GEOTECHNICAL STUDY REPORT

PIER 70 CRANE COVE PARK
PHASE II
PORT OF SAN FRANCISCO
SAN FRANCISCO, CALIFORNIA

Prepared for:

AECOM

Prepared By:

AGS

November 2014

GEOTECHNICAL STUDY REPORT

PIER 70 CRANE COVE PARK
PHASE II
PORT OF SAN FRANCISCO
SAN FRANCISCO, CALIFORNIA

AGS Job No. KK0210

Prepared for:
AECOM

Prepared By:

AGS

November 2014

5 Freelon Street, San Francisco, CA 94107 • Phone (415) 777-2166 • Fax (415) 777-2167

TABLE OF CONTENTS

TABLES

PLATES

APPENDICES

1. INTRODUCTION

1.1 GENERAL

This report presents the results of our Phase II Geotechnical Study for the proposed Pier 70 - Crane Cove Park Project in San Francisco, California. The study addresses the latest Crane Cove Master Plan, 2013. The project site location is shown on Plate 1, Site Location Map. AGS previously (2012) performed a Phase I Geotechnical Study primarily to provide recommendations for foundation retrofit of the two existing cranes and evaluate the geotechnical concerns at this site. The current study included additional borings to evaluate the feasibility of the latest Master Plan including construction of additional structures, grading, roadways, and pavement designs from the geotechnical point of view.

This report includes our geotechnical recommendations for this site associated with the design and construction of the proposed Crane Cove Park. Of particular importance to the design of this project are an assessment of the liquefaction potential of the subsurface materials, static and seismically-induced settlements, and seismically-induced lateral deformation potential. The conclusions and recommendations presented in this report are based on subsurface conditions encountered in 13 borings drilled for Phase I and Phase II, as shown on Plate 2, and laboratory testing results for this study. These conclusions and recommendations should not be extrapolated to other areas or used for other facilities without our prior review.

1.2 PROJECT DESCRIPTION

Crane Cove Park covers approximately 9 acres of improvements within the Pier 70 area. The park is generally bounded by The Ramp Restaurant to the north, Illinois Street to the east, a future extension of 19th Street to the south, and the Bay to the east.

Based on the Illustrative Plan Exhibit of the Master Plan dated December 2013, the park improvement includes five areas that each serves a different program of uses as described below. Plate 3, Proposed Improvement Plan, shows the limit of each area. These five areas are:

- **Open Green Area:** A roughly trapezoidal shape area adjacent to Illinois Street is planned to be a multi-use lawn and picnic site. Existing buildings within this area will be demolished as part of the development except Building 49. It is AGS' understanding that a shallow foundation system is supporting the Building 49. The existing Building 49 is planned to be remodeled to a recreation and aquatic center.

- **Slipway 4 (Keel Park) /19th Street Extension:** The plan proposes creation of a new plaza by reconfiguration of the existing Slipway 4 and its associated runways in this area where large outdoor public events can occur. As a part of developments, the existing Crane 30 will be retained in its current location on the eastern runway but the existing Crane 14 will be relocated bay-ward on the western runway. In addition, a new overlook ramp structure (Overlook Ramp 2) will be constructed at the northern terminus point of the eastern runway of the Slipway 4. It is AGS' understanding that a retaining wall will be constructed at the southern part of the west runway. The proposed retaining wall will retain the proposed rise in grade within the Open Area ranging from approximately 1 to 6 feet. It is AGS' understanding that a shallow foundation system is supporting the existing Building 30. Existing Building 30 is planned to be remodeled for park maintenance and support uses.

  The existing 19th Street is planned to be extended to provide vehicular access to the park and connect to 20th Street and other Pier 70 shoreline open spaces. Concrete seat-walls will connect the Slipway 4 to the sidewalk adjacent to the proposed 19th Street extension.

- **Building 109/110 Area:** This park space will include areas on the western side of Building 109 and Building 110. It is AGS' understanding that a shallow foundation system is supporting Buildings 109 and 110. A playground, picnic area, parking, and outdoor café area are proposed in this area. The new Master Plan shows that the existing Building 50 will be demolished in the proposed development.

- **Maritime Fields:** This space, including Slipways 2 and 3 which were completely filled in the early 1960's, will be partially excavated at the shoreline to subject this area to tidal activity and allow for a planted shoreline edge. The majority of the southern portion of this area will be utilized for a potential off-leash dog area and concrete plaza within the park. It is AGS' understanding that the area of Slipway 1 is not part of Phase II development.

- **Northern Shoreline Area (Urban Beach Area):** This feature is a sloped, beach-like gravel, pebble, boulder and riprap area including boat launch shoreline area with staging area. A curving or crescent-shaped waterfront walk-path arcs from the existing The Ramp Restaurant all the way into Slipway 4. An overlook ramp structure (Overlook Ramp 1) adjacent to The Ramp Restaurant is planned for this project site. Areas on west of the proposed waterfront walk is planned to be improved to picnic sites and landscapes. Existing buildings in this area will be demolished except Kneass Building. It is AGS' understanding that a shallow foundation system is supporting Kneass Building.

The existing slopes in some areas in the site are protected by seawalls. No as-built information of the seawalls was available at the time of this report. The structural integrity of the existing seawalls is not within the scope of work of this report. A Structural Engineer should evaluate any structural integrity of the existing seawalls and recommend a remediation plan, if necessary.

Based on the preliminary grading plan provided to us by the project Civil Engineer, majority of the site will be cut from 6 inches to 6 feet (roughly 20,000 yd^3) and will be filled along Illinois Street and proposed 19th Street extension and east and southeast of Kneass Building from 4 inches to 6 feet (roughly 2,500 yd^3). It is AGS' understanding that the ground level on the western part of the site is planned to be risen to match the existing Illinois Street grade. Currently, a concrete gravity retaining wall exists between the Illinois Street and the site. There was no as-built information regarding the foundation of the existing retaining wall.

Structural loading information for the existing cranes was provided by the Structural Engineer of the project. No loading information was available for the proposed overlook ramp structures and the proposed retaining wall at the time of this report. Preliminary proposed grading plan was provided to AGS by the Civil Engineer of the project. No traffic information for the 19th Street extension and parking was available at the time of preparation of this report.

1.3 WORK PERFORMED

The work that AGS completed for this project is outlined in AGS' proposal to AECOM, dated January 23, 2014. The five tasks that AGS completed include the review of available information, preparation of a geotechnical investigation Work Plan, completion of a subsurface exploration program, completion of a laboratory testing program, performance of appropriate engineering analyses and development of geotechnical evaluations, and preparation of a report, including the findings and recommendations.

1.3.1 Review of Available Information

AGS reviewed available geotechnical and environmental information obtained by AGS and others at the site and its vicinity, including the reports by AGS (2012), Dames and Moore (1969 and 1973), AGS (1989), and Treadwell and Rollo (2010). AGS also reviewed the drawings and construction photos from Bethlehem Steel Company dated March 1945 and May 1941, respectively. In addition, AGS reviewed the photos provided in the Architectural Resources Group Report, "Documentation and Assessment of Historic Artifacts", dated December 2011. The available data reviewed included geologic and seismologic information pertaining to the site, including the USGS Geological Maps (Schlocker, Graymer and others, 2006), Seismic Hazard Zone Report and Map (California Division of Mines and Geology, 2000), and other sources.

1.3.2 Geotechnical Investigation Work Plan

AGS prepared a geotechnical investigation Work Plan to describe the objectives of the field investigation and procedures for drilling and logging borings. The geotechnical investigation program was carried out in accordance with the general approach and field procedures presented in the Work Plan.

1.3.3 Field Exploration

AGS completed a site reconnaissance to evaluate site access, ingress and egress, and to mark the proposed boring locations. After selecting the exploratory boring locations, AGS applied for permits from the Port of San Francisco, and the City and County of San Francisco Department of Public Health. In addition, AGS notified Underground Service Alert (USA) so that utility companies could mark their underground utilities. For this phase of the project, eleven (11) exploratory borings, Boring B-3 through Boring B-13, were drilled by Geo-Ex Subsurface Exploration of Dixon, California from February 12th through February 14th, and from February 17th through February 21st, using a CME 75 truck-mounted drilling rig.

The field exploration program was performed under the technical supervision of a qualified AGS geologist, who completed a log of each boring, documented the drilling progress, and recorded the subsurface conditions encountered at the location of each boring. The location of the exploratory borings is shown on Plate 2, Approximate Boring Location Map, including the locations of previous borings drilled by AGS and others. The deep borings, Borings B-3 through B-10, were extended to depths ranging from approximately 29.5 to 110.5 feet below the existing ground surface (bgs). Three shallow borings, Borings B-11 through B-13, were extended to depths of approximately 6 to 11 feet bgs. Details of the subsurface exploration including the logs of test borings are presented in Appendix A.

1.3.4 Laboratory Testing

A laboratory testing program was performed on selected soil samples obtained during the field exploration program. The laboratory tests included moisture content, dry density, sieve and wash analyses, Atterberg limits, unconfined compressive strength, unconsolidated undrained triaxial, incremental consolidation, R-value, and corrosivity testings as appropriate for the various soils encountered. Consolidation, unconsolidated undrained triaxial, corrosivity, and R-value testings were performed by Cooper Testing Laboratories of Palo Alto, CA, while the other tests were performed by AGS. The details of the geotechnical laboratory testing program are included in Appendix B - Geotechnical Laboratory Testing.

Analytical testing was completed on a composite soil sample for purposes of waste characterization, off-haul, and disposal. The soil sample was delivered with the appropriate chain of custody forms to Curtis & Tompkins, Ltd., a state licensed analytical testing laboratory located in Berkeley, California.

1.3.5 Geotechnical Evaluations and Report Preparation

Engineering analyses were performed based on the field and laboratory data to develop geotechnical conclusions and recommendations for the design and construction of the proposed project. Our geotechnical findings, conclusions, and recommendations, along with the supporting field and laboratory data, are presented in this engineering report, which includes the following:

- Site surface and subsurface conditions;
- Earthwork recommendations, including subgrade preparation, fill and backfill criteria, and over-excavation requirements;
- Geoseismic hazards including bedrock and ground surface accelerations, liquefaction potential, seismically-induced settlements and lateral deformations;
- Consequences of liquefaction on the shoreline slopes and the utilities and proposed mitigation measures;
- Liquefaction mitigation design considerations;
- Time-dependent settlements and mitigation measures;
- Foundation design criteria and design parameters for the cranes, overlook structures, and retaining structures, including the type and size of the foundations, allowable bearing pressures, settlements, and uplift resistance;
- Retaining wall design recommendations and parameters;
- Effects of the new fill on adjacent structures and required mitigation measures;
- Flexible pavement design;
- Shoreline protection alternatives and design parameters; and
- Construction considerations.

2. FINDINGS

2.1 SITE CONDITIONS

The area covered by the Phase II Geotechnical Study is larger than the area covered by the Phase I Geotechnical Study, as it not only includes the large yard originally part of the historic Union Ironworks / Bethlehem Steel Yard at Pier 70, but also the neighboring properties. The neighboring properties include a boat breaking yard and maintenance facility, vacant Pier 66 Boatyard site, public access walkway, and waterfront area.

At the time of field exploration, the towing yard and stone warehouse within the large yard area were abandoned. The two existing warehouse buildings located approximately 50 feet east of the 18th Street and Illinois Street intersection, and 230 feet southeast of the same intersection were empty. The major feature consists of Slipway 4, with its large runways situated in the center of the yard. On either side of the concrete ramp, are the platforms and dilapidated support systems formerly used as a cargo rigging, and also used to house the mechanical, electrical and lighting systems. The slipway ramp is constructed of concrete ties laid perpendicular to one another to form three independent cribwork structures running the length of the ramp. The height of the cribwork forms decreases as the ramp approaches the launching basin. Based on a review of “Building Slip No. 4 - Plan, Elevations, and Sections”, the drawing provided by Bethlehem Steel Company (March 1945) the slipway is supported by timber piles. Detailed information regarding the size and location of the piles was not available at the time of this report.

Due to age and lack of corrosion protection of existing steel seawalls, the exposed steel members of the structures have been corroded. The steel members were reinforced by tie-back; however, the contact between the steel members and tie-backs were lost due to corrosion.

The two inoperative cranes at the inland end of Slipway 4, 50-ton crane (Crane 14) and 30-ton crane (Crane 30), are located at the head of the slipway. These cranes appear in very similar conditions to conditions that were observed at the time of the Phase I subsurface exploration in 2011. Both cranes are tower crane mounted on wheeled traction tracks that allow them to travel along the runway rail at the sides of Slipway 4.

The existing buildings in the project site include Buildings 50 and 110 located east of the Slipway 4, Building 109 located southeast of the Slipway 4, Buildings 49 and 30 located west of the Slipway 4, and Kneass Building located northwest of the project site. There was no information of the remaining structures within the site. OLMM Consulting Engineers prepared a Seismic Review Report for Buildings 50 and 110, dated May 2008. According to the OLMM report, Building 50 is a one-story, approximately 30 feet by 25 feet steel frame building built in 1941 and is supported on a shallow spread footing. The size of the foundation for Building 50 is unknown. Signs of settlement and cracks on the ground floor concrete slab were observed during the OLMM site visit. Building 110 is a one story, approximately 85 feet by 46 feet steel frame structure built in 1936. No structural drawings were available to show the type or size of the foundations.

Existing Slipways 2, and 3 located northwest of the site are in-filled with metal debris and wood, and are covered with concrete slabs. The grade between the slipways varies between the docking levels and top of berm elevations. A large pile of concrete rubble exists in the area of Slipways 2.

Existing underground utilities at the site includes sanitary sewer main force, gas, electric, storm drain, air, oxygen, acytylene and salt and domestic water. Some of the utility lines such as oxygen, air, acytylene and salt water have already been abundant. Except for electric line and part of sanitary sewer main force which extend parallel to the shorelines, the other utility lines extend transverse to the shorelines.

Ground elevations around the Slipway 4 and the cranes range from approximately -8 to +4 feet, based on the San Francisco City Datum, which is about 11.8 feet above the Mean Lower Low Water (MLLW) datum.

The majority of the site that includes Slipways 2 through 4 is covered by gravel, with an area of weeds, grass and sod to the southeast of the Slipway 4, including the location where Borings B-4 through B-10 were drilled as part of the current Phase II study, and where Borings B-1 and B-2 were previously drilled for the Phase I study. Remnants of old railroad tracks were also noted during AGS' site visits. In the area situated north of the main yard, where Boring B-3, and Borings B-11 through B-13 were drilled, the ground is covered by a thick layer of concrete and wood piles seems to underlie several areas.

2.2 SITE HISTORY

Pier 70 is the location of the most important intact 19th century industrial complex west of the Mississippi River and has built or repaired ships. Accordingly, under the Pier 70 Plan, it will be nominated to the National Register as a Historic District for its contribution to the industry between 1884 – 1945 for steel hull ship building, and for its industrial architecture and design. The entire Pier 70 area includes approximately 47 historic resources that may contribute to an eligible National Register Historic District, six of these historic resources, most significantly Slipway 4, are located within the Crane Cove Park site and in many ways have provided the context for the park Master Plan.

The site is underlain by artificial fill placed seaward of the historic shoreline. Fill was placed on top of the Young Bay Mud (YBM), and a majority of fill was placed from the late 1800s to the early 1900s. The original shoreline was comprised of serpentinite bluffs overlooking mud flats that extended into the San Francisco Bay. Along many portions of the shoreline, these bluffs were blasted and the resulting rock used as bay fill, including at the project site, where serpentinite rock and other Franciscan rock fragments are found in the fill.

2.3 GEOLOGY

The northern San Francisco peninsula, including the project area, is considered a part of the Coast Ranges Geomorphic Province. The province is a seismically active region characterized by northwest-trending mountains, valleys, and faults. The peninsula is bordered on the east by the San Francisco Bay, a drowned, northwest-trending structural depression. The bay and much of the peninsula are underlain by the Jurassic to Cretaceous-age (Late Mesozoic) rocks of the Franciscan Complex.

Beneath the San Francisco Bay, and along much of its margin, the Franciscan bedrock is overlain by a young, geologically unconsolidated sedimentary sequence, which, in places, exceeds 400 feet in thickness. The sequence is often subdivided into three "natural" units - Old Bay Mud (OBM), Bay Deposits (Bay Side Sand), and Young Bay Mud (YBM) (Goldman, 1969). In the Mission Bay and Potrero Point area, which include the project site, the shoreline has been extensively modified through the placement of artificial fill placed along the margins of the bay to claim marshland and land once covered by shallow water. The geologic units underlying the project site and its vicinity are briefly described below and shown on Plate 4, Geology Map.

Fill. The project site is underlain by artificial fill materials placed seaward of the historic shoreline between the late 1800s and early 1900s. Shoreline bluffs in the project vicinity near the edge of Potrero Hill were blasted and sometimes used as fill. Some of this fill appears to have been placed on top of old wood pilings that were driven into the bay along the old shoreline. More recent fill includes concrete rubble and mixed earth fill associated with past demolition and grading.

Young Bay Mud (YBM). This Holocene-age unit typically consists of soft and unconsolidated silty clay, with subsidiary lenses of loose to medium dense silty sand or poorly graded sand. Scattered shell fragments are common within these deposits due to their shallow marine origin. Deposits are young, with deposition beginning at the end of the last ice age, when sea-levels started to rise about 11,000 years ago, eventually inundating the valley which is now defined by the Bay. The YBM is still forming where there is gradual sedimentation beneath bay waters.

The thickness of the unit varies, generally reflecting the former bathymetry of San Francisco Bay. At the project site, the YBM unit forms a layer varying in thickness from approximately 25 to 40 feet layer beneath the artificial fill.

Alluvium / Bayside Deposits (BD) and Old Bay Mud (OBM). The non-marine sediments of this unit were deposited during glacial periods of low sea level. This unit is highly variable in stratigraphy, having been deposited under a range of environments, such as alluvial fans, streams, lakes, and marshes. In the San Francisco area, this unit is generally sandy, with silty sands in the lower part of the unit. Alluvium in the project area commonly includes some angular to subangular gravel of the Franciscan Formation that has been eroded from the nearby hillsides.

The OBM is a silty/sandy clay unit that was deposited in similar conditions to that of the Bay Deposits, but has been consolidated to a stiff to very stiff clay by the overlying sediments. This unit was encountered during our field exploration program at depths below the YBM. This unit was sometimes not distinguished from Bay Deposits, but the occurrence of stiff, often greenish-gray or olive-colored clay indicates the units was deposited in an extensive layer underneath the more seaward portions of the project site.

<u>Bedrock.</u> Bedrock underlying the project site and its vicinity consists of serpentinite and sedimentary rocks of the Franciscan Complex. Bedrock outcrops within approximately 300 feet of the project site, in the vicinity of the intersection of Illinois Street and 20th Street. The bedrock surface slopes northeastward and underlies the project site and vicinity at depths below the existing ground surface typically ranging from approximately 20 to 60 feet near Illinois Street and the vicinity south of Crane 30, to an estimated 80 to 120+ feet along the existing waterfront.

2.4 FAULTS AND SEISMICITY

The project area is located in a seismically active region that is subject to periodic strong to violent ground shaking. Active faults in the area are shown on Plate 5, Earthquake Epicenters and Fault Map.

The San Andreas Fault, which is situated about 12 kilometers (km) southwest of the site, dominates the tectonics, geology, and physiography of the San Francisco Bay region. The Hayward Fault is situated about 16 km northeast of the site. Other major active faults, which could cause significant shaking at the project site, include the San Gregorio, Concord, Calaveras, and Rodgers Creek Faults. Active faults that are pertinent to the site, and historic earthquakes attributed to each fault are listed in Table 1, Historical Earthquakes.

The maximum moment magnitude earthquake (Mmax) is defined as the largest earthquake that a given fault is considered capable of generating. The Mmax on the San Andreas Fault would be a magnitude 7.9 event occurring approximately 12 km from the project site (USGS, 2008). The Mmax on the Hayward Fault would be a magnitude 7.1 event occurring approximately 16 km from the project site (USGS, 2008). The Mmax given for the Hayward Fault is based on a rupture of the entire length of the fault. The seismicity associated with each pertinent fault, including estimated slip rates, is summarized below in Table 2, Active Fault Seismicity.

TABLE 1
HISTORICAL EARTHQUAKES

Date	Magnitude	Fault	Epicenter Area
June 10, 1836	6.5[1], 6.8[4]	San Andreas	San Juan Bautista
June 1838	7.5[1], 7.0[4]	San Andreas	San Juan Bautista
Nov. 26, 1858	6.25[4]	Calaveras	San Jose Area
October 8, 1865	6.3[2], 6.5[4]	San Andreas	South Santa Cruz Mountains
October 21, 1868	7.0[2,4]	Hayward	Berkeley Hills, San Leandro
February 17, 1870	6.0[4]	San Andreas	Los Gatos
April 19, 1892	6.5[4]	Uncertain	Vacaville
April 21, 1892	6.25[4]	Uncertain	Winters
June 20, 1897	6.25[4]	Calaveras	Gilroy
March 31, 1898	6.5[4]	Uncertain	Mare Island
May 19, 1889	6.25[4]	Uncertain	Antioch
April 18, 1906	7.9[3]	San Andreas	Golden Gate
July 1, 1911	6.6[2], 6.5[4]	Calaveras	Diablo Range, East of San Jose
October 22, 1926	6.1[4]	San Gregorio	Monterey Bay
April 24, 1984	6.1[4]	Calaveras	Morgan Hill
October 17, 1984	7.1[4]	San Andreas	Loma Prieta, Santa Cruz Mountains

(1) Borchardt & Toppozada (1996)

(2) Toppozada et al (2000)

(3) Petersen (1996)

(4) Ellsworth, W.L. (1990)

TABLE 2
ACTIVE FAULT SEISMICITY

Fault Name	Distance to Site[1] (km)	Maximum Moment Magnitude[2]	Contributing Segments	Total Slip Rate (mm/year)
San Andreas	12	7.9	SAO+, SAN+, SAP+, SAS+	24 ± 3
Hayward	16	7.1	HN+, HS+	9 ± 2
San Gregorio	18	7.3	SGN+, SGS+	7 ± 3
Calaveras	33	6.8	CN+, CC+, CS+	15 ± 3
Mount Diablo Thrust	34	6.6	MTD	2 ± 1
Concord-Green Valley	41	6.7	CCD+, GV+	4 ± 2
Monte Vista-Shannon	38	6.8	MVS	0.4 ± 0.3
Rodgers Creek	40	7.0	RC	9 ± 2
Point Reyes	44	7.0	PR	1 ± 0.5
West Napa	46	6.5	WN	1 ± 1
Greenville	50	7.0	GN+, GS+	2 ± 1
Great Valley (segment 5)	57	6.5	CVS5	1.5 ± 1
Great Valley (segment 4)	66	6.6	GVS4	1.5 ± 1
Hunting Creek - Berryessa	78	7.1	HCB	6 ± 3
Zayante - Vergeles	83	7.0	ZV	0.1 ± 0.1
Maacama - Garberville	94	7.5	MG	9 ± 2

1. USGS Quaternary Faults and Folds Database (2006).
2. WGCEP (2008), Working Group on California Earthquake Probabilities. Only the highest maximum moment magnitude of all segments reported.

2.5 SUBSURFACE CONDITIONS

As part of the current Phase II Geotechnical Exploration Program, AGS drilled, sampled and logged eleven (11) borings, Borings B-3 through B-13. Previously, AGS (2011) drilled, sampled, and logged two (2) soil borings, Borings B-1 and B-2. The boring locations are shown on Plate 2, Approximate Boring Location Map. Boring B-1 was drilled through a gravel surface at the northwest end of the abandoned Slipway 4. Boring B-2 was drilled east of the existing Crane 30. Borings B-3 through B-6 were drilled near the waterfront. Borings 7 through 13 were drilled in the main yard.

Generalized subsurface profiles along cross sections A-A', B-B', and C-C,' are shown on Plates 6 through 8, respectively. In addition, Table 3 provides some information regarding the elevation and thickness of each stratum in borings for this project. A general description of the soils and bedrock at the project site is presented in the following paragraphs.

