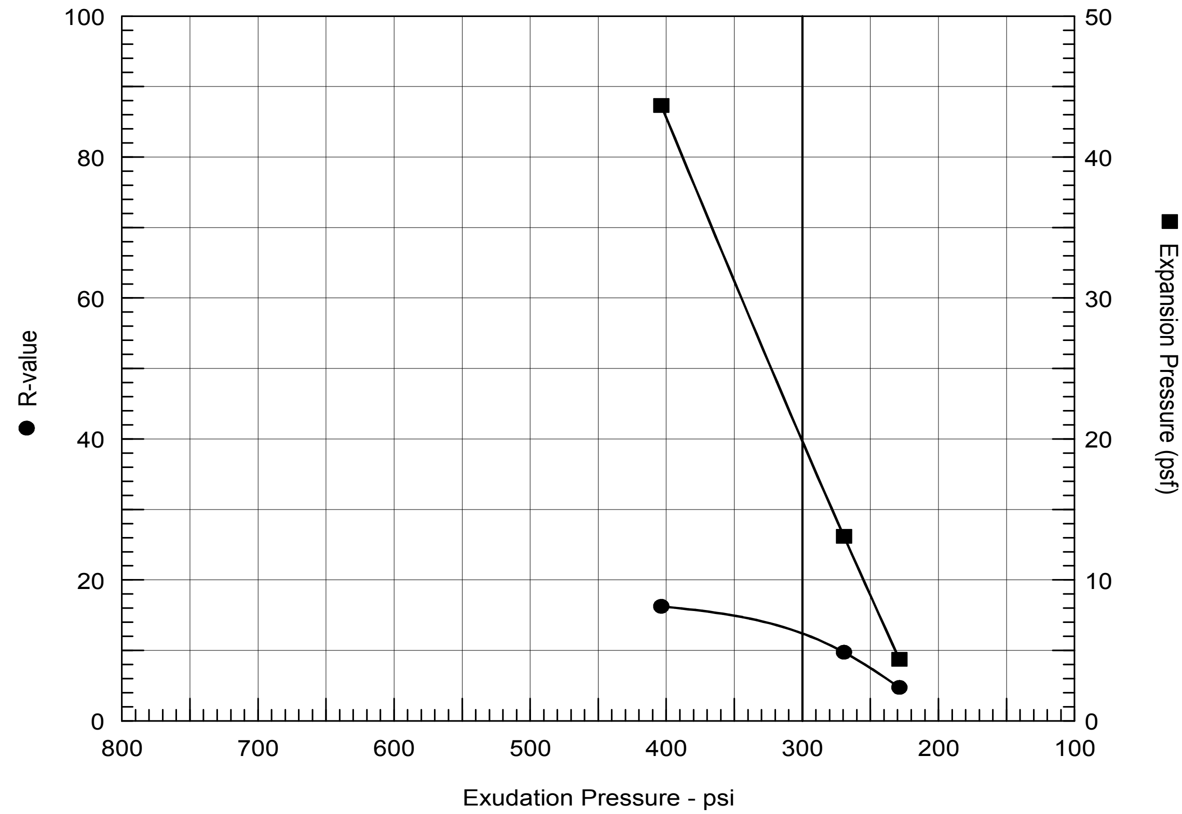
Job No: 1993.39.04.1 Date: JUNE 2015

CONSOLIDATION TEST DATA
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE

15

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	55	110.5	18.8	4	146	2.50	229	5	5
2	100	112.5	17.6	13	133	2.52	269	10	10
3	170	118.6	14.8	44	121	2.53	404	16	16

Test Results	Material Description
R-value at 300 psi exudation pressure = 12 **Exp. pressure at 300 psi exudation pressure =** 20 psf	Brn Clayey Sand W/ Gravel (SC)

Project No.: 1993.39.04.1
Project: Dunphy Park Improvements
Source of Sample: B-2
Sample Number: Bulk
Date: 4/9/2015

R-VALUE TEST REPORT

Tested by: SEF
Checked by: GEF
Remarks:

RGH CONSULTANTS

Job No: 1993.39.04.1 | Date: JUNE 2015

RESISTANCE (R) VALUE DATA
Dunphy Park Improvements
Bridgeway Boulevard
Sausalito, California

PLATE
16

APPENDIX B - REFERENCES

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APPENDIX C - DISTRIBUTION

City of Sausalito (6,0,1e)
Attn: Jonathon Goodman
420 Litho Street
Sausalito, CA 94965
JGoldman@ci.sausalito.ca.us

Prunuske Chatham, Inc. (0,0,1e)
Attn: John Ferons
400 Morris Street, Suite G
Sebastopol, CA 95472
jferons@pcz.com

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