TABLE 3
SUBSURFACE CONDITION DATA

Boring	Ground Surface Elevation (feet)	Fill Thickness (feet)	Bottom of Fill Elevation (feet)	YBM Thickness (feet)	Bottom of YBM Elevation (feet)	Top of Bedrock Depth (feet)	Top of Bedrock Elevation (feet)	Encountered Groundwater Depth (feet)
B-1	2	12.5	-10.5	35.5	-46	87.5	-85.5	9
B-2	1	15	-14	24	-38	55	-54	6.5
B-3	-4	18	-22	26	-48	>110.5	<-114.5	7.5
B-4	2	17	-15	43	-58	>105.5	<-103.5	8
B-5	-5.5	15	-20.5	36.5	-57	105	-110.5	8
B-6	1	10	-9	73	-82	>95.5	<-94.5	NM
B-7	1	15	-14	22	-36	50	-49	6
B-8	2.5	15	-12.5	5.5	-18	23	-20.5	7.5
B-9	3	17	-14	NE	NE	43	-40	9.5
B-10	2.5	16	-13.5	26	-39.5	71	-68.5	8.5
B-11	-5	5	-10	5.5(1)	NA(1)	NA	NA	5
B-12	-6	9	-15	2	-17	NA	NA	4
B-13	-3	NA	NA	NA	NA	NA	NA	NE

(1) To the bottom of the subsurface exploration
(2) NA : Not Available
(3) NE : Not Encountered
(4) NM : Not Measured
(5) Holes were not left open at the time of drilling to measure the stabilized groundwater level.

2.5.1 Artificial Fill

Fill materials encountered in the field exploration program consist of loose to very dense gravelly material with varying amount of sand, clay, and silt, and very loose to medium dense sand with varying amount of gravel, clay, and silt. Debris consisting of wood, asphalt pieces, metal slag, and concrete fragments and few cobbles were encountered in portions of the fill. In addition, some burn material, including charcoal and metal slag, as well as some oily material was encountered in Borings B-5, B-7, B-8, and B-11. The thickness of fill generally varied with elevation and ranged from approximately 5 feet (Elevation of -10) thick in Boring B-11 to approximately 18 feet (elevation of -22 feet) thick in Boring B-3. At the grade of the main yard, the fill thickness typically varied from about 15 to 17 feet (Elevation of -12.5 to -14 feet). Table 3 shows the thickness and bottom elevation of fill material in each boring within project site.

2.5.2 Young Bay Mud (YBM)

Except in Boring B-9, the fill material was underlain by very soft, black to dark gray highly compressible, high plasticity clay. The thickness of the YBM varies considerably from approximately two (2) feet (near Illinois Street) to 73 feet in Boring B-6. Table 3 shows the thickness and bottom elevation of YBM in each boring within project site.

2.5.3 Alluvium / Bayside Deposits and Old Bay Mud (OBM)

Fill material was underlain with sandy and gravelly alluvium with varying amount of clay and clayey material with varying amount of sand and gravel that continued to bedrock at a depth of approximately 43 feet (elevation of -40 feet) in Boring B-9. In other borings, YBM was underlain by layers of Alluvium / Bay Deposits and Old Bay Mud (OBM) that continued to depths exceeding about 95.5 feet in the waterfront area near Borings B-3 through B-6. In general, the OBM underlies intermediate alluvial deposits that typically consist of stiff to very stiff or hard lean clay with varying amounts of sand and gravel. The OBM typically consists of soft to stiff clay that is fat, with little sand and little gravel.

2.5.4 Bedrock

The Alluvium / Bayside Deposits and OBM are underlain by bedrock at variable depths across the site. Bedrock was encountered at depths of 23 feet and 43 feet in vicinity of existing Illinois Street in Borings B-8 and B-9, respectively. In the waterfront area near Borings B-3 through B-6, bedrock was encountered greater than 95.5 feet. Table 3 shows the depth and top elevation of the bedrock in each boring within project site. Bedrock underlying the site consists of moderately weak to hard, and moderately to severely weathered sedimentary rock, such as sandstone and shale; or serpentinite.

2.6 GROUNDWATER

As part of the subsurface exploration program, AGS measured groundwater levels inside the open boreholes with a Solinst Water Level Meter, and the recorded water depths varied from about 4 feet (elevation of -10 feet) in Boring B-12 to 9.5 feet (elevation of -6.5 feet) in Boring B-9 below the existing ground surface. Due to proximity to San Francisco Bay, the groundwater level varies considerably as the tides rise and fall. The tidal range measured during drilling was approximately 5 to 6 feet.

Based on a Sea Level Rise and Adaptation study performed by URS in 2010, approximate tide elevations were determined based on City of San Francisco Datum for this project site, as shown in Table 4.

TABLE 4
APPROXIMATE TIDE ELEVATION

	Approximate Tide Elevation*
100-Year Tide	-2.1
500-Year Tide	-1.9
MHHW (Mean Higher High Water)	-5.1
Mean Sea Level	-8.0
MLLW (Mean Lower Low Water)	-11.4

* (City of SF Datum)

Based on Table 4, an appropriate design level for the maximum groundwater table could be the 100-year tide elevation. However, the water table is unlikely to approach this level on a regular basis. Regardless, the water table is anticipated to be regularly shallow according to Seismic Hazard Zone Report 43 (CGS, 2000), which incorporates hundreds of water depths recorded from various agencies dating back to 1913. The depth to groundwater in the area is consistently less than 10 feet below the ground surface. Changes in precipitation and temperature, or other factors that could affect the water quality were not evaluated as part of our study.

This study did not assess contamination of on-site soils and groundwater; however, AGS recommends that appropriate measures be taken to test and properly dispose of any contaminated excavated soils or groundwater encountered at the site.

2.7 CORROSIVITY

Selected soil samples from the borings were transferred to Cooper Testing Laboratory for analysis and evaluation of the corrosivity potential to buried construction materials such as metal and concrete.

Two (2) samples were selected for testing including resistivity, chlorides, sulfate, PH, Redox, and Sulfide. Tests were performed in accordance with California Test Methods 417, 422, and 532. Test results are presented in this report.

3. RECOMMENDATIONS

3.1 GENERAL

Based on the results of our field exploration and laboratory testing program, it is AGS' opinion that the proposed improvement is feasible from a geotechnical point of view, provided the recommendations in this report are incorporated in the design and construction of the project.

Major geotechnical factors that affected the project are the presence of uncontrolled fill materials, highly compressible YBM clays and potentially liquefiable soils above and below YBM layer. The existing very loose to medium dense upper sandy and gravelly fill materials and loose to medium dense sands below the YBM at the site have high liquefaction potential when subjected to a significant earthquake shaking. The consequences of liquefaction include downdrag on pile foundations, settlement of the ground surface and utilities, lateral deformation, development of excess pore water pressure, buoyancy effects on the below groundwater utilities, loss of allowable bearing pressure, and increased lateral pressures on utilities extending below the groundwater table. The results of our liquefaction analyses using the information obtained during Phase II study indicate that the seismically-induced settlements at the site may range from approximately 4 to 10 inches due to a major earthquake event.

It is the opinion of AGS that the existing cranes, proposed overlook structures, and proposed retaining wall may undergo seismically-induced lateral deformations and seismically-induced settlement due to loss of passive and/or base shear resistances. Therefore, some ground improvement program or a proper foundation system is needed to resist settlement and lateral forces induced by a liquefaction event.

Presence of continuous liquefiable materials is the primary criteria for development of the seismically-induced lateral deformation at this site. In Phase I study, AGS could not confirm continuity of the liquefiable layer. Based on the subsurface information obtained during Phase II study, the existing fill materials are potentially liquefiable throughout the project site and will develop seismically-induced lateral deformation during a major earthquake event. The results of our seismically-induced lateral deformation analyses indicate that liquefaction-induced lateral deformations at the site may be up to 3 feet during an earthquake with a moment magnitude of

M7.9 on the San Andreas Fault and may result in damages to the existing and proposed structures, unless some form of ground improvement be implemented at the site and recommendations provided in this report are incorporated in the design and construction of the project.

The existing buildings at the site are founded on shallow foundation system bearing on the existing uncontrolled fill and may experience damage due to loss of bearing capacities, seismically-induced settlements, and seismically-induced lateral deformations.

Existing underground gravity utilities could experience damage or lose their serviceability due to differential seismically-induced settlement at the project site. Maximum differential seismically-induced settlement of 4.5 inches in approximately 400 lineal feet was estimated between Borings B-7 and B-10.

Proposed grading may cause static short-term and time-dependent settlements near the existing structures. Performance of the foundations of the existing buildings and structures should be evaluated based on the results of settlement analyses presented in this report. The implications and potential adverse effects of settlement require consideration of alternative methods such as ground improvement, additional structural support particularly deep foundations, and/or using light-weight fill material.

3.2 SEISMIC DESIGN CONSIDERATIONS

3.2.1 Fault Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Zone (California Geologic Survey, Special Publication 42, 2007). Therefore, the risk from surface fault rupture is considered to be very low.

3.2.2 Maximum Earthquake

The Maximum Moment Magnitude (Mmax) earthquake is the largest reasonable earthquake that a given fault is capable of generating in the current tectonic setting. The controlling Mmax earthquake that could affect the project site would be a magnitude 7.9 seismic event occurring on the San Andreas Fault, with a seismogenic source (focus of seismic energy at depth) located about 12 kilometers southwest from the site. Regional faults and the values of Mmax earthquakes are shown on Table 2. The locations of active faults, as adopted by California Geological Survey (2002), and the epicenters of historical earthquakes in the Bay Area and vicinity are shown on Plate 5, Earthquake Epicenters and Fault Map.

3.2.3 Estimated Earthquake Ground Motions

Ground surface accelerations were estimated using both deterministic methods and probabilistic methods, using the EZ-FRISK™ computer software package Version 7.40.

3.2.3.1 Deterministic Methods

Correlations between distance from a causative fault and mean values of the peak horizontal accelerations and the effects of local soil conditions on peak ground accelerations have been developed for deep alluvial sites through various attenuation relationships. Recent seismic models use the so called Next Generation Attenuation (NGA) relationships. NGA relationships were used to calculate seismic acceleration values at the Crane Cove Park site. In particular, the relationships by Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008) were used to calculate the peak ground accelerations at the project site.

At the time of Phase I study for this site, ASCE 7-10 was published but not adopted by the agencies. Only recently all agencies have started adopting the ASCE 7-10. In Phase I study, deterministic analysis was performed using the mean value of calculated seismic accelerations based on ASCE 7-5. In Phase II study, 84^{th} percentile of the deterministic analysis was performed and used in developing design spectrum.

Based on these correlations, a Maximum Moment Magnitude (Mmax) of 7.9 occurring on the San Andreas Fault System, located approximately 12 km away, is estimated to generate a peak 84^{th} percentile horizontal bedrock acceleration (PGA) on the order of 0.46g at the site. Lesser values of PGA may be used for design based on the level of risk acceptable to the designer.

3.2.3.2 Probabilistic Methods

For the Crane Cove Park site, peak horizontal ground accelerations were developed in accordance with ASCE 7 for the average earthquake return period of 2,475 years, using the NGA relationships developed by Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008). This earthquake return period corresponds to an approximately 2 percent probability of being exceeded in 50 years (2475 years). The estimated average value of peak horizontal bedrock acceleration calculated using probabilistic method from the four attenuation relationships discussed above is 0.61g.

3.2.4 ASCE 7 Seismic Design Criteria

Based on the explored subsurface conditions and the ASCE 7 requirements, seismic design parameters were determined using ASCE 7 procedures as described in Table 5.

TABLE 5
SEISMIC CRITERIA BASED ON ASCE 7

	ASCE 07 Table/Figure/Equation	**Factor / Coefficient / Type based on Site Class E**	**Value**
Site Class	Section 11.4.2		F
Short-Period MCE, g	Figures 22-1	S_s*	1.5
MCE at 1 sec, g	Figures 22-2	S_1*	0.6
Site Coefficient	Table 11.4-1	F_a**	1.0
Site Coefficient	Table 11.4-2	F_v**	2.4

* Based on Site Class B

** Based on Site Class E

3.2.5 Design Earthquake

During report preparation of Phase I, ASCE 7-10 was published but not adopted by the governing agencies. Recently, most governing agencies use ASCE 7-10. In accordance with Section 11.4.7 of ASCE 7, a site response analysis should be performed for structures on Site Class F sites. Therefore, a site response model was developed based on low-strain shear wave velocities, non-linear stress-strain relationships, and unit weights to comply with Section 21.1.2 and 21.1.3 of the ASCE.

In Phase I, the bedrock site specific maximum considered earthquake geometric mean (MCE_G) peak ground acceleration was developed by taking the lesser of the probabilistic analysis for 2,475 event and 150 percent of mean value in accordance with Section 21 of the ASCE 7-5. In Phase II study, the bedrock site specific MCE_G peak ground acceleration was developed by taking the lesser of the probabilistic analysis of 2,475 event and 84th percentile deterministic analysis in accordance with Section 21 of the ASCE 7-10.

Three time-histories were selected and matched to the developed bedrock site specific MCE_G spectrum acceleration. A soil profile was selected for this site and dynamic properties of the soil layers were calculated using laboratory and field data following relationships developed by others. For purpose of this study, the soil profile was developed using the information from Boring B-1 which was judged to approximate the average subsurface conditions at the project site. The upward response of the soil profile to the matched acceleration time histories was calculated using equivalent-linear soil properties in an one-dimensional, ground response analyses and surface ground motion time histories were calculated using SHAKE2000 computer program in accordance with Section 21 of the ASCE 7-10. The average peak horizontal acceleration at the ground surface at the site calculated from the SHAKE analyses was 0.54g. The design spectral response acceleration (S_a) at any period is calculated as 2/3 of the spectral response acceleration obtained using the procedure described above for Site Class F.

The design acceleration response spectra corresponding to a 5 percent structural damping ratio are presented on Table 6 and Plate 9.

TABLE 6
SPECTRAL ACCELERATIONS

Period	Probabilistic Analysis for 5% Damping 2% in 50 Years (Bedrock)	Deterministic Analysis (Bedrock)		Design Response Spectrum (ASCE 7-10 Section 21.3) (S_a)	
		Mean	84th Percentile	Horizontal	Horizontal
(sec)	(g)	(g)	(g)	(g)	(g)
PGA	0.61	0.28	0.46	0.36	0.44
0.05	0.83	0.36	0.61	0.48	0.88
0.10	1.25	0.52	0.90	0.61	0.93
0.20	1.48	0.62	1.09	0.86	0.67
0.30	1.27	0.54	0.96	0.90	0.46
0.40	1.10	0.47	0.83	0.90	0.39
0.50	0.96	0.40	0.72	0.90	0.34
0.75	0.71	0.29	0.54	0.80	0.25
1.00	0.55	0.23	0.42	0.72	0.22
2.00	0.28	0.12	0.22	0.44	0.14
3.00	0.19	0.08	0.15	0.30	0.10
4.00	0.13	0.06	0.11	0.22	0.09
5.00	0.11	0.04	0.09	0.19	0.08
7.50	0.07	0.03	0.06	0.12	.0.05
10.00	0.05	0.02	0.04	0.08	0.04

3.2.6 Vertical Acceleration

AGS recommends that the peak vertical component of the acceleration be taken as equal to two-thirds of the peak horizontal acceleration component discussed in Sections 3.2.4 and 3.2.5.

3.2.7 Liquefaction Hazard

Soil liquefaction is a phenomenon in which saturated (submerged) cohesionless soils lose their strength due to the build-up of excess pore water pressure, especially during cyclic loadings such as those induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements, if not confined. Soils most susceptible to liquefaction are loose, clean, uniformly graded, fine-grained sands. Silty and clayey sands may also liquefy during strong ground shaking.

The nature of liquefaction depends greatly on the characteristics of the soil. In loose soils, liquefaction results in significant loss of soil strength, which can lead to large deformations. In dense soils, although a condition of liquefaction can be initiated, the tendencies for loss of strength and deformations are resisted by dilation of the soils. Deformations in dense soils result in a tendency for soil volume increase (dilation), which in turn results in reduction of pore water pressures, increase in effective stresses, and increase in resistance to further deformations.

The liquefaction potential of soils at the site was evaluated using a simplified, analytical, and empirical procedure that is correlated with the liquefaction behavior of saturated sands during historic earthquakes (Youd, 2001; and Idriss and Boullanger, 2008). The primary data utilized in the analysis consisted of standard penetration test (SPT) and modified California (MC) sampler blow counts, which were obtained from ten (10) borings drilled at the site. The SPT and MC blow counts recorded in the field were corrected for various factors to obtain corrected N-values, which were used in the liquefaction analysis. The factors used to obtain corrected N-values, included the effects of overburden pressure, rod length, sampler type and size, and fines content.

Chapter 11 of ASCE 7-10 suggests that maximum considered earthquake geometric mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_m) be used for evaluation of the liquefaction, seismically-induced lateral deformation, seismically-induced settlement, and other soil related issues. For soil class “F”, AGS calculated PGA_m of 0.36g for the site.

During drilling, the groundwater was measured at depths ranged from approximately 4 feet to 9.5 feet below the existing ground surface. A design groundwater of 2 feet below ground surface was assumed because of the likelihood of the rise and fall in response to the local tides in the future.

The liquefaction analysis was conducted using the following parameters.

- Magnitude 7.9 earthquake
- Peak horizontal acceleration of 0.36g
- Groundwater at 2 feet below ground surface

Based on the results of the liquefaction analysis, all the very loose to medium dense granular material at the site is considered to have a potential for liquefaction. Plates D-1 through D-8 in Appendix D show the liquefiable analysis of the borings.

3.2.8 Consequences of Liquefaction

The main effects of liquefaction at the site include settlement of the ground surface and utilities, lateral deformation, development of excess pore water pressure, buoyancy effects on the below groundwater structures, loss of allowable bearing pressure, and increased lateral pressures on utilities and foundations extending below the groundwater table.

Liquefaction of soils underlying the existing seawalls may also induce temporary buoyant uplift pressures. The impact of the liquefaction on the existing seawall was not part of the AGS scope of work.

3.2.8.1 Seismically-Induced Settlements

As discussed previously, liquefaction of the in-situ, loose to medium dense, saturated fill may occur and would result in liquefaction-induced settlement. The seismically-induced settlement will be in addition to the static settlement.

The liquefaction analyses were performed based on subsurface exploration results. The estimated seismically-induced settlements and the thickness of the liquefiable layers for the borings are presented on Table 7. The estimated seismically-induced settlements range from approximately 4 to 10 inches. It is the opinion of AGS that the maximum differential seismically-induced settlement of 4.5 inches in approximately 400 lineal feet should be anticipated as established by subsurface conditions encountered between Borings B-7 and B-10.

If the anticipated seismically-induced settlements are not acceptable to the designer, AGS recommends that a soil improvement program as discussed in Section 3.2.10, be used to reduce liquefaction consequences to an acceptable level. However, AGS understands that it would not be economically feasible to construct ground improvements to reduce liquefaction effects in large areas and even with soil improvements, the structures could be damaged during a magnitude 7.9 or greater earthquake. Another option to reduce the liquefaction impact on the structures is to support the cranes on a deep foundation system.

TABLE 7
ESTIMATE OF SEISMICALLY-INDUCED SETTLEMENT

Boring ID	Area	Boring Depth (feet)	Potentially Liquefiable Layer Depth Interval (feet)	Thickness of Potentially Liquefiable Layer (feet)	Estimated Total Seismically -Induced Settlement (inch)
B-3	Urban Beach	110.5	2 to18 44 to 50	22	7.5
B-4	Urban Beach	105.5	2 to10	8	4
B-5	Maritime Field	108.5	2 to15	13	5.5
B-6	Maritime Field	95.5	2 to10	8	1
B-7	109+110 Area	53	2 to15 46 to 50	17	5.5
B-8	Slipway 4 (Keel Park) /19th Street Extension	29.5	2 to15 21 to 23	15	5
B-9	Open Green	49.3	2 to 25	23	10
B-10	Open Green	74.5	2 to16 42 to 50	22	9.5

3.2.8.2 Seismically-Induced Lateral Deformation

Seismically-induced lateral deformation is another phenomenon which can occur during a seismic event. AGS evaluated the potential for lateral deformation of the soil using empirical relationships developed by Youd et al. (2002) and Zhang (2004). These relationships incorporate the thickness of the liquefiable layer, the magnitude and distance of the earthquake from the site, the slope of the ground surface, and boundary conditions, such as a free face, to estimate the horizontal ground movement.

The continuity/discontinuity of potentially liquefiable soil layers is a key consideration in evaluating the potential for seismically-induced lateral deformation. Potentially liquefiable uncontrolled liquefiable fill materials consist of loose to very dense gravelly material with varying amount of sand, clay, and silt, and very loose to medium dense sand with varying amount of gravel, clay, and silt was encountered all over the project. Some potentially liquefiable material was observed below the YBM layers at this site. In general, the bottom depth of the potentially liquefiable fill material above YBM varied from 10 feet to 28 feet within the project site. It should be noted that for a significant areal lateral deformation to occur, a continuous layer of potentially liquefiable soil extending for a considerable distance (on the order of several hundred feet) is required.

During seismically-induced lateral deformation, surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial soil is transported downslope or in the direction of a free face by earthquake and gravitational forces.

For our analysis, AGS used two different boundary conditions; one with free face (such as a situation where the seawall exists but has been broken during earthquake shaking) and one with gentle sloping ground (such as in front of seawall and in the area where no seawall existed). For free face boundary condition, we assumed a free face height of 12 feet. For sloping ground condition, AGS assumed a sloping ground gradient of 0.1 percent.

The results of the liquefaction-induced lateral deformation analysis based on the predictive relationships for liquefaction-induced lateral deformation is about 3 feet for the Borings B-3 and B-4 which are within 100 feet of the free face boundary and is less than 1 foot for the Borings B-7 and B-8 which are located near the proposed 19th Street extension.

Occurrence of seismically-induced lateral deformation would result in additional loads on pile foundations. The lateral load from the unliquefied crust layer may often be the critical load for the integrity of the pile because of its large magnitude and unfavourable position as a “top-heavy” load acting above a laterally unsupported portion of the pile in the liquefied soil. Cubrinovski and Ishihara (2007) suggested two loading conditions from the crust on piles subjected to seismically-induced lateral deformation. They suggest three (3) times Rankin passive pressure (equivalent unit-fluid weight of 300 pcf above the groundwater) for active loading where the mobilized earth pressure provides the resistance force. They also suggest 4.5 times Rankin passive pressure for passive loading where the mobilized earth pressure provides the driving force and materials in front of the piles move away from the piles. Under this condition, the piles should withstand passive forces without shearing off or deforming excessively. AGS recommends that ultimate lateral pressure from 4 feet of the crust layer be approximated as being 4.5 times the Rankine passive pressure applied over the width of pile cap. If this load cannot be tolerated, the designer may consider construction of a 5-foot deep buffer trench upstream of the crane structures filled with compressible materials to absorb soil deformation.

3.2.9 Seismically-Induced Lateral Earth Pressures

Horizontal accelerations during seismic events will momentarily increase lateral earth pressures on underground structures such as retaining walls. We recommend using an equivalent seismically-induced earth pressure with a rectangular pressure distribution F x H psf, where F depends on the magnitude of the ground acceleration and H is the height of the underground wall in feet measured from the bottom of the slab to the adjacent grade. The resultant seismic force would act at 0.5 H above the base of the wall. The seismic earth pressures are in addition to the static earth pressures and should be considered in the design of the underground structures.

For the peak ground surface acceleration of 0.36, the seismically-induced lateral earth pressure is about 14H. The F value is applicable for both saturated and unsaturated conditions. Also, the F value is applicable for level backfill conditions only (gentler than 6H:1V, horizontal to vertical).

If retaining walls taller than 7 feet are not designed for seismically-induced earth pressures, consequences during strong earthquake loading might include lateral movement of, or distress to, the walls.

The magnitudes of seismically-induced earth pressures were calculated based on the simplified procedure developed by Seed and Whitman (1970) and incorporated a reduction factor on the order of 20 percent to judgmentally account for possible effects of wave scattering or passage, the transient nature of earthquake ground motions, and possible wall-soil interaction effects.

3.2.10 Liquefaction Mitigation

3.2.10.1 General

The existing and structures are immediately underlain by uncontrolled fill and potentially liquefiable soils. Some settlements and cracking may result from differential seismically-induced settlements of liquefiable soils during an earthquake and could damage the existing structures. Seismically-induced settlements of up to 10 inches and seismically-induced lateral deformations up to 3 feet were estimated within the project site and along the existing seawall. Evaluation of deformation of seawall was not part of scope of work for this project. Therefore, effect of the existing or improved seawall was ignored in the estimation of seismically-induced lateral deformation for this project. A mitigation plan should be employed where estimated seismically-induced settlements or seismically-induced lateral deformation cannot be tolerated.

Ground improvement can be performed in areas where the total calculated seismically-induced settlement exceeds the structurally acceptable level, and be designed to reduce the total liquefaction-induced deformation to a tolerable level. Ground improvement of liquefiable materials blow YBM which is a costly operation will reduce the seismically-induced settlement to a lower desirable value to the design team. Based on AGS's borings, seismically-induced settlement will reduce to less than 4 inches with differential seismically-induced settlement of up to 1.5 inches along 400 lineal feet if the fill materials which are above the YBM are improved. The total thickness of the zone to be improved depends both on the actual thickness of the potentially liquefiable material and the desired reduction in predicted settlement.

Alternatively, the existing structures may be supported on deep foundations as discussed in Section 3.4.

3.2.10.2 Soil Improvement

This section provides options for liquefaction mitigation through a program of ground improvement. Due to presence of coarse gravel and construction debris within the existing fill materials, pre-drilling may be required.

Some available techniques for soil improvement which may be applicable to this site include vibro-replacement stone columns, grouting techniques, and dynamic deep compaction.

The vibro-replacement stone column technique of ground treatment, in which a vibrator penetrates to depth by means of its weight and vibrations and horizontal vibrations are generated at treatment depth by the use of eccentric weights that are rotated by means of electric motors, has proven successful in reducing the liquefaction potential of sands and low plasticity silt. Stone columns are used for loose silty sands having greater than about 15 percent fines. Cohesive, mixed and layered soils generally do not densify easily when subjected to vibration alone, therefore, the vibro-replacement stone column technique was developed specifically for these soils, effectively extending the range of soil types that can be improved with the deep vibratory process.

Grouting techniques (compaction, permeation, deep mixing, chemical, and jet grouting) of soil improvement have also proven successful in reducing the liquefaction potential of sandy material. The grouting techniques become less efficient with increased fine content, such as silt and clay. Of these grouting techniques, jet grouting appears to be the most efficient method for the site. Essentially, in jet grouting, ultra high pressure fluids or binders are injected into the soil at a high velocity. These binders break up the soil structure completely and mix the soil particles in-situ to create a homogeneous mass which in turn solidifies. Other grouting techniques, such as deep mixing, involve the use of large augers both to introduce cement grout and to mix it with the soil, producing a treated soil cement column.

Dynamic deep compaction can densify and reduce the liquefaction potential of sandy soils. This method becomes less effective with high groundwater level and increased fine content in soils, but has relatively lower cost compared to other methods. However, due to the effects of vibrations on the adjacent properties, AGS believes that this method is not applicable for this site.

The soil improvement design, if chosen as an economically feasible, will depend on the costs of performing the work as well as the technical specifics of the work, and is beyond the scope of this study.

The practical applications of many of these measures have been presented in the literature (Hryciw 1995; Stewart et al. 1997; Boulanger et al. 1997; Mitchell et al. 1998b) and summarized in Table 8.

TABLE 8
SUMMARY OF LIQUEFACTION MITIGATION TECHNIQUES

Liquefaction Mitigation Technique		Advantages	Disadvantages	Relative Cost
Vibro-Replacement Stone Column		Effective and economical method in many situations. Able to reach depths unattainable by other methods.	Ineffective for densifying soils with greater than 20% fine contents. The liquefiable soil should have a minimum thickness for this method to be effective. Waste spoils disposal is required.	Moderate
Grouting	compaction grouting	Pinpoint treatment, Speed of installation, Wide applications range. Can be performed in very tight access and low headroom conditions, Non-hazardous, no waste spoil disposal. Able to reach depths unattainable by other methods.	Not effective at depths with low confining pressure (<15 feet). Ground surface heave due to grout pressure. Very low reinforcing effects of the compaction grout bulbs/columns.	Low to moderate
	deep mixing grouting	Wide applications range (even with high fine contents), Cost savings over deep foundation designs. Installation methods are customized for the site conditions.	Waste spoils disposal is required. Significant overhead clearance is required. Pinpoint treatment is not applicable. Very low reinforcing effects of the compaction grout columns.	High
	permeation grouting	Minimum disturbance of the native soil. Can be performed in very tight access and low headroom conditions. Pinpoint treatment.	Construction process is complex. Very costly. limited to clean sands and ineffective in soils with fines.	High
	chemical grouting	Minimum disturbance of the native soil. Can be performed in very tight access and low headroom conditions. Pinpoint treatment.	Construction process is complex. Very costly. limited to clean sands and ineffective in soils with fines.	High
	jet grouting	Nearly all soil types groutable. Most effective method of direct underpinning of structures and utilities. Safest method of underpinning construction. Ability to work around buried active utilities, can be performed in limited workspace, treatment to specific subsurface locations, no harmful vibrations. Much faster than alternative methods.	Soil erodibility plays a major role in predicting geometry, quality and production. Cohesionless soils are typically more erodible than cohesive soils. Pinpoint treatment is not applicable. Very low reinforcing effects of the compaction grout bulbs/columns.	High

3.2.11 Effect of Seismically-Induced Lateral Deformation on Utilities

Based on the results of seismically-induced settlements and lateral spreading analyses, the site is identified as hazard zones which have the potential of underground utility breaks due to ground movements. Pease and O'Rouke (1995) performed a study of liquefaction-induced damages including pipe damage. Based on their findings, there is a strong link between pipeline damage and differential settlement. They concluded that differential settlement and angural distortion were most heavily concentrated at the boundary of among the fills, natural soils, and old seawalls. Their study could not find a strong correlation between repair rate and the magnitude of settlement. Additionally, ground strains can result in axial compressive and tensile strains in a buried pipeline. We are not aware of the materials used for each utility lines. Using the calculated seismically-induced lateral deformations for the site, the tensile strains for different lengths of pipes range from 5 to 300 microstarins which are less than acceptable tensile strain of 500 microstrains for cast-iron pipes. However, the concrete pipes may break or disconnect from each other during axial compression or tension.

It is our opinion that potential liquefaction and seismically-induced lateral deformation will cause damage to the buried utilities. Mitigation plans should be employed to reduce the risk of the anticipated damages or repair should be anticipated after a liquefaction event.

3.3 EXCAVATION AND EARTHWORK

3.3.1 Site Preparation

Prior to the site grading, footings, pavements, concrete members, and woods should be removed and debris should be properly disposed of outside the construction area. Existing above and underground utilities located within the proposed construction areas, if affected by construction activities, should be relocated prior to excavation. Debris generated from the demolition of underground utilities, including abandoned pipes, should be removed from the site as construction proceeds.

The existing cranes and associated rails will remain in the site. The piping, electrical conduit, and other appurtenances related to the cranes are to be deconstructed and transported off-site for recycling and disposal.

3.3.2 Fills and Backfills

Fills and backfills may either be structural or nonstructural. Structural fills and backfills are defined as providing support to foundations, slabs, pile cap, and pavements. Nonstructural fills and backfills include all other fills such as those placed for landscaping, and not planned for future structural loads. Structural fills and backfills should be compacted to at least 95 percent of the maximum dry density per ASTM D-1557; nonstructural fills and backfills should be compacted to at least 90 percent of the maximum dry density as determined per ASTM D-1557.

Structural fill and backfill materials should be placed in lifts not exceeding approximately 8 inches in loose thickness, brought to near-optimum moisture content and compacted using mechanical compaction equipment. Nonstructural fills and backfills may be placed in lifts not exceeding 12 inches in loose thickness and compacted in a similar manner.

The fill material should be placed and compacted under the full time inspection and testing of the project geotechnical engineering firm. Material to be used as compacted fill and backfill should be predominantly granular, less than 3 inches in any dimension, free of organic and inorganic debris, and contain less than 20 percent of mostly non-plastic fines passing the No. 200 sieve. The fill and backfill soils should have a liquid limit less than 35 and plasticity index less than 12. Samples of fill and backfill materials should be submitted to the geotechnical engineer prior to use for testing to establish that they meet the above criteria.

It is AGS' opinion that the existing uncontrolled fill may be suitable to support the slab-on-grade and pavements in this project site if the requirements mentioned above are met. The uncontrolled fill below the proposed slab-on-grade and pavement should be excavated up to 2 feet below the bottom of the slab-on-grade and pavement and replaced by structural fill materials.

3.3.3 Temporary Excavations

All excavations must comply with the current requirements of OSHA or Cal-OSHA, as applicable. Additionally, all cuts deeper than 5 feet should be sloped or shored. Shallow excavations above the groundwater level may be sloped if space permits. It is our opinion that temporary excavations may be sloped at 1H:1V (horizontal to vertical) above and 1½ H:1V (horizontal to vertical) below the groundwater level, respectively. The groundwater is estimated to be as shallow as 2 feet below the existing ground surface; however, it is the responsibility of the contractor to maintain safe and stable slopes or design and provide shoring during construction. Flatter slopes will be required if clean or loose sandy soils are encountered along the slope face.

Heavy construction equipment, building materials, excavated soil, and vehicle traffic should not be allowed within 7 feet of the top of excavations.

3.3.4 Permanent Slopes

The recommended inclination of permanent slopes depends on the nature of the materials forming the slope. Slopes up to 20 feet in height consisting of clayey and sandy soils should not exceed an inclination of 2H:1V.

Slopes may experience severe erosion when grading is halted during rainy weather. Before work is stopped, a positive gradient away from the slopes must be established to carry the surface runoff water away from the slopes to areas where erosion and sediment can be controlled.

After completion of the slope grading, erosion protection must be provided. Slope planting, preferably with deep-rooted native plants, must be completed on all exposed surfaces of fill slopes. Graded slopes should not be left exposed through a winter season without the completion of erosion control measures and slope planting.

3.4 FOUNDATIONS

3.4.1 General

AGS evaluated lateral and axial capacity of the foundation system for the existing cranes in the Phase I of this study. In addition, AGS performed analyses to evaluate the resistance of the existing foundation system against the liquefaction-induced pressures due to an earthquake event in that study. AGS's evaluation was based on limited boring data obtained in 2011 and AGS' assumptions regarding the size and length of the timber piles. AGS assumed that 45-foot long; 12-inch diameter timber piles spaced 5 feet by 6 feet are supporting the existing cranes and Slipway 4. AGS recommended that if liquefaction-induced lateral deformation and downdrag forces due to seismically-induced settlements cannot be tolerated by the existing timber piles, either soil improvement should be performed or the existing cranes and proposed structures and retaining walls should be founded on a new deep foundation system. The lateral load from the non-liquefied near surface crust layer may often be the critical load for the integrity of the piles because of its large magnitude and unfavorable position as a top-heavy load acting above laterally unsupported portion of the pile in the liquefiable soil. The Structure Engineer for the project determined that the existing timber piles supporting the crane will likely shear off due to lateral load during a seismically-induced lateral deformation event. In the Phase I study, AGS recommended a new foundation system to improve the foundation of the existing cranes which were supposed to remain at their current locations. Several foundation options with different diameters were evaluated in the Phase I of this study.

Based on the Master Plan dated December 2013, the park plan includes configuration of the existing cranes, currently located at east and west runways, construction of two (2) new overlook ramp structures, a retaining wall at the southern part of the western runway within the project site. An axial compression load of 120 kips and lateral loads of 25, 35, 50, and 75 kips were estimated by the project Structural Engineer for analyses of each pile for the existing cranes. No loading information was available for the foundation design of the proposed overlook ramp and retaining wall.

Based on the additional subsurface data and provided loading information, AGS evaluated axial capacity and lateral resistance against the liquefaction-induced pressures for a new foundation system for the cranes, proposed overlook ramp structures, and proposed retaining wall.

Cast-in-Drilled-Hole piles (CIDH) may be used to support proposed structures and existing cranes. Driven piles will generate vibrations and may damage the adjacent existing structures such as The Ramp Restaurant and existing seawalls which are located less than 50 feet from the proposed foundation. In addition, driven piles may reach refusal during installation if they encounter the existing timber piles. Unless the layout and details of the existing timber piles are known, AGS recommends using CIDH piles. According to the general state-of-practice for construction of CIDH below groundwater, diameter of the CIDH must be at least 24 inches. To provide the maximum lateral resistance needed during a seismic event, AGS recommends using 30-inch or 36-inch diameter CIDH piles.

3.4.2 Proposed Cast-in-Drilled-Hole Piles (CIDH)

AGS recommends using 30-inch or 36-inch diameter CIDH piles to support the existing cranes and 24-inch diameter CIDH piles to support the proposed overlook structures and proposed retaining wall. The CIDH piles should be extended to the recommended depths presented in this report.

The allowable compression and uplift capacity developed by friction between the soils and the CIDH piles are presented on Table 9 based on the subsurface conditions. The allowable compression capacity includes estimated downdrag loads due to static and seismically-induced settlement; therefore, no additional reduction is necessary to account for downdrag. It is AGS' understanding that no grading change is planned near the cranes and overlook ramp structures. Therefore, no additional downdrag load due to additional fill material was considered for axial capacity calculation of CIDH piles for cranes and overlook ramp structures. However, the proposed retaining wall will experience downdrag force due to the settlement generated by additional fill material. A factor of safety of two (2) was used to calculate the allowable compression and uplift capacity of the CIDH pile. The allowable capacities may be increased by one-third for momentary loading due to wind or seismic forces.

TABLE 9
ESTIMATED LENGTHS AND ALLOWABLE CAPACITIES OF DEEP FOUNDATION SYSTEM

Structure	Pertinent Borings	Estimated Top of Bedrock Depth (feet)	Minimum Required Length of Deep Foundation (feet) (1)	Maximum Allowable Axial Capacity (Kips)	Maximum Allowable Uplift Capacity (Kips)	Maximum Downdrag Load(2) (Kips)
Overlook Ramp 1	B-3	>110.5	65	45	25	40
Overlook Ramp 2	B-5	105	75	70	40	15
Crane 14	B-4	>105.5	100	140	80	25
Crane 30	B-7	50	55	130	75	40
Retaining Wall	B-10	23 to 74.5	60(3)	20	12	40

(1) From existing ground surface
(2) Due to static settlement and liquefaction
(3) If bedrock encountered shallower than 60 feet, the pile should be installed minimum 5 feet into the bedrock.

3.4.3 Resistance to Uplift in Deep Foundations

Uplift capacities of the proposed CIDH piles are presented on Table 9. The recommended allowable uplift capacities do not include the effect of the weight of piles. The buoyant weight of the piles should be added to the recommended uplift capacities to estimate total allowable uplift capacities. The uplift is based on the resistance capacities of the soils; the structural tension capacity of the piles should be checked by the project structural engineer.

3.4.4 General CIDH Pile Recommendations

The structural capacities of the CIDH Piles depend on the strength of the materials used, which should be checked by the project Structural Engineer.

Spacing should be determined as required by the load distribution, but minimum spacing between the proposed CIDH piles should not be less than 3 times the diameter of piles, center-to-center. Maximum spacing is to be determined by the Structural Engineer.
The allowable capacities of new piles should be reduced by group action when the new piles are spaced closer than what specified above, and where this occurs additional geotechnical analyses will be necessary.

CIDH piles should be constructed in accordance with generally accepted engineering practices, such as recommendations of the American Concrete Institute and Deep Foundation Institute.

Groundwater will be encountered in the excavations for CIDH pile construction and in this event casing will be required. The casing should be removed as the concrete is placed. Continuous vibration of the concrete or casing (or other methods) may be required to reduce the potential for the concrete to "hang up" on the casing. Where hang up occurs, unacceptable voids could result in the poured concrete mass.

Any loose soil and cuttings at the bottom of the drilled holes should be removed. Up to maximum of 2 inches of loose soil is allowed to remain at the bottom of the drilled holes.

For drilled excavations below the water table, the steel reinforcement should be set and the concrete tremied or pumped immediately upon completion of the holes. A minimum 3-inch space should be maintained between the steel cages and the sidewalls of the holes prior to placing the concrete.

To minimize aggregate separation, concrete used for the CIDH piles should be tremied or pumped into the drilled holes. Under no circumstances should concrete be allowed to free fall more than 15 feet in a dry hole.

3.4.5 Resistance to Lateral Load

3.4.5.1 Pile Cap

The ultimate coefficient of friction between poured-in-place concrete pile cap and the underlying structural fill may be taken as 0.35. Passive pressure available in compacted backfill may be taken as equivalent to the pressure exerted by a fluid weighing 300 pcf above groundwater level and 100 pcf below groundwater under liquefiable condition. The above-recommended value includes a calculated factor of safety of at least 1.5; therefore, frictional and passive pressure resistance may be used in combination without reduction.

3.4.5.2 CIDH piles

Resistance to lateral loads for structures supported on deep foundation systems will be provided by passive soil pressure against the pile and by the bending resistance of the piles. Lateral deformation, shear forces, and bending moments curves were developed for each structure using the L-pile computer program. Fixed head CIDH piles, 30-inch and 36-inch in diameters, with axial compression load of 120 kips and lateral loads of 25, 35, 50, and 75 Kips were used to develop the lateral analysis data for existing cranes. Lateral load analyses were performed for proposed overlook ramp and the proposed retaining wall assuming the axial compression load of 20 kips and lateral loads of 5, 10, 15, and 20 Kips for a 24-inch fixed head pile. The lateral capacities and bending moments for these piles were computed considering liquefaction occurs.

The results of the lateral pile deformation analyses are presented on Plate 10 through Plate 14 and include no factor of safety.

3.4.6 Pile and Pile Cap Corrosion Potential

Based on the Caltrans Corrosion Guideline, the corrosivity test results on the soils, as presented in Appendix C, indicated that the uncontrolled fill material in Boring B-4 is "non-corrosive". However the result of corrosivity testing on uncontrolled fill material in Boring B-5 indicated a potential of "corrosive" material. Therefore, some corrosion protection measures may be required for the concrete pile caps.

Based on our experience, YBM below the fill material is "corrosive". The corrosive Younger Bay Mud may adversely affect the reinforced CIDH piles. The adverse effects may be mitigated for CIDH piles by the use of sulfate-resistant high density/low porosity cement, and providing a good coverage of mortar over the concrete pile reinforcements.

We recommend that a Corrosion Engineer be consulted for the development of site-specific corrosion protection measures.

3.5 STATIC SETTLEMENT

3.5.1 General

It is anticipated that static settlements will occur as a result of the proposed grading within the project site. The majority of the static settlements will be time-dependent and will result from consolidation of the YBM. The magnitude and time rate of the static settlements will depend upon the thickness of the proposed fill and the YBM. The primary consolidation of the YBM underlying the site is expected to be complete and negligible under its own weight and the weight of the existing fill and existing structures.

AGS estimated static settlement in different areas within project site based on the proposed grading plan provided to AGS by the project Civil Engineer. The existing uncontrolled fill materials within the project site may experience static settlement up to 1.5 inches due to the proposed grading of the ground. Based on the nature of the fill materials encountered in this site, AGS anticipate that the majority of the settlement occurs immediately during or shortly after the construction.

AGS estimated the primary and secondary consolidation settlement along with required time for 50 percent and 90 percent of the primary static settlement to happen, as shown on Table 10.

It is AGS' anticipation that the proposed grading will impact the road, sidewalk, and foundation of the existing retaining wall along the Illinois Street. Therefore, minor maintenance of the road and sidewalk is anticipated. AGS does not have any information regarding the foundation of the existing retaining wall. If the retaining wall is supported on shallow foundation, damage to the wall due to settlement is anticipated. In the case that the existing retaining wall is supported on deep foundation, the downdrag load generated by static settlement and liquefaction should be considered by Structure Engineer of the project and the integrity of the existing foundations should be verified.

Performance of the foundation of the existing buildings should be evaluated due to differential settlement generated by the proposed grading within the project site.

Design finish grades should be adjusted accordingly to compensate for additional settlement. If the estimated static settlements presented on Table 10 are not acceptable, AGS recommends that the proposed grading plan be adjusted to limit the lateral extent and thickness of the additional fill.

3.5.1 Settlement Mitigation

AGS recommend the placement of geogrid at the bottom of the proposed fill to reduce the magnitude of the differential settlement. Surcharging with or without wick drains may be used before the construction to reduce the amount of time-dependent settlements. Alternatively, light-weight fill (geofoam or light weight aggregate fill) may be used in lieu of the regular fill to reduce the settlement.

Geofoam is manufactured into large lightweight blocks. The blocks vary in size but are often 6 feet x 2.5 feet x 2.5 feet. The primary function of geofoam is to provide a lightweight void fill to minimize the damage to the existing retaining wall and sidewalk along Illinois Street due to settlement below a highway, bridge approach, embankment or parking lot. The minimum required thickness of geofoam and type of the geofoam should meet the requirements of NCHRP Report 529 (2004).

Light-weight fill materials may be used within 5 feet of the existing retaining wall face and under the proposed 19th Street extension.

If geofoam is used, AGS recommends placing the geofoam wrapped by a geomembrane to about 1 foot below the final grade. Base materials may be placed on top of the geomembrane. The geofoam should be treated against insect and fire hazard.

TABLE 10
ESTIMATED STATIC SETTLEMENT OF YOUNG BAY MUD

Area	Pertinent Boring	Proposed Fill Thickness (feet)	Primary Settlement (inch)	Required Time for Settlement (Years) (1)	
				50%	90%
Urban Beach (between existing Ramp Restaurant and existing Building 49)	B-4 & B-1	1	1	3-4	15-20
		3	2	4-6	15-25
		5	5	4-7	15-30
		8	10	5-10	15-50
Open Green (South of Existing Building 49)	B-10	1	1	3-4	15-20
		3	3	4-6	15-25
		6	9	5-10	15-50
Slipway 4 (Keel Park) / Proposed 19th Street Ramp	B-8	1	1	3-4	15-20
		3	2	4-6	15-25
		6	4	4-7	15-30
Maritime Field	B-6	1	1	3-4	15-20
		3	3	4-6	15-25
		6	4	4-7	15-30
109+110 Area	B-2	1	1	3-4	15-20
		3	2	4-6	15-25
		5	3	4-6	15-25

(1) Variation in required time for 50 percent and 90 percent settlement is due to the possible presence of sandy lenses within YBM which create shorter drainage path and faster settlement rate.

3.6 SLAB-ON-GRADE

AGS understands that the concrete platforms and pedestrian paths near shorelines will be constructed as slab-on-grade structures. The existing uncontrolled fill material ranges from about approximately 5 to 18 feet below ground surface. The existing fill is not suitable to support the slab-on-grade and should be excavated up to 2 feet below the bottom of the slab-on-grade and replaced by structural fill materials. Structural fill materials should be compacted to 95 percent of the Relative Density (RD) per ASTM D1557 criteria.

For the proposed slab-on-grades, it is recommended that a coefficient of subgrade reaction of 80 pounds per cubic inch be used for design of the slab-on-grade. This value of subgrade reaction is based on immediate, elastic settlement estimates.

All slab-on-grade subgrade should be observed by the Geotechnical Engineer and any weak or disturbed areas should be replaced by structural fill material.

3.7 RETAINING STRUCTURE

It is AGS' understanding that the southern part of the west runway will be raised in grade ranging from approximately 1 to 6 feet. A retaining wall is required to support the elevated area. If no movement is allowed at the top of the retaining wall, at-rest earth pressures need to be resisted. If the wall is allowed to deflect outward at the top at least 0.005 H, where H is the wall height, it may be designed to resist active pressures. Plate 15, Lateral Pressures, presents the recommended static lateral pressures assuming both at-rest and active conditions.

In addition to the above pressures, walls must be designed for anticipated surcharge loads. Plate 16, Lateral Surcharge Pressures, Point and Line Loads, and Plate 17, Lateral Surcharge Pressures, Areal Loads, may be used to calculate the lateral surcharge pressures on restrained retaining walls (at-rest case). About 50 percent and 35 percent of any uniform areal load surcharge placed at the top of the retaining wall will act as a uniform horizontal pressure along the entire height of the retaining wall for at-rest and active conditions, respectively. For unrestrained retaining walls (active case), the values from Plates 16 and 17 may be reduced by one-third. Traffic loads can either be considered concentrated loads or converted to an equivalent uniform areal load. Where traffic surcharge loads cannot be accurately anticipated, a minimal uniform lateral surcharge pressure of 150 psf could be used for the full height of the retaining walls.

The above recommended values do not include lateral pressures due to hydrostatic forces. Therefore, a drainage system is recommended behind retaining walls to reduce the risk of over-loading due to hydrostatic pressure buildup. The drainage system should consist of a drainage blanket of Class 2 permeable material with a collector pipe placed at the base behind the wall as described below. Alternatively, a prefabricated synthetic drainage system such as a Miradrain 6000 or equivalent may be used instead of Class 2 permeable material. A third alternative would be to design the walls for lateral pressures including hydrostatic pressures.

The drainage blanket alternative, if used, should consist of a 6-inch-diameter perforated pipe surrounded by Caltrans Class 2 permeable drain material, described in Table 11, Class 2 Permeable Drain Material. This material should be compacted to at least 90 percent relative compaction, and should be encapsulated in filter fabric.

TABLE 11
CLASS 2 PERMEABLE DRAIN MATERIAL

Sieve Size (U.S. Series)	Percentage Passing
1 inch	100
3/4 inch	90 – 100
1/2 inch	40 – 100
No. 4	25 – 40
No. 8	18 – 33
No. 30	5 – 15
No. 50	0 – 7
No. 200	0 – 3

The zone of drain material should be at least 12 inches wide and should extend to the bottom of concrete slabs or pavements. The perforated pipe along the base of the wall should be placed with the perforations down and should drain by gravity to a suitable discharge location.

To prevent excessive lateral forces from being applied to walls, heavy compaction equipment should not be allowed within about seven feet from the tops of the walls. The backfill directly behind the walls should be compacted using small equipment such as self-propelled vibrating plates or rollers.

Consideration should be given to designing underground walls to resist seismically-induced earth pressures.

3.8 PROPOSED FLEXIBLE PAVEMENT DESIGN

It is AGS' understanding that the existing 19th Street is planned to be extended as an access road to the proposed park. In addition, a parking lot located at west of the existing Building 109 and pedestrian paths are planned to be constructed within the project site. R-value test results were obtained on upper 5 feet sample from Boring B-8, in vicinity of the proposed 19th Street extension and proposed parking lot. If existing on-site material is not used, soil samples from import materials should be obtained and R-value tests should be performed on the import materials to properly design the pavement section in the elevated sections. The result of laboratory testing on the on-site soil samples shows an R-value of 54. Due to heterogeneity of the existing uncontrolled fill, pavement section design was performed using an R-value of 30 which is less than the measured R-value of 54. Traffic Index (TI) of 4.5, 7.5, and 10.5 were assumed for this project. The pavement section design is shown in Table 12. Caltrans Highway manual Guideline was used to design the pavement section.

TABLE 12
FLEXIBLE PAVEMENT SECTION DESIGN

	Pavement Section (inch)		
	TI=4.5	TI=7.5	TI=10.5
Asphaltic Concrete (AC)	4	6	7
Aggregate Base (AB) (Minimum R-Value = 78)	6	12	20
Full Depth AC	6	10.5	15

The pavement components should be designed and constructed using the latest Caltrans Standard Specification and procedures.

It is AGS' recommendation that the uncontrolled fill below the proposed pavement should be over-excavated up to 2 feet below the base rock and replaced by structural fill materials. The structural fill should be compacted to 95 percent of the ASTM D1557 procedures. The base rock should meet the requirements of Caltrans Class 2 Aggregate Base compacted to at least 95 percent of the ASTM D1557 procedures.

3.9 CORROSIVITY TESTING

Based on the Caltrans Corrosion Guideline, the result of the corrosivity testing on the select soil samples indicated that the soil in Boring B-4 is "non-corrosive". However the result of corrosivity testing in Boring B-5 indicated "corrosive" potential.

Geotechnical studies do not routinely provide corrosion mitigation measures. A Corrosion Engineer should be consulted to evaluate the effects of the corrosive soils and to provide mitigation procedures alternatives. The Geotechnical Consultant should be aware of corrosion mitigation requirements for various structures.

3.10 EROSION MITIGATION MEASURES

It is AGS' understanding that the existing cove in the northern shoreline area will be removed and a crescent-shaped sloped partially covered with riprap will be constructed in that area.

Placement of riprap is a long-term solution for erosion mitigation and protection measure of the slope.

Riprap should be placed properly over the area of the existing slope to protect the erodible soils from the tidal forces due to the wave action of the bay. Placement of filter material wrapped in filter fabric will prevent the migration of the fine materials from the slope into the riprap.

A keyway should be constructed along the toe of the riprap. The keyway should be at least 7 feet wide, and extend at least 2 feet below the ground surface; however, the actual width and depth should be determined by the geotechnical engineer or geologist during grading.

For riprap and filter material wrapped in filter fabric, it was assumed that the maximum wave height at the reservoir is about 2 feet. Based on the design criteria for riprap (NAVFAC DM-7.1), the maximum rock size should have a weight about 500 pounds. Rock material and grading should be in accordance with Section 72 of the latest Caltrans Standard Specification using Class ¼ -ton riprap and Method B of placement.

In addition, based on the NAVFAC DM-7.1, the rock should meet soundness and density requirements for concrete aggregate under the extreme temperature changes; otherwise any un-weathered rock with specific gravity more than 2.6 is suitable. Argillaceous type and calcium based rocks are not suitable.

Fabric should be UV protected and durable against ground pollutants, insect attack, and penetration by burrowing animals.

All filter fabrics should meet the minimum average roll values shown in Table 13 unless otherwise specified by AGS.

Unless a water-break system is constructed, sand material placed on the beach will wash away due to the wave actions. Replacing the washed-away sand should be considered as part of the maintenance program.

TABLE 13
FILTER FABRIC SPECIFICATION

	Minimum Average Roll Values
Grab Strength (ASTM D-4632)	180 lbs
Mass Per Unit Area (ASTM D-4751)	6 oz/yd2
Apparent Opening Size (ASTM D-4751)	80-100 U. S. Std. Sieve
Flow Rate (ASTM D-4491)	80 gal/min/ft2
Puncture Strength (ASTM D-4833)	100 lbs

3.11 SLOPE STABILITY EVALUATIONS

A Senior Engineering Geologist and a Senior Geotechnical Engineer form AGS performed a site reconnaissance to evaluate the current stability of the shoreline and the existing seawalls which are supporting the shoreline. Based on the AGS' findings, the existing cove next to The Ramp Restaurant appears to be unstable due to either seismically-induced settlement or landslide. A major crack approximately 16 inches wide with 6 inches drop was observed on the ground at the cove area reducing in the size and extending toward the north of The Ramp Restaurant. Another crack approximately 2 inches wide with 1 inch of vertical drop was also observed south of The Ramp Restaurant which appears to be related to the settlement in that area. In addition, shear failure of one of the chain-link poles indicates that the 2-inch crack may be due to the settlement in the project area.

In addition, AGS assessed the static stability of the slopes which are protected by seawalls. No as-built information regarding the embedment depth of the seawalls was available at the time of preparation of this report. The steel sheet-pile seawall seems highly corroded and a lateral movement up to approximately 6 inches at top of the seawalls near the Slipway 4 was observed. The structural integrity of the existing seawalls is not within the scope of work of this report. A Structural Engineer should evaluate any structural deficiency of the existing seawalls and recommends a remediation plan, if necessary. No obvious lateral movements were observed where concrete seawall was used; however, signs of settlement up to 1 inch were observed. In addition, the impact of the addition surcharge load due to proposed improvement on the seawalls should be evaluated by a Structural Engineer.

Since the seawall has been in place for many years, the static factor of safety must be significantly greater than one. Since the seawalls was constructed and existing fill was placed after 1906 San Francisco earthquake and prior to the Lorna Prieta earthquake. It may be beneficial that performance of the seawall and the existing fill during Lorna Prieta earthquake to be used as a guide in estimating future movements under a postulated 7.9 magnitude earthquake.

No seismological station was established near Pier 70 area. Based on a review of peak ground accelerations measured at USGS stations north and south of the site, we judge that peak ground accelerations during the Lorna Prieta earthquake at the site were probably close to 0.2g. Based on this information the potential seismically-induced lateral deformation of the seawall was calculated for a 0.2g peak ground acceleration using several Newmark-type deformation analyses (Newmark, 1965; Hynes-Griffin and Franklin, 1984; and Duncan and Brandon, 2005). The calculated seismically-induced permanent lateral deformations are within reasonable range from each other ranging from 6 inches to 1 foot. It is worth noting that the calculated seismically-induced permanent lateral deformations are in agreement with the deformations observed near the Ramp Restaurant.

3.12 CONSTRUCTION CONSIDERATIONS

3.12.1 General

Although the information in this report is primarily intended for the design engineers, data from our borings may also be useful to the contractor. However, it is the responsibility of the bidders and contractors to evaluate soil and groundwater conditions independently and to develop their own conclusions and designs regarding soil densification, excavation, grading, foundation construction, and other construction and safety related issues. Extreme care should be exercised by the contractor to avoid excessive deflections of the existing seawalls due to seismically-induced lateral deformation during the construction.

3.12.2 Construction Dewatering and Control of Runoff

Construction dewatering is not anticipated to be necessary since based on the results of the subsurface explorations; groundwater is not anticipated to be shallower than excavations.

However, surface runoff could collect on the project area and depending upon the time of year work is in progress an erosion control and storm water treatment plan should be developed, per regulations. It is the responsibility of the contractor to provide an adequate dewatering system during construction.

The design of the dewatering system should also provide for collection and removal of surface water and rainfall.

3.12.3 Protection of Utilities

Underground utilities exist within the boundaries of the proposed project area. Before the start of construction all utility locations must be verified in the field and protective measures taken. The project area is transected by several subsurface utilities, including, gas and electrical lines, water lines, sewer lines, storm drains, and telecommunication lines.

3.12.4 Effects on Adjacent Facilities

During the construction adjacent to existing structures, care should be taken to adequately support facilities that might be affected by the proposed construction procedures. Proposed grading will generate settlement within the project site which will affect the existing structures utilities in the project site.

3.12.5 Geotechnical Services During Construction

The Contractor should review project plans and specifications prior to construction to confirm that the geotechnical issues of the project are consistent with the intent of the recommendations presented herein. A qualified Geotechnical Engineer should also be retained during construction to observe the following items.

- Site preparation and earthwork;
- Installation of the deep foundations;
- Grouting and soil improvement, if any; and
- Placement and compaction of fills and backfills.

Presence of a geotechnical engineer during construction will allow providing consultation regarding the geotechnical issues of the project during the construction. Geotechnical Engineer representative will observe the soil conditions encountered during construction, verify the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures, if the conditions differ from those described herein. In addition, the Geotechnical Engineer will perform field density tests during the placement and compaction of the engineered fill and backfill.

4. CLOSURE

This report has been prepared in accordance within the generally accepted professional geotechnical engineering reporting guidelines and is submitted for the exclusive use of the Port of San Francisco and the proposed Crane Cove Park Improvement project in San Francisco, California. No other warranty, expressed or implied, is made.

The analyses and recommendations provided in this report are based on the data obtained from the two borings drilled for this study, and data compiled from previous subsurface investigations. The nature and extent of the variations between the borings may not become evident until construction. In the event that variations occur, it may be necessary to reevaluate the recommendations in this report.

It is the responsibility of the owner and/or their representatives to ensure that the provisions of this report are incorporated into the plans and specifications and that the necessary steps are taken to see that the contractors carry out such provisions.

Respectfully submitted,
AGS, Inc.

Bahram Khamenehpour, Ph.D.	Kamran Ghiassi, Ph.D.
Geotechnical Engineer 2104	Geotechnical Engineer 2792

5. REFERENCES

Abrahamson, N.A. and Silva, W.J., 1997, Empirical Response Spectral Attenuation Relations for Shallow Crustal Earthquakes, Seismological Research Letters, vol. 68, no.1, January February, pp. 94-127.

AGS, 1989, Report Geotechnical Investigation Mariposa / Transport Facilities, San Francisco, prepared for San Francisco Clean Water Program, June

Anderson, J.G., 1979, Estimating the Seismicity from Geological Structure for Seismic-Risk Studies, Bulletin of the Seismology Society of American, V. 69, 163-158.

Anderson, R.G., and Luco, S.E., 1983, Consequences of Slip Rate Constraints on Earthquake Recurrence Relations: Bulletin of the Seismological Society of America, v. 73, no. 2, 471-496.

Architectural resources Group Report, 2011, Documentation and Assessment of Historic Artifacts, prepared for Port of San Francisco, December

Bernhardt, G., and Toppozada, T.R., Relocation of the 1836 Hayward Fault Earthquake to the San Andreas Fault, Transactions of the American Geophysical Union, vol. 77, no. 46 (supplement), 1996.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1997, Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of Recent Work, Seismological Research Letters, vol. 68, no. 1, January February, pp. 127-153.

Borchardt, G. and Toppozada, T.R., 1996, Relocation of the “1836 Hayward Fault Earthquake” to the San Andreas Fault, in EOS Transactions, 1996 Fall Meeting, American Geophysical Union, vol. 77, no. 46, November.

Campbell, K.W., 1997 and 2000, Empirical Near-Source Attenuation Relationships for Horizontal and Vertical Components of Peak Ground Velocity, and Pseudo-Absolute

Acceleration Response Spectra, Seismological Research Letters, vol. 68, no. 1, January February, pp. 153-179.

Clough, G. and T. O'Rourke, 1990. "Construction Induced Movements of Insitu Walls", Design and Performance of Earth Retaining Structures, ASCE Geotechnical Special Publications, 25: 439-470.

Cornell, C.A., 1968, Engineering Seismic Risk Analysis@, Bulletin of the Seismology Society of America, v. 58, no. 5, 1583-1606.

Dames and Moore, 1969, Soil and Foundation Investigation, Proposed Dry Dock and High Water Platform, San Francisco Yard, prepared for Bethlehem Steel Corporation Shipbuilding Department.

Dames and Moore, 1973, Soils Investigation Marginal Wharf and Container Yard, Pier 70, Alvord Grant, San Francisco, California, prepared for the Port of San Francisco, May 15.

Ellsworth, W.L. 1990. Earthquake History, 1769-1989. Chapter 6 in the San Andreas Fault System, California. U.S. Geological Survey Professional Paper 1515. U.S. Government Printing Office, Washington, D.C. 283 pp.

Federal Emergency Management Agency, 2007, Coastal Structures Form, Certified by Port of San Francisco, December 5, 2007.

Goldman, H.B., 1969, Geologic and Engineering Aspects of San Francisco Bay Fill, CA. Division of Mines and Geology Special Report 97, pp 11-29.

Graymer, R.W., 2006, Geologic Map of the San Francisco Bay Region http://geomaps.wr.usgs.gov/sfgeo/geologic/downloads.html

Harding Lawson Associates, 1983, South Beach Small Boat harbor and Park, Piers 40 through 46A, San Francisco, California, HLA Job No. 2222,041.04

Harding Lawson Associates, 1992, Final Report Liquefaction Study of North Beach, Embarcadero Waterfront, South Beach, and Upper Mission Creek Area, San Francisco, California, HLA Job No. 17952,041.04

Hayward Fault Paleo-earthquake Group, 1997, The Northern Hayward fault, California; preliminary timing of paleo-earthquakes, American Geophysical Union Fall 1997 Conference Abstracts, San Francisco.

Hryciw 1995; Stewart et al. 1997; Boulanger et al. 1997; Mitchell et al. 1998b

Idriss, I.M., 1985, Evaluating Seismic Risk in Engineering Practice, in Proceedings, Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, v. 4, p. 255-320.

Idriss, I.M., 1987, Earthquake ground motions, Lecture notes, Course on Strong Ground Motion, Earthquake Engin. Res. Inst., Pasadena, Calif., April 10-11, 1987.

Ishihara, K., and Yoshimine, M. 1992, Evaluation of settlements in sand deposits following liquefaction during earthquakes, soils and foundations, vol. 32, no. 1, pp. 173-188.

Jennings, C.W., 1992, Preliminary Fault Activity Map of California, Cal Div. Mines & Geology Open-File Report 92-03.

Joyner, W.B. and Boore, D.M., 1988 Measurement Characterization, and Prediction of Strong Ground Motion , Earthquake Engineering and Soil Dynamics II, Proceedings of the Specialty Conference Sponsored by the Geotechnical Engineering Division of the American Society of Civil Engineers.

McGuire, R.K., 1976, FORTRAN Computer Program for Seismic Risk Analysis, U.S. Geotechnical Survey, Open-File Report 76-67.

Newmark, N.M., 1967, Problems in Wave Propagation in Soil and Rock, International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, Albuquerque, N.M.

Petersen, M.D., Bryant, W.A., Cramer, C.H., Cao, T., and Reichle, M.S., 1996, Probabilistic Seismic Hazard Assessment for the State of California, Cal Div. of Mines & Geology Open-File Report 96-08; USGS Open-File Report 96-706.

Port of San Francisco, 2010, Pier 70 Preferred Master Plan, April

Real, C.R., Toppozada, T.R., and Parke, D.L., 1978, Earthquake Epicenter Map of California; California Division of Mines and Geology, Map Sheet 39, Scale 1:1,000,000.

Seed, H.B., and Idriss, I.M., 1982, Ground Motions and Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute Monograph.

Seed, H.B., Ugas, C., and Lysmer, J., 1976, ADepartment Spectra for Earthquake Resistant Design@, Bulletin of the Seismological Society of America, v. 66, no. 1, 221-243.

Seed, H.B., Tokimatsu, K., Harder, L.F., and Chang, R.M., 1984, Influence of SPT Procedures in Soil Liquefaction Resistance Evaluation, Rep# UCB/EERC 84/15, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Shah, H.C., Mortagt, C.P., Kiremedjian, A.S., and Zsutty, T.C., 1975, AA Study of Seismic Risk for Nicaragua, Part I, The John A. Blume Earthquake Engineering Center, Technical Report No. 11, Department of Civil Engineering, Stanford University.

Shamoto, Y., Zhang, J.M., and Tokimatsu, 1998, Methods for Evaluating Post-liquefaction Ground Settlement and Horizontal Displacement, Soils and Foundations Special Issue No. 2, September 1998

Sun, J.I., Golesorkhi, R., and Seed, H.B., 1998, Dynamic Soil Moduli and Damping Ratios for Cohesive Soils, Rep# UCB/EERC 88/15, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Tokimatsu, Hohji, and H. Bolton Seed, 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, J. of Geotechnical Engineering, V. 113, No. 8.

Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters of and Areas Damaged by M>5 California Earthquakes, 1800-1999: California Division of Mines and Geology, Map Sheet MS 49, scale 1:1546500.

Treadwell and Rollo, 2011, Environmental Site Investigation Report for the Pier 70 Master Plan Area, prepared for the Port of San Francisco, January 13

United States Geological Survey, 2006, Quaternary Fault and Fold Database of the United States

United States Geological Survey, 1989, Lessons Learned from the Loma Prieta, California Earthquake of October 17, 1989, Circular 1045.

Vucetic, M. and Dobry, R. Effect of Soil Plasticity on Cyclic Response, Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 117, #1.

Working Group on California Earthquake Probabilities (WGCEP), 2008, prepared for the 2008 USGS National Seismic Hazard Report

PLATES

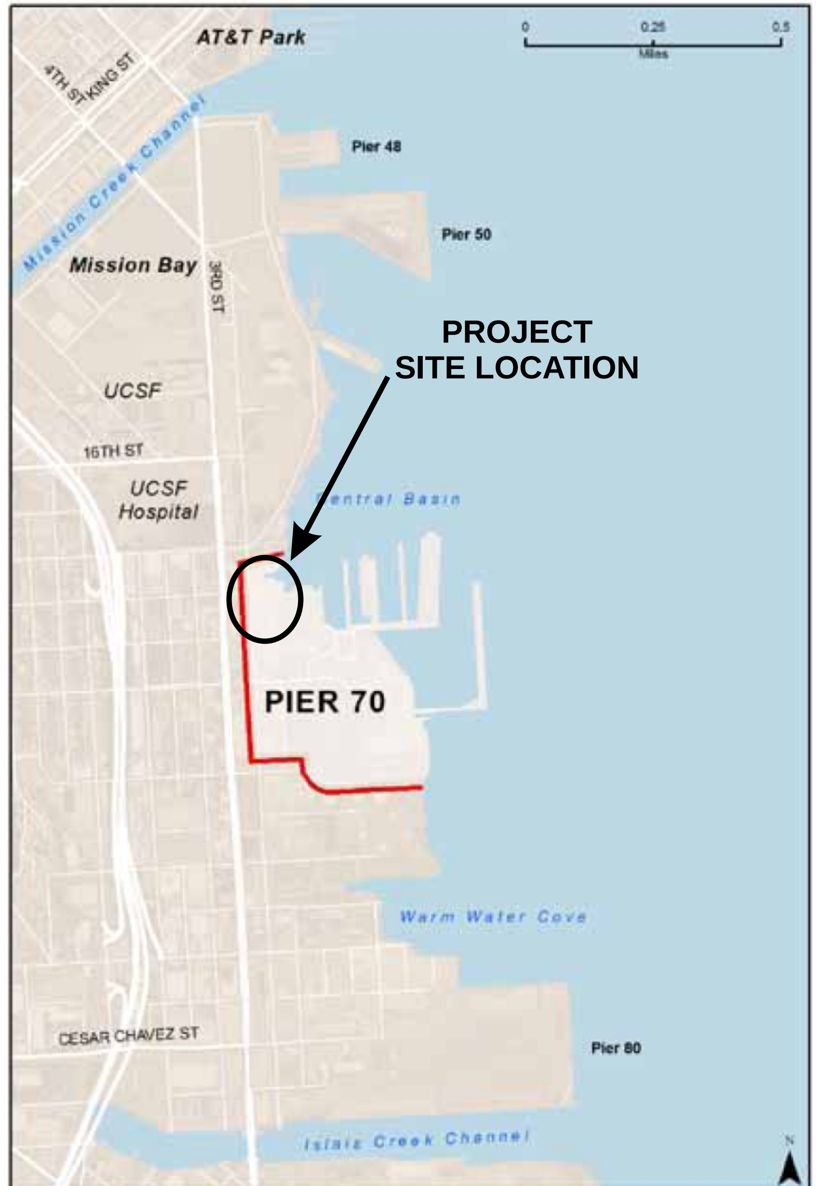


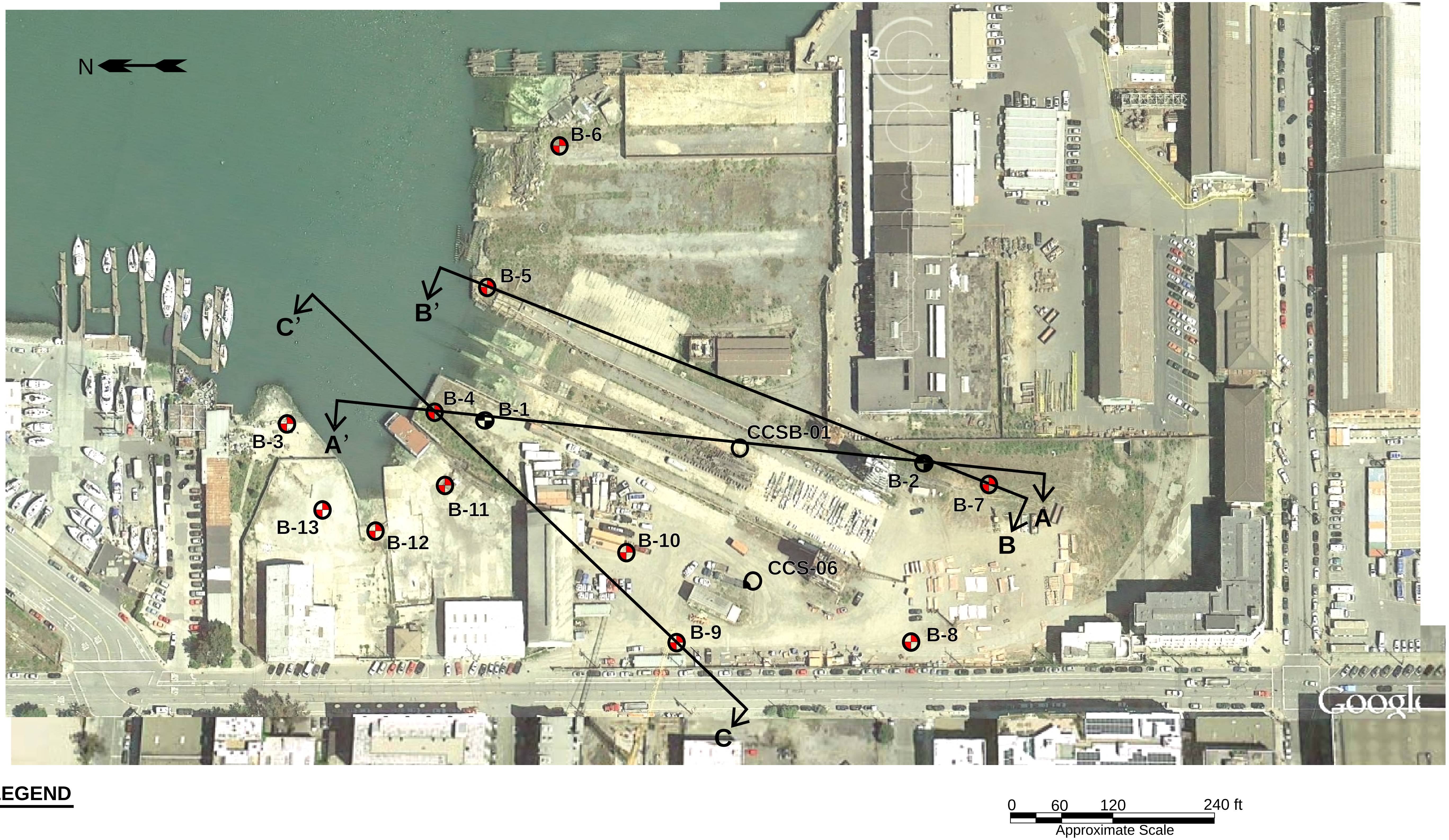
EXHIBIT 1: PIER 70 SITE SETTING AND LOCATION

Reference: Port of San Francisco, April 2010,
Pier 70 Preferred Master Plan, Exhibit 1.

SITE LOCATION MAP
Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, California

AGS

JOB NO. KK-0210	DATE: APRIL 2014	PLATE 1



LEGEND

B-10 — Approximate AGS Boring Locations, 2014 (B-3 through B-10)

B-2 — Approximate AGS Boring Locations, 2012 (B-1 and B-2)

CCSB-06 — Approximate Treadwell and Rollo Geotechnical Boring Locations, 2011

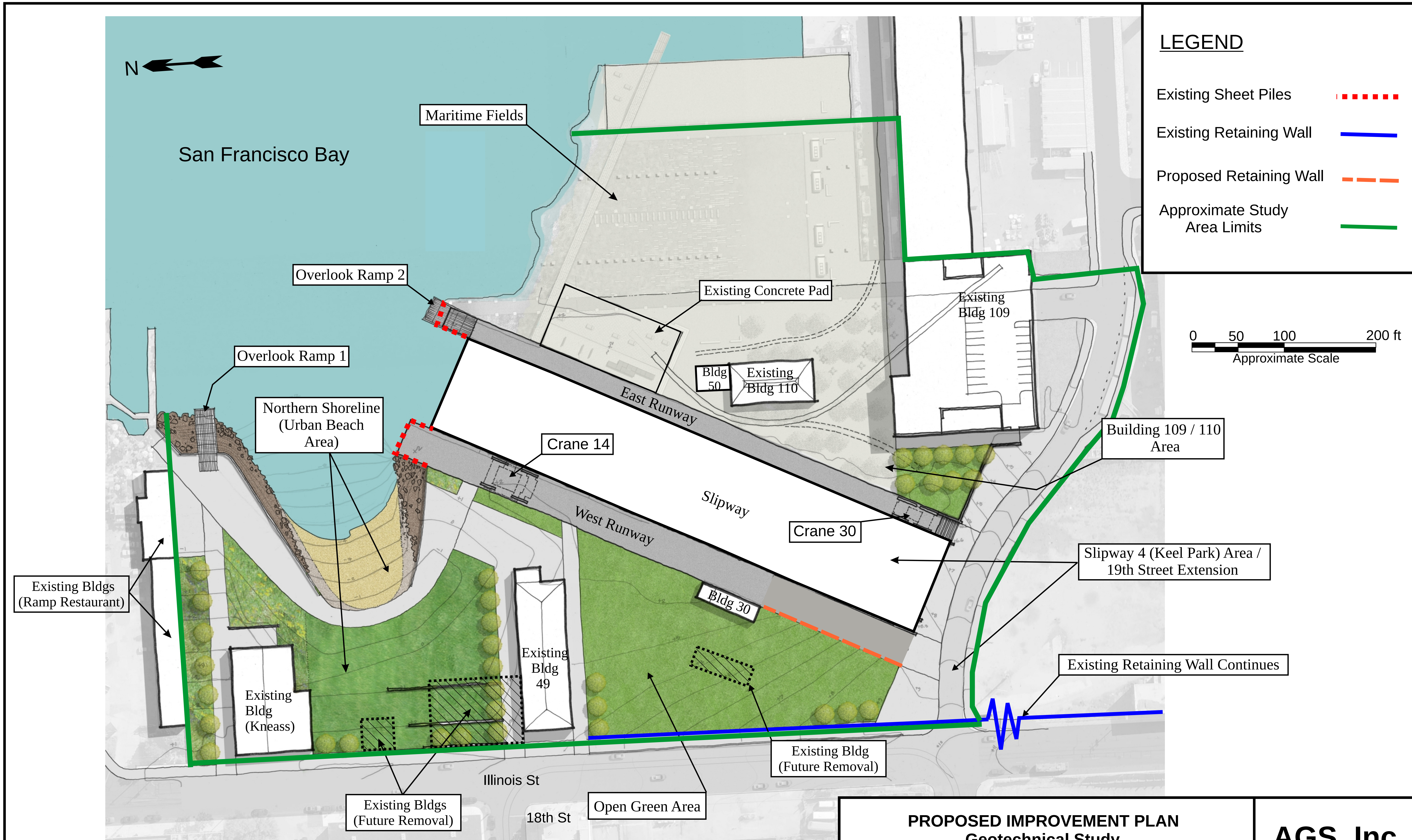
C' C — Subsurface Profile Cross Section

APPROXIMATE BORING LOCATION MAP

Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, CA

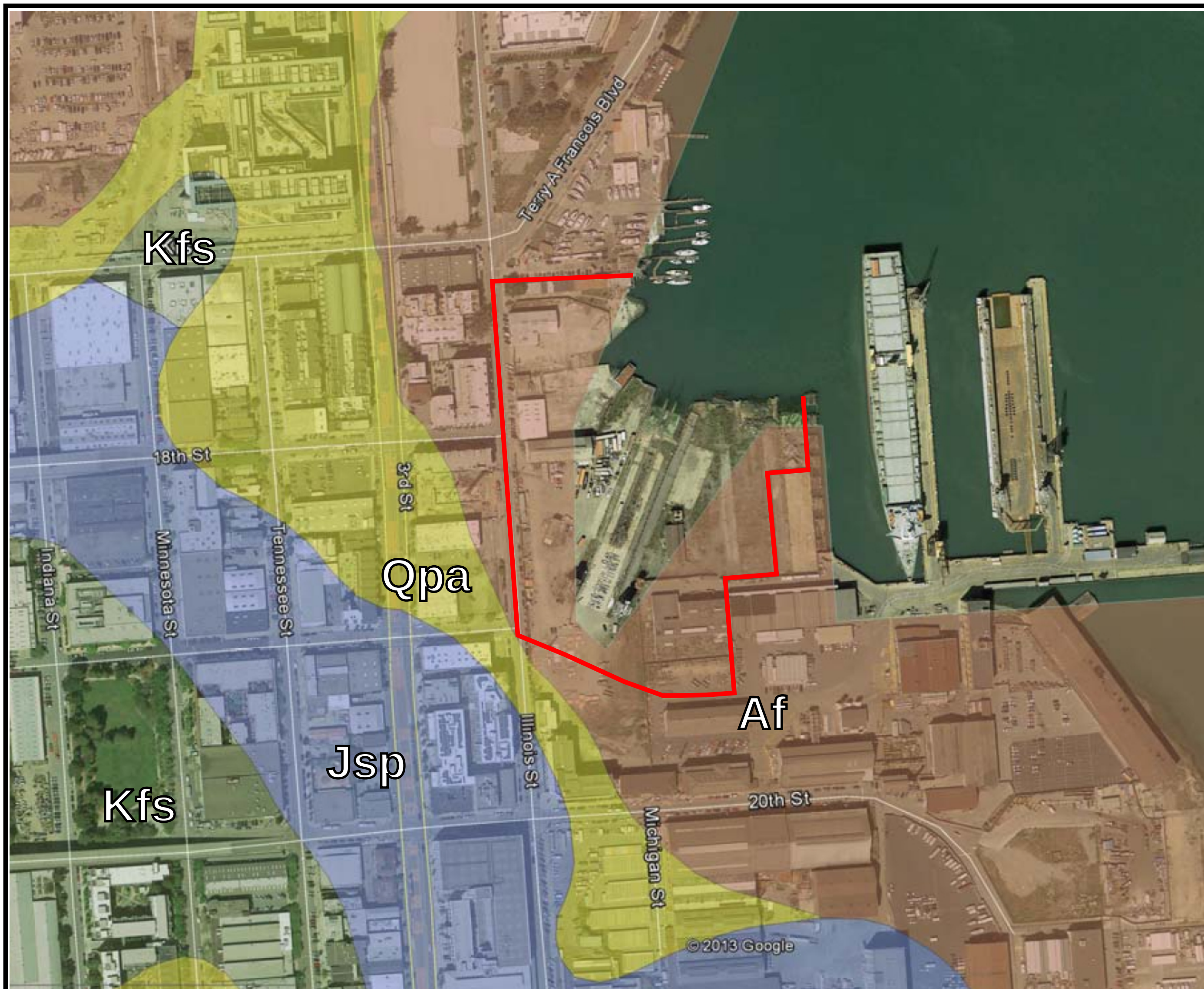
AGS

JOB NO. KK-0210	DATE: APRIL 2014	PLATE 2



Sources: (1) Illustrative Plan, December 2013, Crane Cove Park Master Plan

PROPOSED IMPROVEMENT PLAN **Geotechnical Study** **Pier 70 Crane Cove Park - Phase II** **San Francisco, California**		**AGS, Inc.** CONSULTING ENGINEERS
JOB NO. KK-0210	DATE: APRIL 2014	PLATE 3



Reference: http://pubs.usgs.gov/sim/2006/2918/

LEGEND

Af Artificial Fill (Historic)

Qpa Alluvium (Pleistocene)

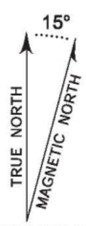
Kfs Sedimentary Rock (Cretaceous)

Jsp Serpentinite (Jurassic)

Approximate Site Boundaries

0 200 400 ft

APPROXIMATE SCALE



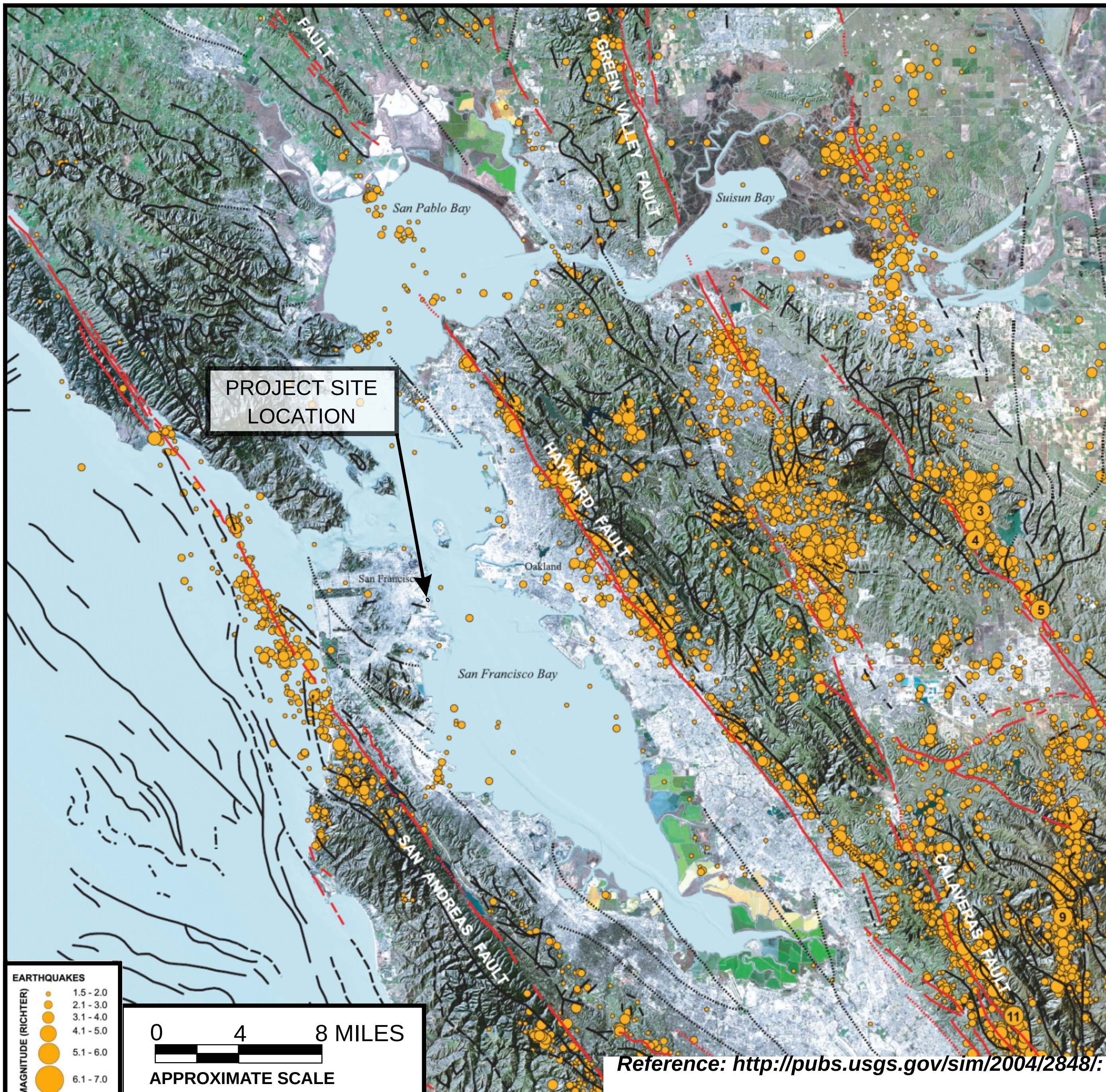
APPROXIMATE MEAN DECLINATION, 2004

GEOLOGY MAP

Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, CA

AGS CONSULTING ENGINEERS

JOB NO. AGS KK-0210	DATE: APRIL 2014	PLATE 4



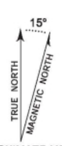
LEGEND

FAULTS

- Documented
- Approximately located
- Inferred
- Active in last 700,000 years
- Active prior to 700,000 years

MAGNITUDE 5.0 AND GREATER EARTHQUAKES

Earthquake No.	Date	Time (Universal Time Coordinated)	Latitude (N.)	Longitude (W.)	Depth (km)	Magnitude (Richter scale)
1	28 Nov 1974	23:01:24.56	36.9202	121.4663	6.11	5.37
2	6 Aug 1979	17:05:22.91	37.1042	121.5127	8.93	5.80
3	24 Jan 1980	19:00:08.63	37.8367	121.7708	14.72	5.23
4	24 Jan 1980	19:01:01.53	37.8115	121.7772	7.07	5.10
5	27 Jan 1980	02:33:35.32	37.7493	121.7062	14.69	5.18
6	23 Jan 1984	05:40:19.93	36.3650	121.8878	8.19	5.12
7	24 Apr 1984	21:15:18.75	37.3095	121.6787	8.74	5.82
8	26 Jan 1986	19:20:50.93	36.8040	121.2847	8.72	5.50
9	31 Mar 1986	11:55:39.80	37.4788	121.6858	9.17	5.70
10	20 Feb 1988	08:39:57.24	36.7957	121.3117	9.62	5.03
11	13 June 1988	01:45:36.52	37.3923	121.7415	9.71	5.30



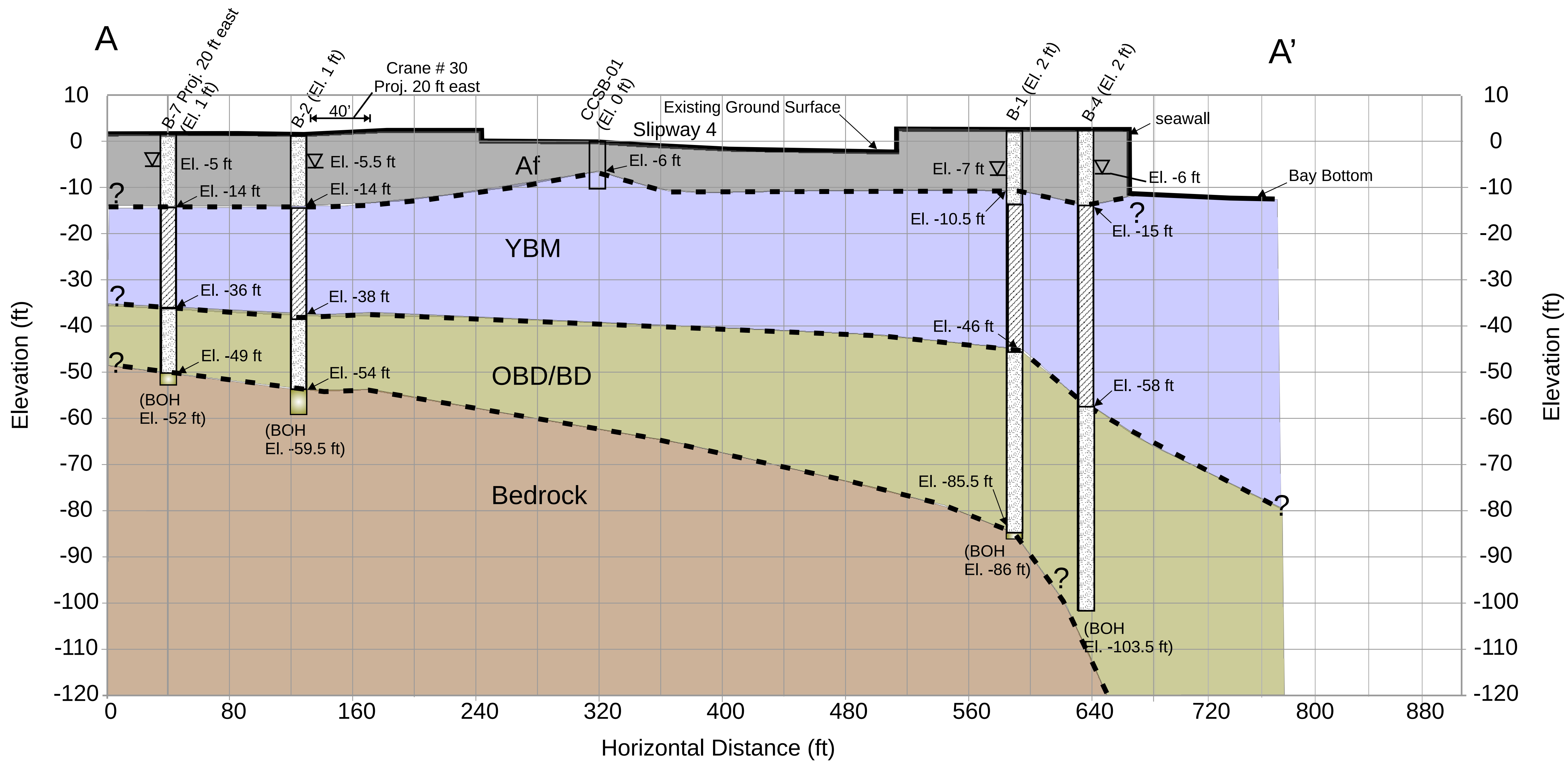
APPROXIMATE MEAN DECLINATION, 2004

EARTHQUAKE EPICENTERS AND FAULT MAP

Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, CA

AGS CONSULTING ENGINEERS

JOB NO. AGS KK-0210 | DATE: APRIL 2014 | PLATE 5



LEGEND

Af	Artificial fill; typically loose gravel and sand mixture with little clay fines, some concrete rubble and few cobble to boulder-size rocks, particularly nearer the seawall, fill grades to Bay Mud with wood pieces from old pilings and sometimes strong tar odor in the bottom few feet
YBM	Young Bay Mud; typically Holocene-age (Quaternary) clay and silt with few shells and sand lenses, very soft and saturated, commonly black or dark gray
OBD / BD	Alluvium, Bayside Deposits, and Old Bay Mud; typically Late Pleistocene to Early Holocene-age (Quaternary), stiff or very stiff clay with sand and gravel, commonly brown, olive or gray, sand includes shells, Old Bay Mud is not separated but is commonly green or blue-gray, soft to stiff fat clay with sand and gravel
Bedrock	Franciscan Complex Sedimentary Rock; includes sandstone, shale, and siltstone, may also include serpentinite inclusions, commonly severely to moderately weathered, apparently weak to moderately strong within the explored depths
? - - -?	Approximate geologic contact
BOH	Bottom of Hole

NOTES

(1) All elevations and dimensions are approximate, based on the City of San Francisco Datum.

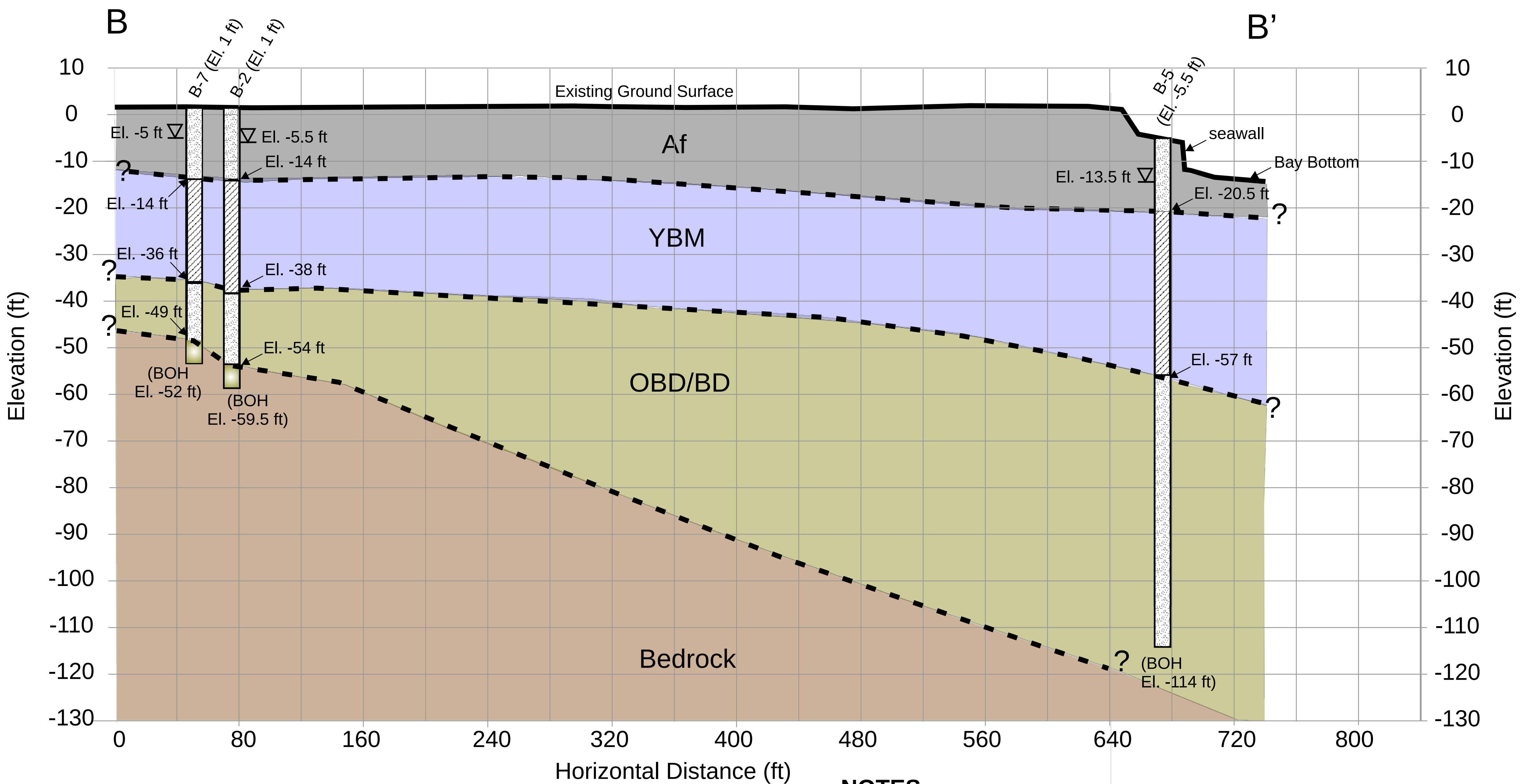
(2) Groundwater level measured immediately after drillling.

(3 The subsurface conditions were interpolated. Site elevations are from the Crane Cove Park Topographic Survey, 2013. Our interpolation and extrapolation of the subsurface soil, rock, and groundwater conditions are based on widely spaced borings. Variations in the actual conditions from those assumed should be anticipated. Neither boring logs nor the generalized soil profiles are stand-alone documents, and they should only be used within the full context of the soil report. The profile was prepared for the sole purpose of developing geotechnical recommendations. It is not intended for use in estimating quantity of various soil types.

SUBSURFACE PROFILE - SECTION A-A'
Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, California

AGS CONSULTING ENGINEERS

JOB NO. KK-0210	DATE: APRIL 2014	PLATE 6



LEGEND

Af	Artificial fill; typically loose gravel and sand mixture with little clay fines, some concrete rubble and few cobble to boulder-size rocks, particularly nearer the seawall, fill grades to Bay Mud with wood pieces from old pilings and sometimes strong tar odor in the bottom few feet
YBM	Young Bay Mud; typically Holocene-age (Quaternary) clay and silt with few shells and sand lenses, very soft and saturated, commonly black or dark gray
OBD / BD	Alluvium, Bayside Deposits, and Old Bay Mud; typically Late Pleistocene to Early Holocene-age (Quaternary) stiff or very stiff clay with sand and gravel, commonly brown, olive or gray, sand includes shells, Old Bay Mud is not separated but is commonly green or blue-gray, soft to stiff fat clay with sand and gravel
Bedrock	Franciscan Complex Sedimentary Rock; includes sandstone, shale, and siltstone, may also include serpentinite inclusions, commonly severely to moderately weathered, apparently weak to moderately strong within the explored depths
? - - -?	Approximate geologic contact
BOH	Bottom of Hole

NOTES

(1) All elevations and dimensions are approximate, based on the City of San Francisco Datum.

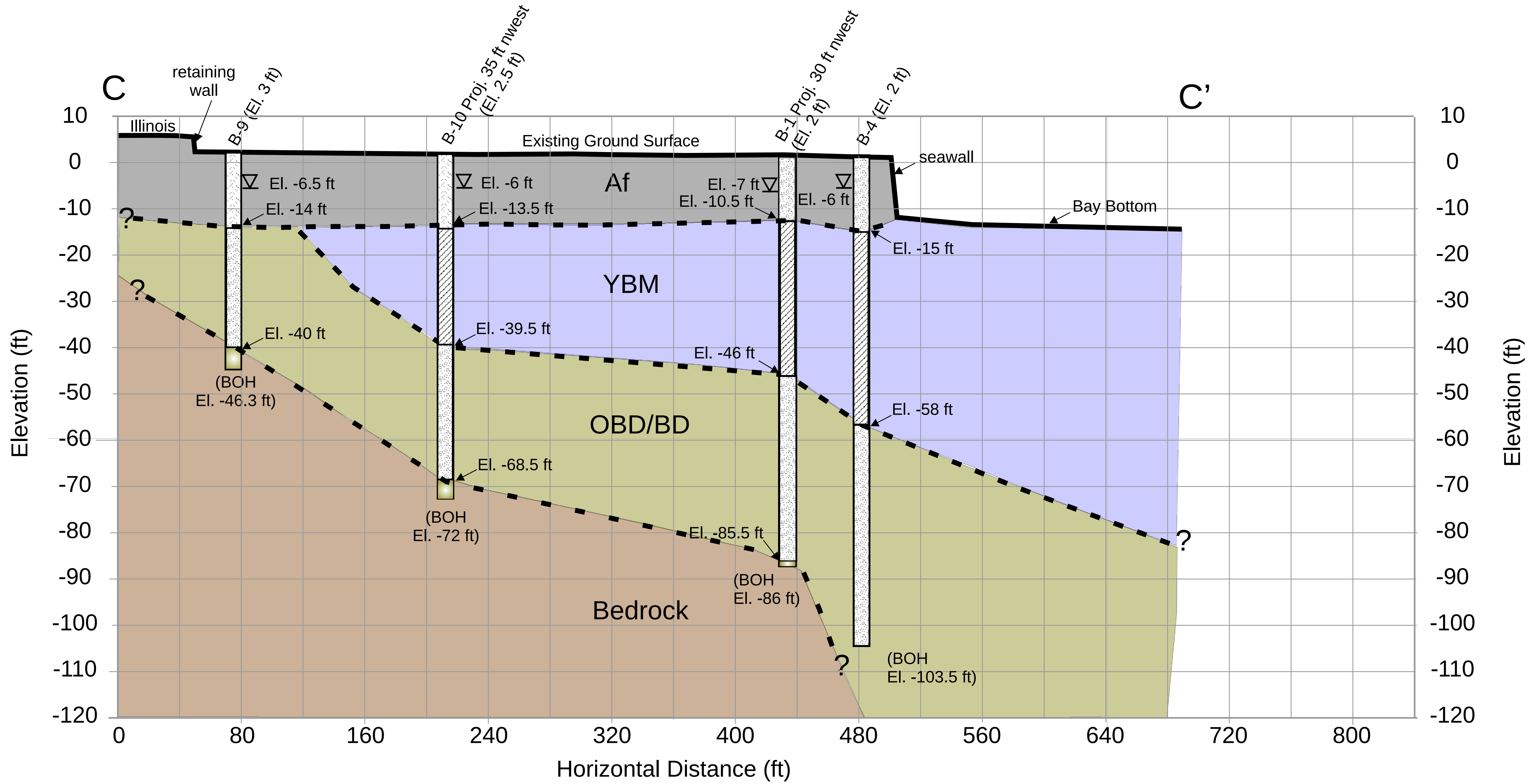
(2) Groundwater level measured immediately after drillling.

(3 The subsurface conditions were interpolated. Site elevations are from the Crane Cove Park Topographic Survey, 2013. Our interpolation and extrapolation of the subsurface soil, rock, and groundwater conditions are based on widely spaced borings. Variations in the actual conditions from those assumed should be anticipated. Neither boring logs nor the generalized soil profiles are stand-alone documents, and they should only be used within the full context of the soil report. The profile was prepared for the sole purpose of developing geotechnical recommendations. It is not intended for use in estimating quantity of various soil types.

SUBSURFACE PROFILE - SECTION B-B'
Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, California

AGS CONSULTING ENGINEERS

JOB NO. KK-0210	DATE: APRIL 2014	PLATE 7



LEGEND

Af	Artificial fill; typically loose gravel and sand mixture with little clay fines, some concrete rubble and few cobble to boulder-size rocks, particularly nearer the seawall, fill grades to Bay Mud with wood pieces from old pilings and sometimes strong tar odor in the bottom few feet
YBM	Young Bay Mud; typically Holocene-age (Quaternary) clay and silt with few shells and sand lenses, very soft and saturated, commonly black or dark gray
OBD / BD	Alluvium, Bayside Deposits, and Old Bay Mud; typically Late Pleistocene to Early Holocene-age (Quaternary) stiff or very stiff clay with sand and gravel, commonly brown, olive or gray, sand includes shells, Old Bay Mud is not separated but is commonly green or blue-gray, soft to stiff fat clay with sand and gravel
Bedrock	Franciscan Complex Sedimentary Rock; includes sandstone, shale, and siltstone, may also include serpentinite inclusions, commonly severely to moderately weathered, apparently weak to moderately strong within the explored depths
? - - -?	Approximate geologic contact
BOH	Bottom of Hole

NOTES

(1) All elevations and dimensions are approximate, based on the City of San Francisco Datum.

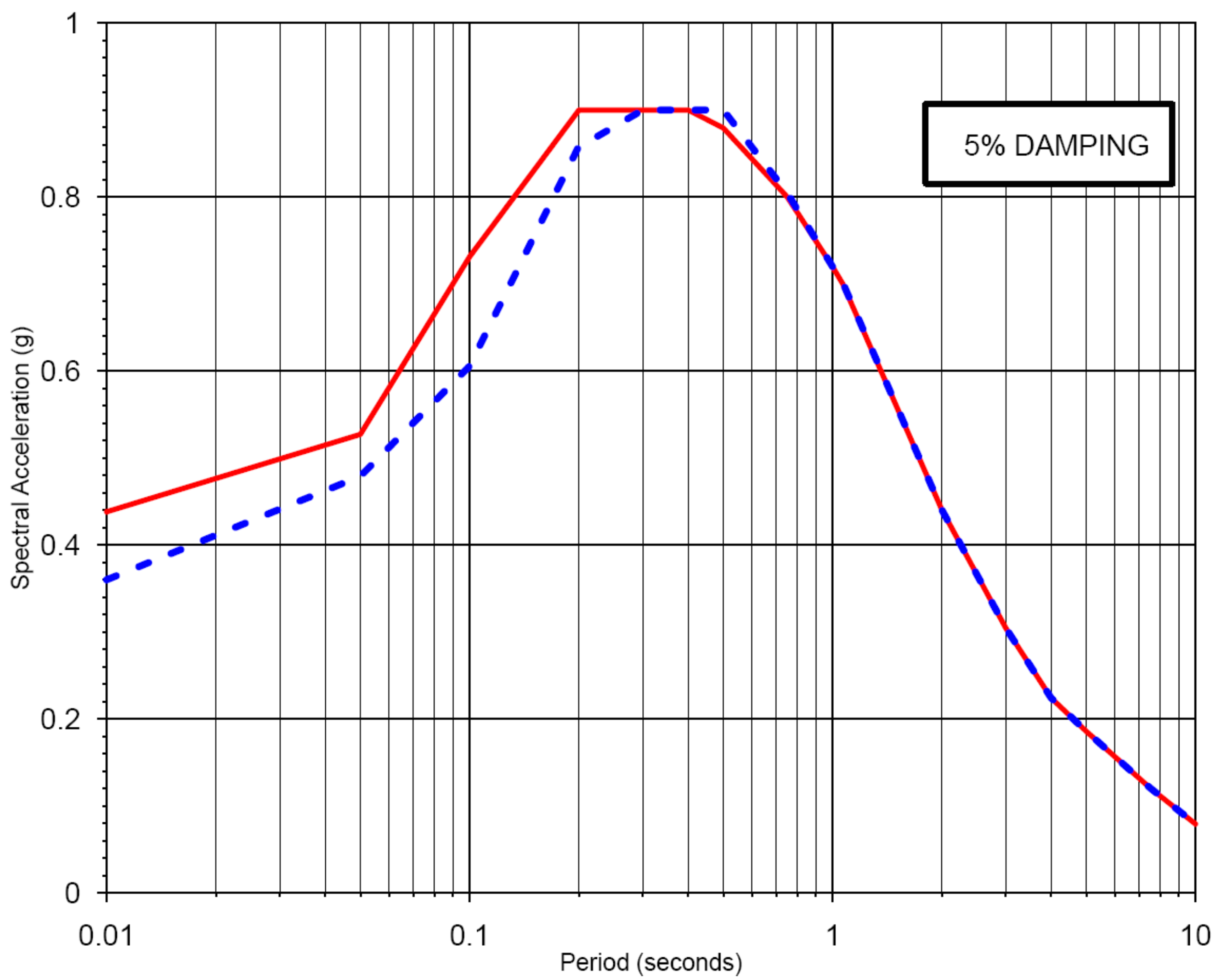
(2) Groundwater level measured immediately after drillling.

(3 The subsurface conditions were interpolated. Site elevations are from the Crane Cove Park Topographic Survey, 2013. Our interpolation and extrapolation of the subsurface soil, rock, and groundwater conditions are based on widely spaced borings. Variations in the actual conditions from those assumed should be anticipated. Neither boring logs nor the generalized soil profiles are stand-alone documents, and they should only be used within the full context of the soil report. The profile was prepared for the sole purpose of developing geotechnical recommendations. It is not intended for use in estimating quantity of various soil types.

SUBSURFACE PROFILE - SECTION C-C'
Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, California

AGS CONSULTING ENGINEERS

JOB NO. KK-0210	DATE: APRIL 2014	PLATE 8



LEGEND

ASCE 7-05

ASCE 7-10

NOTES

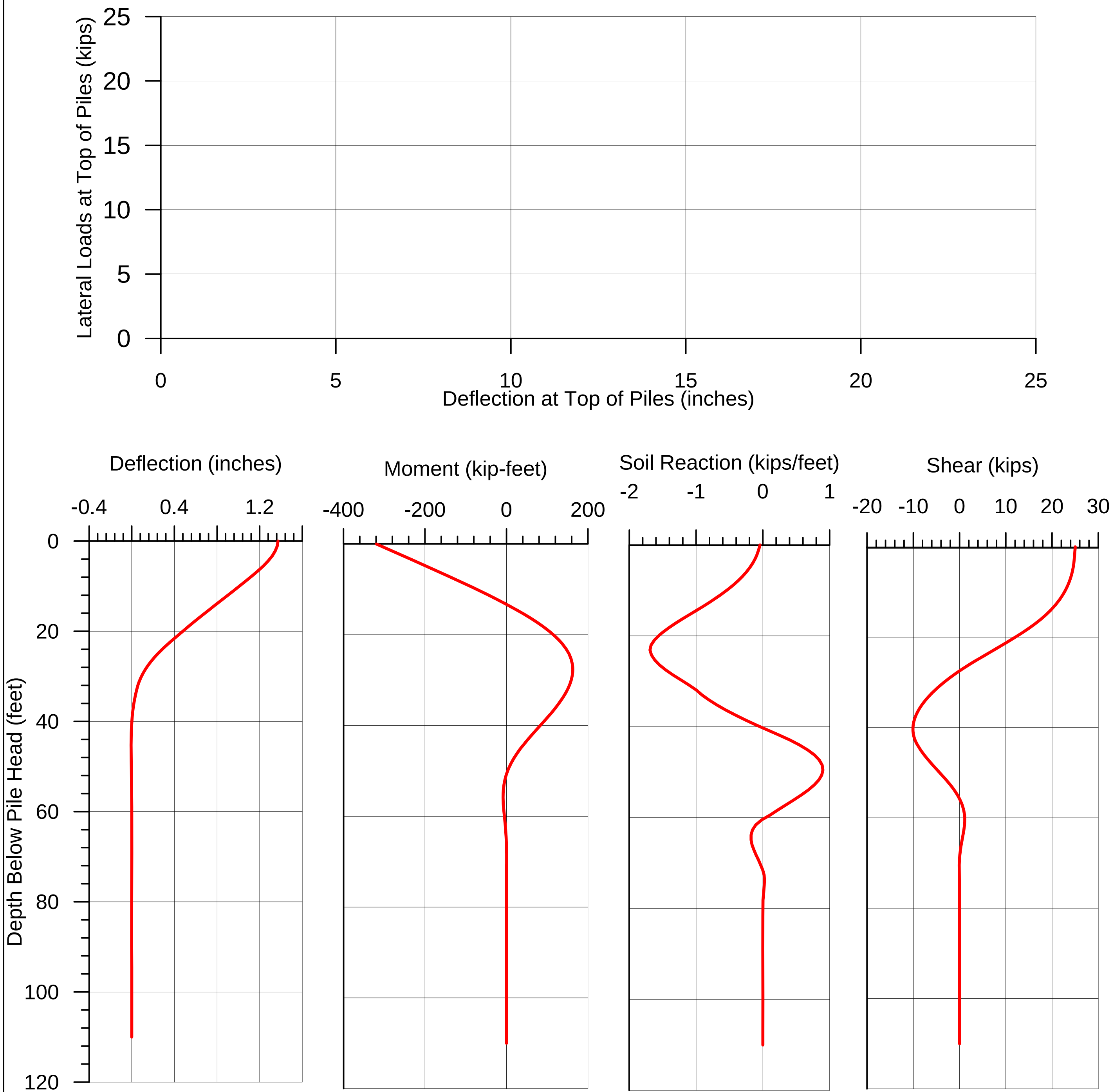
Recommended horizontal response spectra based on Section 21 of ASCE 7.

HORIZONTAL SPECTRAL ACCELERATION BASED ON ASCE 7

Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, CA

AGS CONSULTING ENGINEERS

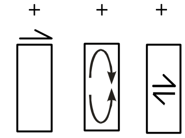
JOB NO. AGS KK-0210	DATE: APRIL 2014	PLATE 9



Notes:

The analyses were performed for 30-inch fixed-head CIDH piles.
This evaluation applies to piles extending to the design tip elevation.
This plate may be used for vertical load of 120 kips.

Sign Conventions
(direction of positive load, moment, and shear)

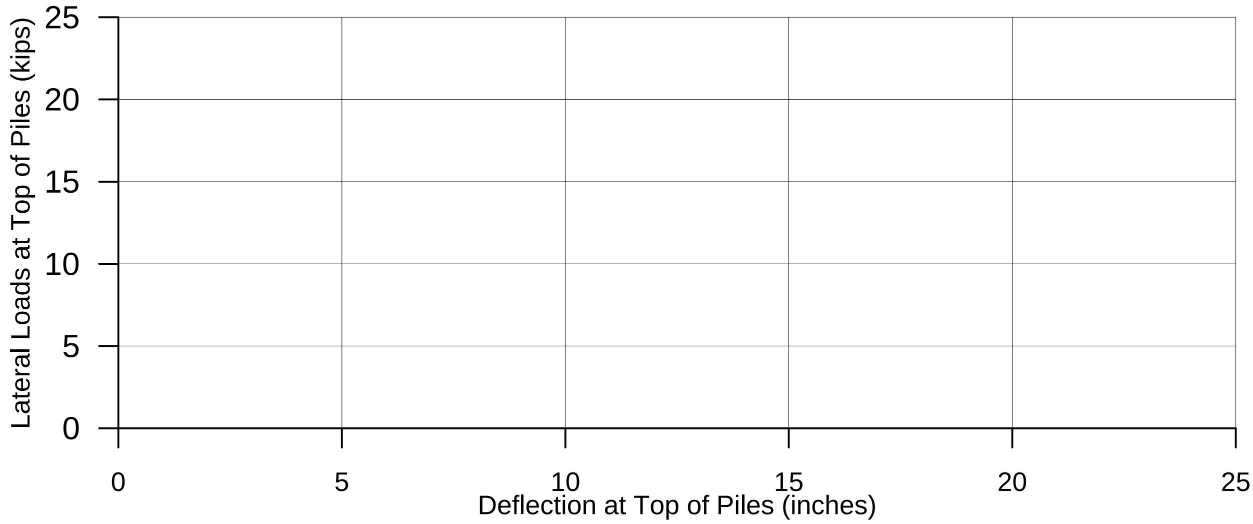


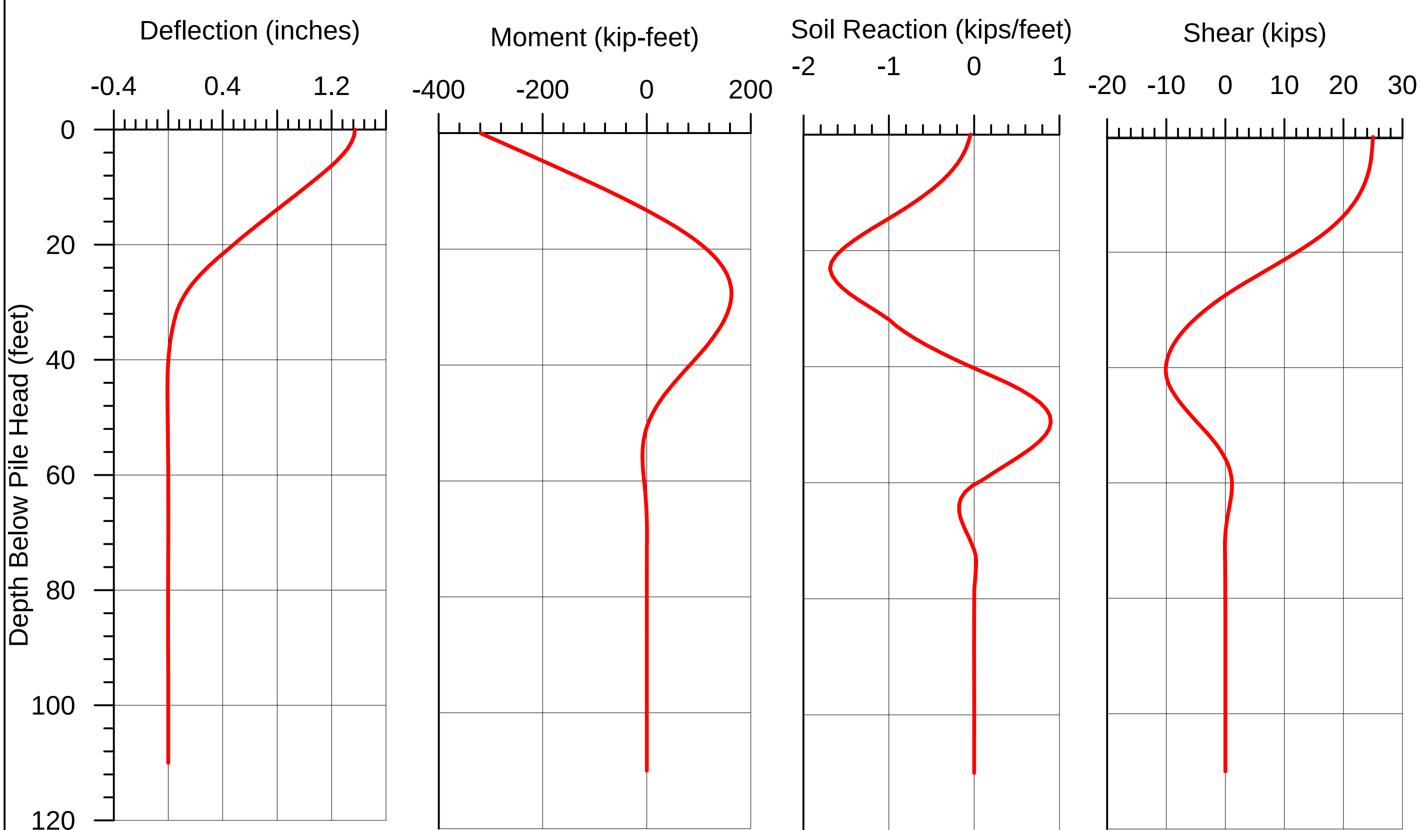
Applied Shear	
25	Kips

RESPONSE TO LATERAL LOADING
EXISTING CRANE 14
PIER 70 - CRANE COVE PARK - PHASE II
SAN FRANCISCO, CALIFORNIA

Project No.: KK-0210 | Date: APRIL 2014

Plate 10

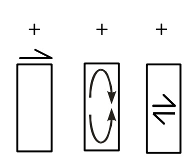




Notes:

The analyses were performed for 30-inch fixed-head CIDH piles.
This evaluation applies to piles extending to the design tip elevation.
This plate may be used for vertical load of 120 kips.

Sign Conventions
(direction of positive load, moment, and shear)



Applied Shear
25 Kips

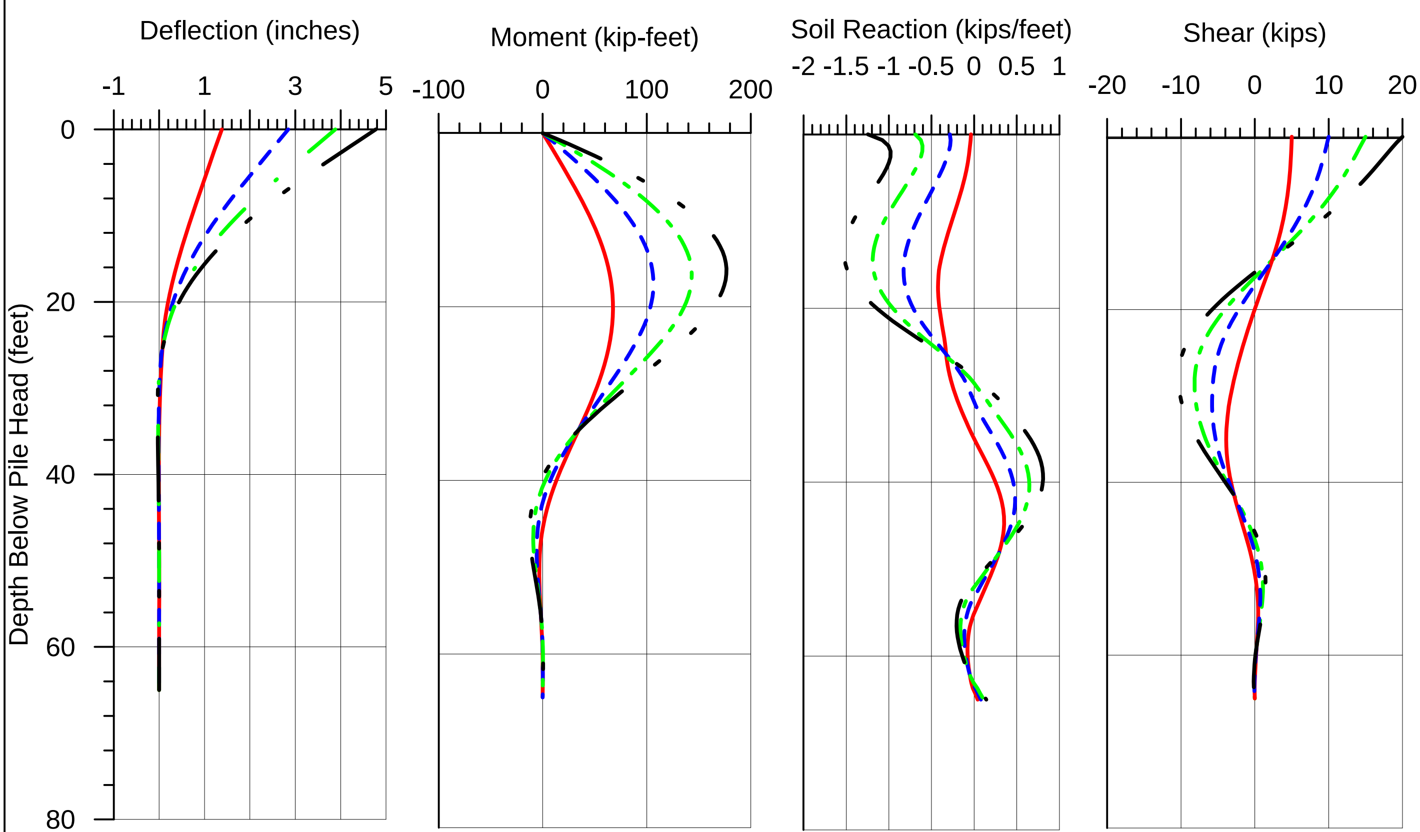
RESPONSE TO LATERAL LOADING
EXISTING CRANE 30
PIER 70 - CRANE COVE PARK - PHASE II
SAN FRANCISCO, CALIFORNIA

Project No.: KK-0210

Date: APRIL 2014

Plate

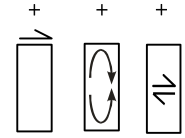
11



Notes:

The analyses were performed for 24-inch free-head CIDH piles.
This evaluation applies to piles extending to the design tip elevation.
This plate may be used for vertical load of 20 kips.

Sign Conventions
(direction of positive load, moment, and shear)

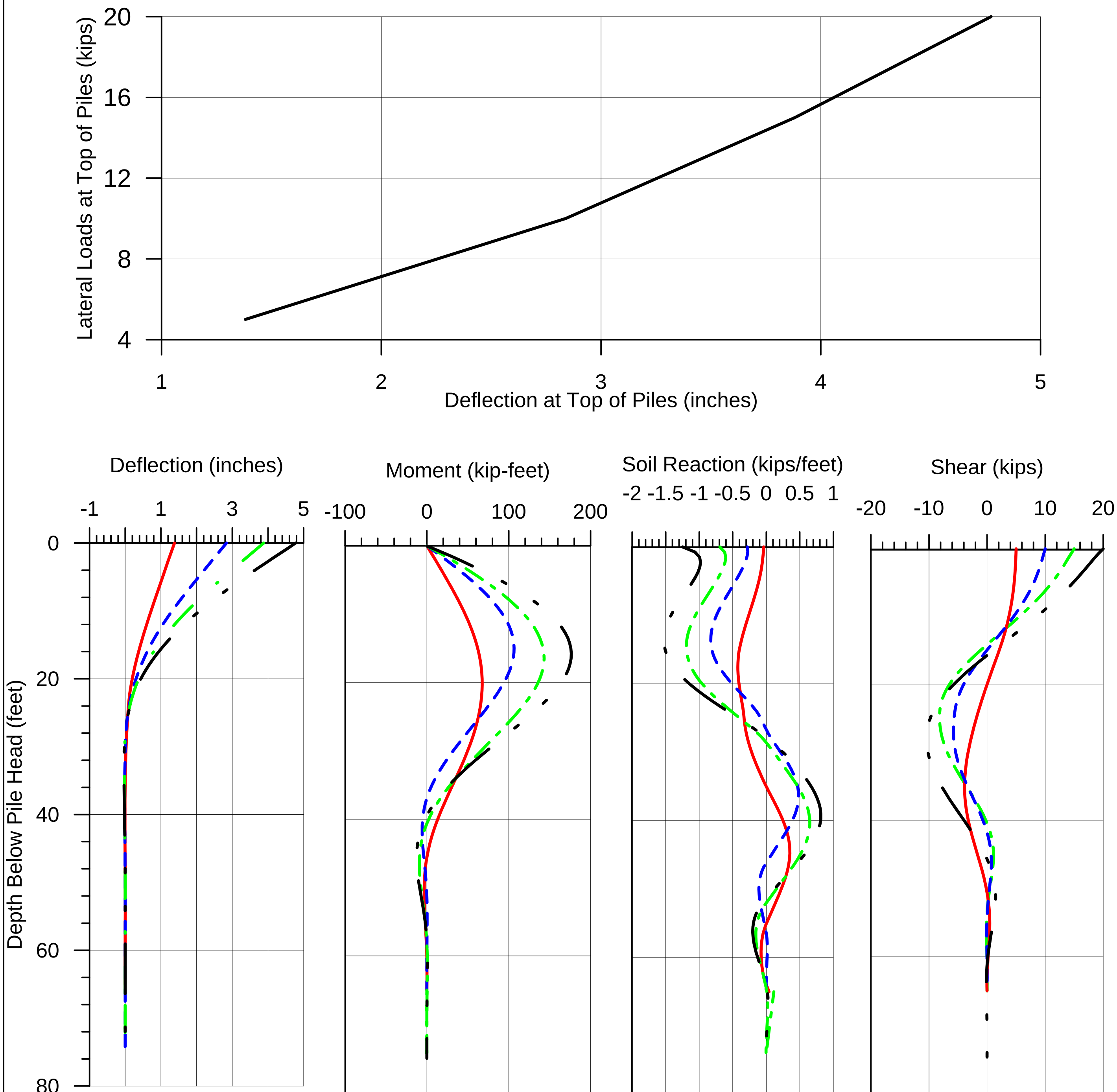


Applied Shear	
5	Kips
10	Kips
15	Kips
20	Kips

RESPONSE TO LATERAL LOADING
PROPOSED OVERLOOK RAMP 1
PIER 70 - CRANE COVE PARK - PHASE II
SAN FRANCISCO, CALIFORNIA

Project No.: KK-0210 | Date: APRIL 2014

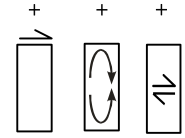
Plate 12



Notes:

The analyses were performed for 24-inch free-head CIDH piles.
This evaluation applies to piles extending to the design tip elevation.
This plate may be used for vertical load of 20 kips.

Sign Conventions
(direction of positive load, moment, and shear)



Applied Shear	
5	Kips
10	Kips
15	Kips
20	Kips

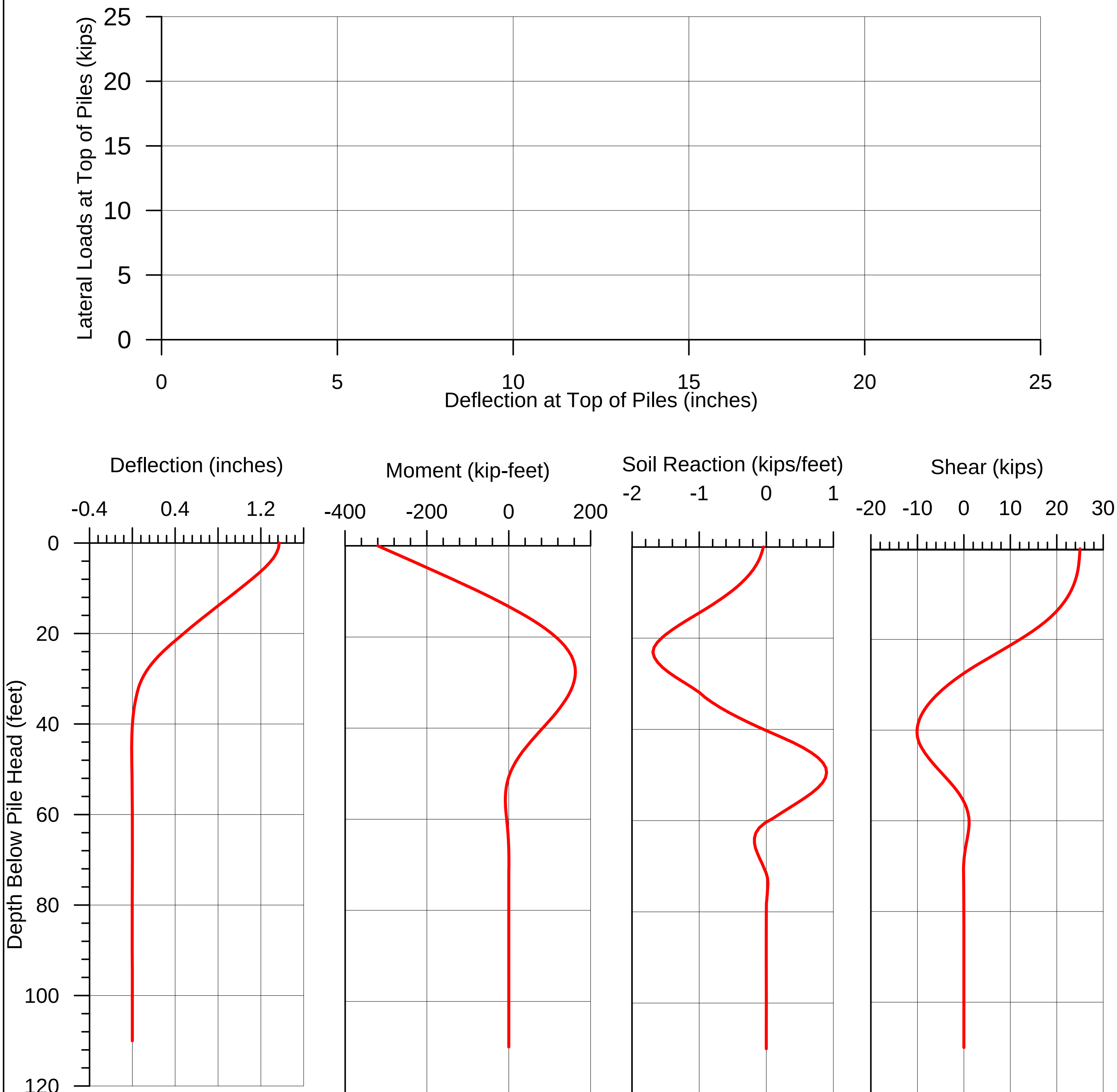
RESPONSE TO LATERAL LOADING
PROPOSED OVERLOOK RAMP 2
PIER 70 - CRANE COVE PARK - PHASE II
SAN FRANCISCO, CALIFORNIA

Project No.: KK-0210

Date: APRIL 2014

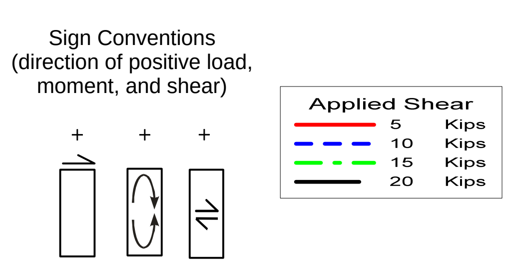
Plate

13



Notes:

The analyses were performed for 24-inch free-head CIDH piles.
This evaluation applies to piles extending to the design tip elevation.
This plate may be used for vertical load of 20 kips.



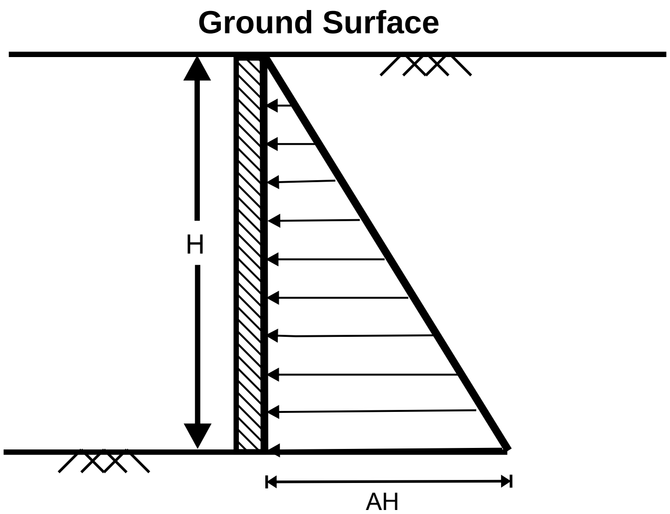
RESPONSE TO LATERAL LOADING
PROPOSED RETAINING WALL
PIER 70 - CRANE COVE PARK - PHASE II
SAN FRANCISCO, CALIFORNIA

Project No.: KK-0210

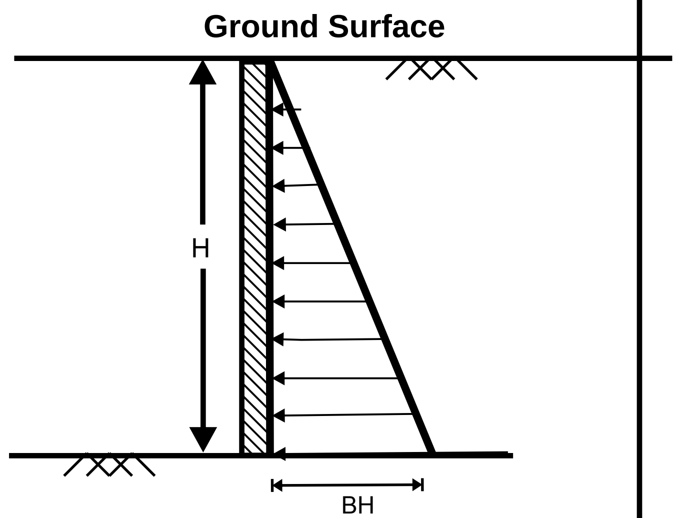
Date: APRIL 2014

Plate

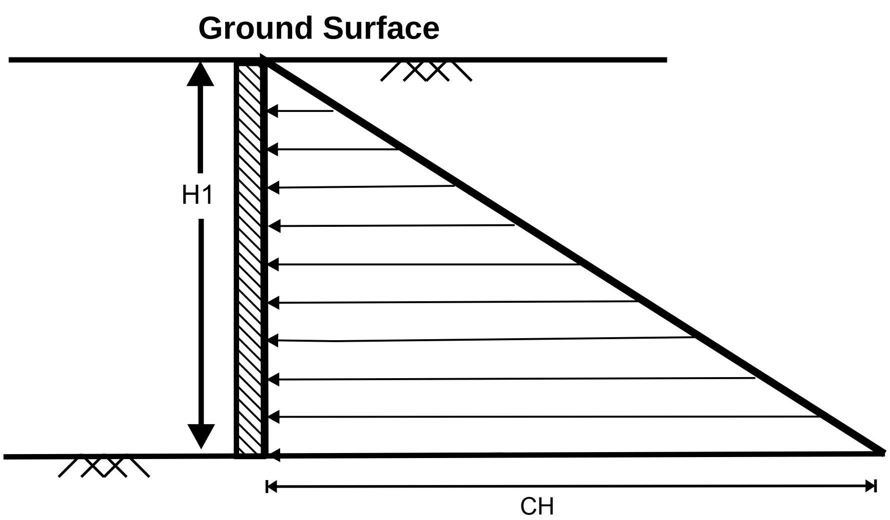
14



Non-yielding (At Rest) Wall Pressure (psf)



Active Wall Pressure (psf)



Passive Wall Pressure (psf)

NOTES:

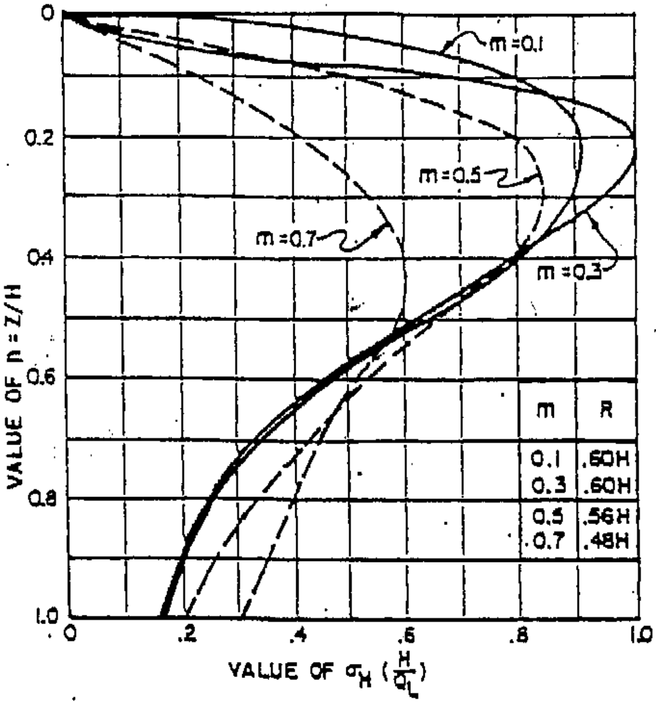
1. Above distributions apply to walls that are backfilled with fill material.

2. When multiplied by H (in feet), coefficients yield lateral earth pressures in pounds per square foot (psf).
   A = 60, B = 40, C = 300

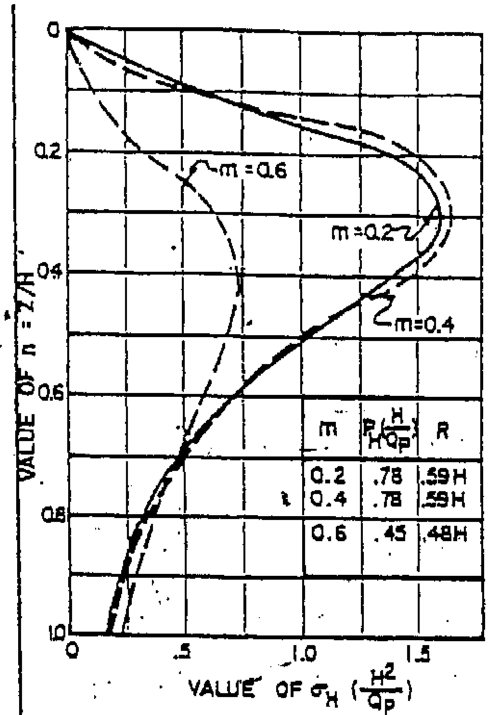
3. Passive pressure acting on wall or footing must be calculated for assumed wall deflection. Ignore upper foot of passive pressure when estimating lateral resistance of shallow foundations.

4. Pressure diagrams shown in this figure are appropriate for the design of buried structure walls.

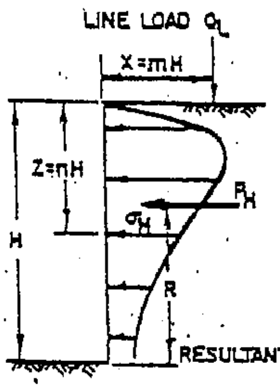
LATERAL EARTH PRESSURES **Geotechnical Study** **Pier 70 - Crane Cove Park - Phase II** **San Francisco, CA**		AGS CONSULTING ENGINEERS
JOB NO. AGS KK-0210	DATE: APRIL 2014	PLATE 15



m	R
0.1	.60H
0.3	.60H
0.5	.56H
0.7	.48H



m	$P_H\left(\frac{H}{Q_p}\right)$	R
0.2	.78	.59H
0.4	.78	.59H
0.6	.45	.48H



FOR $m \leq 0.4$:

$$\sigma_H\left(\frac{H}{Q_L}\right) = \frac{0.20n}{(0.16+n^2)^2}$$

$$P_H = 0.55 Q_L$$

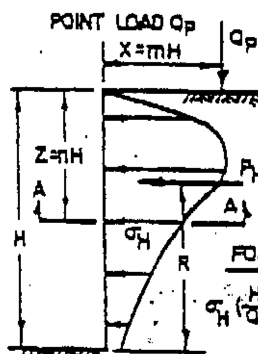
FOR $m > 0.4$:

$$\sigma_H\left(\frac{H}{Q_L}\right) = \frac{1.28 m^2 n}{(m^2+n^2)^2}$$

RESULTANT $P_H = \dfrac{0.64 Q_L}{(m^2+1)}$

PRESSURES FROM LINE LOAD Q_L

(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



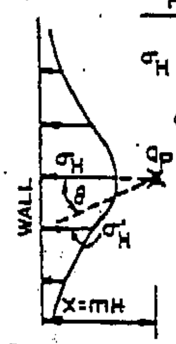
FOR $m \leq 0.4$:

$$\sigma_H\left(\frac{H^2}{Q_p}\right) = \frac{0.28n^2}{(0.16+n^2)^3}$$

FOR $m > 0.4$:

$$\sigma_H\left(\frac{H^2}{Q_p}\right) = \frac{1.77 m^2 n^2}{(m^2+n^2)^3}$$

$$\sigma_H' = \sigma_H \cos^2(1.1\theta)$$



SECTION A-A
PRESSURES FROM POINT LOAD Q_p
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)

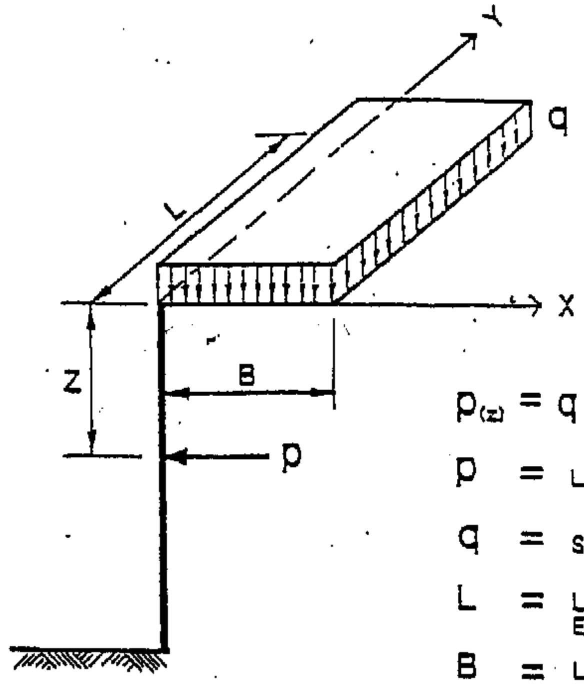
Reference: MARSTON, N. 1930

LATERAL SURCHARGE PRESSURES
POINT AND LINE LOADS

Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, CA

AGS CONSULTING ENGINEERS

JOB NO. AGS KK-0210	DATE: APRIL 2014	PLATE 16



$p_{(z)} = q \times I_p$

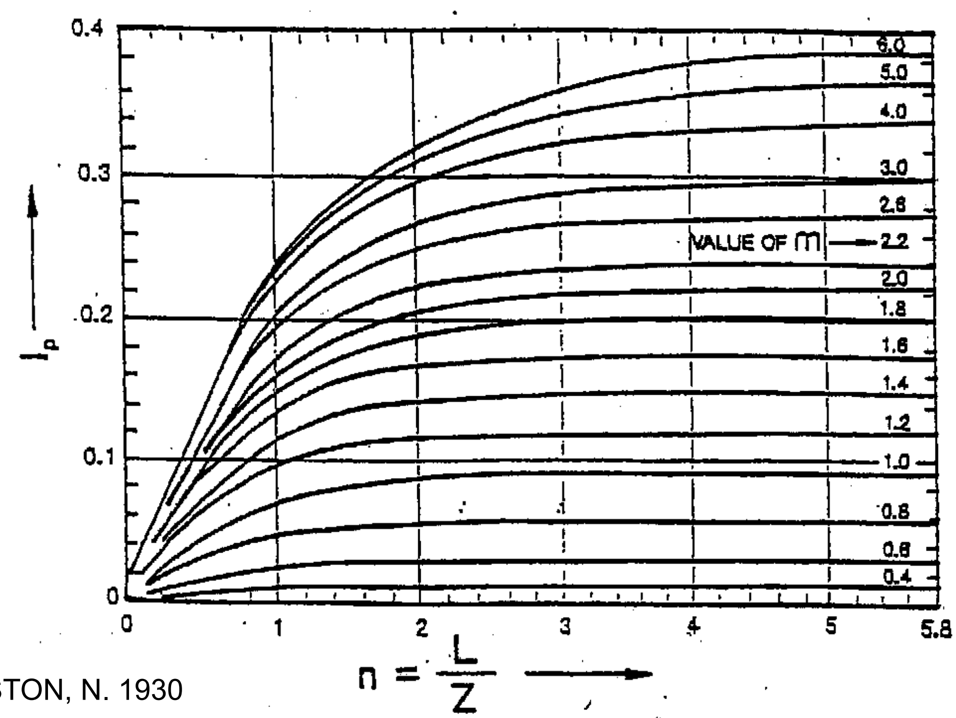
p = LATERAL PRESSURE

q = SURCHARGE

L = LENGTH PARALLEL TO EXCAVATION

B = LENGTH PERPENDICULAR TO EXCAVATION

$$m = \frac{B}{Z}, \quad n = \frac{L}{Z}$$



Reference: MARSTON, N. 1930

LATERAL SURCHARGE PRESSURES
AREAL LOADS

Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, CA

AGS CONSULTING ENGINEERS

JOB NO. AGS KK-0210	DATE: APRIL 2014	PLATE 17

APPENDIX A

FIELD EXPLORATION AND SAMPLING

APPENDIX A

FIELD EXPLORATION AND SAMPLING

A.1 EXPLORATION

AGS completed a site reconnaissance to evaluate the site access, ingress and egress, and to mark the proposed boring locations. After selecting the exploratory boring locations, AGS obtained drilling permit # 5543 through the City and County of San Francisco Department of Public Health, Environmental Services Division. AGS then received the final encroachment permit through the Port of San Francisco, E2014-P70-0009. In addition, AGS notified Underground Service Alert (USA) so that utility companies could mark their underground utilities, and warn AGS (USA Ticket #'s 0047808 and 0047818).

The field exploration program was performed under the technical supervision of a qualified AGS geologist, whom completed a log of each boring, documented the drilling progress, and recorded the subsurface conditions encountered at the location of each boring. The location of the exploratory borings is shown on Plate 2, Approximate Boring Location Map, including the locations of previously borings drilled by AGS and others. Eleven (11) exploratory borings, Boring 3 through Boring 13, were drilled for this project site. The deep borings, Borings B-3 through B-10, were extended to depths ranging from approximately 29.5 to 110.5 feet below the existing ground surface (bgs). Three shallow borings, Borings B-11 through B-13, were extended to depths of approximately 6 to 11 feet bgs. Drilling was performed by Geo-Ex Subsurface Exploration of Dixon, California from February 12th through February 14th, and from February 17th through February 21st, using a truck-mounted CME 75 drill rig. The borings were initially advanced using 7-inch diameter hollow stem augers, but once groundwater and suitable materials were encountered to advance conductor casing they switched to rotary wash (mud rotary) drilling.

Following the completion of each boring, they were backfilled with cement grout per requirements of the City and County San Francisco Department of Public Health, Environmental Health Section permit requirements. The County Inspector observed the grouting operation. Following backfilling and allowance of settlement, the ground surface at each boring location was restored to grade. A total of three 55-gallon steel drums were used to temporarily store the soils waste generated during drilling operation prior to off-haul and disposal. The drums were off-hauled from the site under a non-hazardous manifest on March 28, 2014, and transported to an approved landfill after performing analytical analysis on the soil cuttings. A composite sample was collected from the three drums and was submitted under chain of custody to Curtis and Tompkins Laboratory of Berkeley, California for analytical testing.

The subsurface conditions encountered in the borings were continuously logged in the field during drilling operations by a geologist from AGS. Plates A-1.1 through A-1.11 show the Logs of Borings B-1 through B-13, and give descriptions and graphic representations of the encountered materials, the depths at which samples were obtained, and the laboratory tests performed. The legend to the logs is shown on Plate A-2 - Soil Classification Chart and Key to Test Data, and Plate A-3 - Physical Characteristics of Rock.

A.2 SAMPLING

Soil and rock samples, as appropriate for the various earth materials encountered, were collected using standard penetration test (SPT), modified California (MC) samplers, and Shelby Tube samplers. Samples were typically collected at least once in each 5-foot depth interval.

Relatively undisturbed soil samples obtained with the MC sampler were collected into both 2-inch and 2.5-inch outside diameter by 6-inch long brass or stainless steel liners. The liners were immediately capped, sealed with vinyl tape, and labeled. Soil samples collected from the SPT sampler were placed into plastic bags and labeled. The Shelby Tube samples were stored in 30-inch long Shelby Tubes, capped and sealed. All the liners were kept upright and cushioned from shock. Following collection the samples were preserved in a cool and dark area until delivery to AGS' laboratory or other testing laboratory for examination and analyses.

The SPT and MC samplers were driven with a hydraulically-operated automatic 140-pound hammer, falling 30 inches for an 18-inch penetration, where possible. The blows required to advance the samplers were used to assist in classifying the apparent density of cohesionless soil deposits, and the relative consistency of cohesive soil deposits. The blow counts required to drive the sampler for each 6-inch increment were recorded; except where refusal was met, in which case the number of inches penetrated by 50 blows (typically) was recorded. The blow counts are shown on the Logs of Borings in Appendix A-1. The blow counts shown on the Logs of Borings are the numbers recorded in the field, and have not been corrected or adjusted.

LOG OF BORING B-03	DRILLING DATE: 2/19/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: -4.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
0					**CONCRETE**,6.5 feet thick					
5										
6.5					**WELL-GRADED GRAVEL WITH SAND (GW)** gray, moist to wet, very dense, coarse-grained gravel, cobbles, concrete rubble [FILL]					
8		1	50/2"							
10										
12		2	1 20 4		**WELL-GRADED SAND WITH GRAVEL (SW)**, gray, wet, medium dense, fine-grained subrounded to subangular gravel, fine to coarse-grained sand, concrete rubble [FILL]		20			SA (4)
14		3	3 6 14							
15										
18					**FAT CLAY (CH)**, black, wet, soft, few subangular fine to coarse-grained gravel, petrol odor [YOUNG BAY MUD] - switched to mud rotary at 18.5 feet bgs.					
20		4	24"/ 24" Rec.					85	48	CON
25					- changed to dark gray, few fine-grained sand, trace gravel					
30										
35										
40										

LOG OF BORING B-03

DRILLING DATE: 2/19/14
DRILLING METHOD: 7" HSA/Mud Rotary
DRILL RIG TYPE: CME 75
HAMMER TYPE: 140-lb, falling 30 inches

SURFACE ELEVATION: -4.0 ft
DATUM: City of San Francisco
LOGGED BY: JF
CHECKED BY: MV

AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					FAT CLAY (CH) CONTINUED, very dark gray, soft, few fine-grained sand [YOUNG BAY MUD]					
45		5 6	5 10 8 4 3 3		**POORLY GRADED SAND WITH CLAY (SP-SC)**, dark gray, wet, loose to medium-dense, fine to medium-grained sand, few fine-grained subangular gravel, trace clay		40			SA (7)
50					**SANDY LEAN CLAY (CL)**, yellowish brown, wet, stiff, some fine-grained sand, few fine-grained subangular gravel					
55		7	6 7 14			110	18	24	8	WA (57) UC (1.9) TV (.25)
60										
65		8	13 14 16		- changed to hard					
					FAT CLAY WITH SAND (CH), gray to bluish gray with light brown mottling, wet, very stiff, fine-grained sand					
70		9	6 6 10							
					- changed to bluish gray with red mottling					
75		10	8 10 14							
80										

LOG OF BORING B-03	DRILLING DATE: 2/19/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: -4.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					FAT CLAY WITH SAND (CH) CONTINUED, gray to bluish gray with red mottling, wet, very stiff, fine-grained sand, few fine-grained subangular gravel					
					CLAYEY SAND (SC), yellowish brown and gray, wet, medium dense, fine to medium-grained sand					
85		11	7 11 13			108	18	37	17	WA (38) UC (2.3)
90										
95					**SANDY FAT CLAY (CH)**, olive and greenish gray, wet, stiff, sand lenses, few fine subangular gravel					
100		12	4 4 7							
105					**SANDY FAT CLAY WITH GRAVEL (CH)** , bluish green to greenish gray, wet, very stiff, fine to coarse-grained subangular gravel, few fine-grained sand					
110		13	15 19 21							
					Boring was terminated at a depth of approximately 110.5 feet bgs. Groundwater level measured at a depth of approximately 7.6 feet bgs. Switched From hollow stem auger to mud rotary at 18.5 feet bgs. Hole was backfilled with cement grout after completion of drilling.					
115										
120										

LOG OF BORING B-04	DRILLING DATE: 2/12/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 2.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE NO.	BLOW COUNT	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
0			**POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM)**, light brown, moist, medium dense, fine to coarse-grained subangular gravel, few fine to medium-grained sand, few cobbles, trace clay [FILL]					
	1	11 6 4			14			COR
4			**CLAYEY SAND WITH GRAVEL (SC)**, dark brown to gray brown, moist, very loose, fine to coarse-grained sand, fine to coarse-grained subangular gravel (mostly sepentinite) [FILL]					SA (15)
5	2	push 2 2						
			▽					
	3	push 1 1	- changed to very dark brown to black, wet					
10	4	16 65/6"	**WELL-GRADED GRAVEL WITH CLAY AND SAND (GW-GC)**, dark brown to black, wet, fine to coarse-grained subangular gravel, fine to coarse-grained sand, wood [FILL]					
15			- switched to mud rotary at 15 feet bgs.					
	5	16"/ 24" Rec.				87	52	
17			**FAT CLAY (CH)**, black, wet, very soft, few subangular fine to coarse-grained gravel, petrol odor [YOUNG BAY MUD]					
20			- changed to dark gray					
	6	20"/ 24" Rec.						
25								
30	7	1 1 1						
35	8	19"/ 24" Rec.	- changed to few fine-grained sand, few shells					
40								

LBG 30 KK0210-PIER 70-PHASE 2.GPJ 3/21/14

LOG OF BORING B-04	DRILLING DATE: 2/12/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 2.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					FAT CLAY (CH) CONTINUED, dark gray, wet, very soft, few fine-grained sand, few shells [YOUNG BAY MUD]					
45										
					SANDY FAT CLAY (CH), dark gray to greenish gray, wet, soft to medium stiff, fine-grained sand lenses, organic odor [YOUNG BAY MUD]					
50		9	4 5 6							
		10	1 1 1							
55										
					FAT CLAY (CH), light gray, wet, stiff, few fine-grained sand [YOUNG BAY MUD]					
60		11	6 9 10							
					SILTY/CLAYEY SAND (SC/SM), yellowish brown, wet, medium dense, fine to medium-grained sand, few fine-grained angular gravel					
65										
70		12	23 34 33		**POORLY GRADED SAND (SP)**, yellowish brown, wet, dense, fine to medium-grained sand, few fine-grained angular gravel, many shells					
75		13	14 21 12		- changed to olive to gray, shell fragments		32			WA (3)
80										

LOG OF BORING B-04	DRILLING DATE: 2/12/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 2.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					POORLY GRADED SAND (SP) CONTINUED, olive to gray, moist, dense, fine to medium-grained sand, few fine-grained angular gravel, few shell fragments					
					SANDY LEAN CLAY (CL), yellowish brown, moist, very stiff, fine-grained sand, trace shells, dark speckling					
85		14	7 7 11							
90					**FAT CLAY WITH SAND (CH)**, yellowish brown, moist, stiff, fine to medium-grained sand, few subangular gravel, shale rock fragments					
95		15	5 7 5							
100										
					POORLY GRADED SAND (SP), yellowish brown, wet, medium dense, fine to medium-grained sand					
105		16	5 6 8		**FAT CLAY WITH SAND (CH)**, bluish gray, stiff, medium to coarse-grained sand, few fine-grained angular gravel, shell lenses					
					Boring was terminated at a depth of approximately 105.5 feet bgs. Groundwater level measured at a depth of approximately 8 feet bgs. Switched From hollow stem auger to mud rotary at 15 feet bgs. Hole was backfilled with cement grout after completion of drilling.					
110										
115										
120										

LOG OF BORING B-05

DRILLING DATE: 2/13/14
DRILLING METHOD: 7" HSA/Mud Rotary
DRILL RIG TYPE: CME 75
HAMMER TYPE: 140-lb, falling 30 inches

SURFACE ELEVATION: -5.5 ft
DATUM: City of San Francisco
LOGGED BY: JF
CHECKED BY: MV

AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE NO.	BLOW COUNT	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
0			**POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC)**, yellowish brown, moist, loose, fine to coarse-grained sand, "oily" [FILL]					
	1	10 8 7		111	28			SA (11)
	2	2 3 3						COR
5			- changed to dark brown and black [FILL]					
8	3	8 3 2	**CLAYEY GRAVEL WITH SAND (GC)**, black, wet, loose, fine to coarse-grained angular gravel, fine to coarse-grained sand, little clay, slight organic odor, "oily" [FILL]					
10			**CLAYEY GRAVEL (GC)**, black, wet, fine to coarse-grained subangular gravel, fine to coarse-grained sand, wood [FILL]					
	4							TXU
15			- switched to mud rotary at 15 feet bgs.					
	5	23"/ 24" Rec.	**FAT CLAY (CH)**, black, wet, very soft, little silt, few subangular fine to coarse-grained gravel, petrol odor [YOUNG BAY MUD]					
20								
			- changed to dark gray					
25								
30	6	24"/ 24" Rec.			76	83	53	
35			- changed to few sand, few shells					
40								

LBG 30 KK0210-PIER 70-PHASE 2.GPJ 3/21/14

LOG OF BORING B-05	DRILLING DATE: 2/13/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: -5.5 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					FAT CLAY (CH) CONTINUED, dark gray, wet, very soft, few fine-grained sand, few shells [YOUNG BAY MUD]					
45		7	push 1 1							
50										
					FAT CLAY WITH SAND (CH), greenish gray with dark mottling, wet, very stiff, few fine-grained sand, few silt					
55		8	6 7 10							
60										
		9	3 3 7		**CLAYEY SAND WITH GRAVEL (SC)**, light brown, wet, loose, fine to coarse-grained sand, few fine to coarse-grained angular gravel, with shells	110	35			WA (43) TXU
65										
70										
					FAT CLAY (CH), greenish gray and black, wet, soft, trace fine-grained sand, organic material					
		10	1 2 2							
75										
80										

LOG OF BORING B-05	DRILLING DATE: 2/13/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: -5.5 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					CLAYEY SAND WITH GRAVEL (SC), yellowish brown and gray, wet, medium dense, fine to coarse-grained sand, fine to coarse-grained angular gravel					
85		11	8 9 15							
90					**FAT CLAY (CH)**, greenish gray, wet, very stiff					
		12	5 6 9							
95										
100										
105					**SERPENTINITE**, dark green and light green with reddish brown staining, sheared, alternately soft and hard					
		13	86/6"							
110					Boring was terminated at a depth of approximately 108.5 feet bgs. Groundwater level measured at a depth of approximately 8 bgs. Switched From hollow stem auger to mud rotary at 15 feet bgs. Hole was backfilled with cement grout after completion of drilling.					
115										
120										

LOG OF BORING B-06	DRILLING DATE: 2/21/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 1.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					WELL-GRADED GRAVEL WITH SAND (GW), gray-brown, dry, loose, coarse-grained subangular gravel, fine to coarse-grained sand, little silt [FILL]					
5		1	6 7 8		**POORLY GRADED SAND WITH GRAVEL (SP)**, brown and black, moist, medium dense, medium to coarse-grained sand, few fine-grained subangular gravel [FILL]					WA (18)
					- changed to dark brown					
10		2	push		**FAT CLAY (CH)**, dark gray, wet, very soft, few subangular gravel, organic odor [YOUNG BAY MUD]					
15		3	22"/ 24" Rec.		- switched to mud rotary at 14 feet bgs.			80	50	TV (.14)
					Torvane = 0.14 tsf					
					- changed to no gravel at 17 feet bgs					
20										
25										
30										
35										
40										

LBG 30 KK0210-PIER 70-PHASE 2.GPJ 3/21/14

LOG OF BORING B-06	DRILLING DATE: 2/21/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 1.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
		4	22"/ 24" Rec.		**FAT CLAY (CH) CONTINUED**, dark gray, wet, very soft, trace fine-grained sand, few shells [YOUNG BAY MUD]			66	49	
45										
50										
55										
60										
65										
70		5	3 3 2		- changed to soft, few lenses of fine-grained sand, few caliche		70	81	49	WA (90)
75		6	2 1 2		- changed to very soft, trace shells, no sand					
80										

LOG OF BORING B-06	DRILLING DATE: 2/21/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 1.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					FAT CLAY (CH) CONTINUED, dark gray, wet, very soft, trace shells [YOUNG BAY MUD] - changed to stiff					
					FAT CLAY (CH), greenish gray, wet, very stiff, few fine to medium-grained sand, few coarse-grained sand, few fine-grained subangular gravel, charcoal spotting					
85		7	5 9 16							
		8	9 12 13							
90										
					FAT CLAY WITH SAND (CH), greenish gray and yellowish brown, wet, very stiff					
95		9	6 12 13		Torvane = 0.65 tsf					TV (.65)
					Boring was terminated at a depth of approximately 95.5 feet bgs. Groundwater level not measured. Switched From hollow stem auger to mud rotary at 14 feet bgs. Hole was backfilled with cement grout after completion of drilling.					
100										
105										
110										
115										
120										

LOG OF BORING B-07	DRILLING DATE: 2/18/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 1.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					SILTY/CLAYEY SAND WITH GRAVEL (SC/SM), grayish brown, moist [FILL]					
5		1	4 7 5		- changed to dark brown to black and greenish gray, medium dense, fine to coarse-grained sand, fine to coarse-grained angular gravel, few cobbles					SA (12)
		2	5 6 5		- changed to wet		42			WA (14)
10		3	5 3 3		- changed to black, loose, organic odor, "oily"					
		4	push		**CLAYEY SAND WITH GRAVEL (SC)**, black, wet, loose, fine to coarse-grained subangular gravel, fine to coarse-grained sand,wood, "oily"					
15		5	24"/ 24" Rec.		- switched to mud rotary at 15 feet bgs. **FAT CLAY WITH SAND (CH)**, dark gray, wet, soft, little fine-grained sand [YOUNG BAY MUD] Torvane = 0.20 tsf			98	64	TV (.20)
20										
					FAT CLAY/PEAT (CH/OH), dark gray, wet, very soft, strong organic odor, with brown stringers of plant matter [YOUNG BAY MUD/ MARSH DEPOSIT]					
25		6	push 1 1		Torvane = 0.20 tsf					TV (.20)
30										
35										
		7	5 8 8		**LEAN CLAY WITH SAND AND GRAVEL (CL)**, yellowish brown to olive, wet, stiff, little fine-grained sand, little fine-grained subangular gravel Torvane = 0.40 tsf	119	19	42	29	UC (2.2) TV (.40)
40										

LOG OF BORING B-07	DRILLING DATE: 2/18/14 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 1.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					LEAN CLAY WITH SAND AND GRAVEL (CL) CONTINUED, yellowish brown to olive, wet, very stiff, little fine-grained sand, little fine-grained subangular gravel					
45										
					CLAYEY GRAVEL WITH SAND (GC), greenish gray and yellowish brown, wet, medium dense, severely weathered fragments of sandstone rock					
50		8	8 12 22							
					SANDSTONE, greenish gray and light brown, moderately hard, apparently strong					
		9	50/ 6"							
					Boring was terminated at a depth of approximately 53 feet bgs. Groundwater level measured at a depth of approximately 6 feet bgs. Switched From hollow stem auger to mud rotary at 15 feet bgs. Hole was backfilled with cement grout after completion of drilling.					
55										
60										
65										
70										
75										
80										

LOG OF BORING B-08

DRILLING DATE: 2/14/14
DRILLING METHOD: 7" HSA/Mud Rotary
DRILL RIG TYPE: CME 75
HAMMER TYPE: 140-lb, falling 30 inches

SURFACE ELEVATION: 2.5 ft
DATUM: City of San Francisco
LOGGED BY: JF
CHECKED BY: MV

AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					WELL-GRADED GRAVEL WITH SILT AND SAND (GW-GM), grayish brown, dry, medium dense [FILL]					
		BULK 1	4 7 6		**SILTY/CLAYEY SAND WITH GRAVEL (SC/SM)**, black and greenish gray, moist, mediumm dense, fine to coarse-grained sand, fine and coarse-grained subangular gravel, "oily" [FILL]					SA (30)
5		2	6 5 6		**CLAYEY SAND WITH GRAVEL (SC)**, black and greenish gray, wet, loose, fine to coarse-grained sand, fine to coarse-grained angular gravel, few cobbles, little clay, petrol odor, "oily" [FILL]	99	18			WA (20) TXU
		3	4 3 4							
10										
15		4	12"/ 24" Rec.		- switched to mud rotary at 14 feet bgs.			47	29	TV (.20)
					FAT CLAY WITH SAND (CH), dark gray, soft, little fine to coarse-grained sand, few subrounded gravel [YOUNG BAY MUD] Torvane = 0.20 tsf					
20		5	21"/ 24" Rec.							
		6	3 4 19		**SANDY FAT CLAY WITH GRAVEL (CL)**, greenish gray and brown, very stiff, fine to coarse-grained angular gravel (mostly serpentinite)					
25		7	64/ 6"		**SANDSTONE**, greenish gray and light brown, moderately hard, apparently strong					
30		8	61/ 6"		Boring was terminated at a depth of approximately 29.5 feet bgs. Groundwater level measured at a depth of approximately 7.5 feet bgs. Switched From hollow stem auger to mud rotary at 14 feet bgs. Hole was backfilled with cement grout after completion of drilling.					
35										
40										

JOB NO. KK-0210 | PROJECT: Pier 70 - Crane Cove Park | SHEET 1 OF 1 | PLATE A-1.6

LBG 30 KK0210-PIER 70-PHASE 2.GPJ 3/21/14

LOG OF BORING B-09	DRILLING DATE: 2/20/14 DRILLING METHOD: 7" HSA DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 3.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					Asphaltic Concrete - 8 inches thick [FILL]					
					TREATED WOOD, possible old railroad tie					
					SILTY SAND WITH GRAVEL (SM), brown, loose, little fine to coarse-grained angular gravel [FILL]					
5		1	5 3 3		**SILTY/CLAYEY SAND WITH GRAVEL (SC/SM)**, greenish gray, moist, loose, fine to coarse-grained sand, fine to coarse-grained subangular gravel (mostly serpentinite) [FILL] - changed to dark brown, less clay, mostly fine to medium-grained sand					
10		2	7 6 4		- changed to yellowish brown, finely bedded, few weathered sandstone and shale fragments - changed to grayish brown, wet, little fine to coarse-grained dark colored subangular gravel, mostly fine to medium-grained sand		14			WA (23)
15		3	2 1 2				27			SA (21)
20		4	6 3 4		**WELL-GRADED GRAVEL WITH CLAY AND SAND (GW-GC)**, greenish gray, wet, loose, fine to coarse-grained sand, fine to coarse-grained angular subangular gravel (mostly serpentinite), few cobbles		16			SA (6)
25		5	3 4 6		- changed to little orange staining, more weathered to coarse-grained sand					
30		6	6 15 18		**WELL-GRADED SAND WITH CLAY AND GRAVEL (SW-SC)**, dark gray, wet, dense, mostly coarse-grained sand, fine to coarse-grained subangular gravel, including serpentinite, sandstone and shale fragments		18			WA (10)
35		7	4 11 4				17			SA (12)
40		8	22 23 22		**SANDY LEAN CLAY WITH GRAVEL (CL)**, olive, wet, hard, fine-grained sand, little fine to coarse-grained subangular gravel					

LOG OF BORING B-09	DRILLING DATE: 2/20/14 DRILLING METHOD: 7" HSA DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 3.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					SANDY LEAN CLAY WITH GRAVEL (CL) CONTINUED, olive, wet, hard, fine-grained sand, little fine to coarse-grained subangular gravel					
45					**SANDSTONE**, greenish gray and light brown, moderately hard, apparently strong					
		9	43 47 50/4"							
50					Boring was terminated at a depth of approximately 49.3 feet bgs. Groundwater level measured at a depth of approximately 9.7 feet. Hole was backfilled with cement grout after completion of drilling.					
55										
60										
65										
70										
75										
80										

LOG OF BORING B-10	DRILLING DATE: 1/18/12 DRILLING METHOD: 7" HSA/Mud Rotary DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: 2.5 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
0		BULK			**SILTY GRAVEL WITH SAND (GM)**, brown, loose, little fine to coarse-grained angular gravel [FILL]					
2					**SILTY SAND WITH GRAVEL (SM)**, dark brown to black, moist, medium dense, mostly fine-grained sand, little fine to coarse-grained subangular gravel, slight petrol odor [FILL]					
5										
7		1	7 2 3		- changed to wet, loose, few clay					WA (12)
10					- switched to mud rotary at 10 feet bgs.					
12					**CLAYEY GRAVEL WITH SAND (GC)**, black, wet, loose, fine to coarse-grained subangular gravel, fine to coarse-grained sand, wood					
15										
16					**FAT CLAY (CH)**, dark gray, wet, soft [YOUNG BAY MUD]					
18		2	24"/ 24" Rec.					70	43	CON
20										
25										
30										
35										
40										

LOG OF BORING B-10

DRILLING DATE: 1/18/12
DRILLING METHOD: 7" HSA/Mud Rotary
DRILL RIG TYPE: CME 75
HAMMER TYPE: 140-lb, falling 30 inches

SURFACE ELEVATION: 2.5 ft
DATUM: City of San Francisco
LOGGED BY: JF
CHECKED BY: MV

AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					FAT CLAY (CH) CONTINUED, dark gray, wet, soft [YOUNG BAY MUD]					
					SANDY LEAN CLAY (CL), yellowish brown, wet, very stiff, fine-grained sand, few fine-grained subangular gravel					
45		3	7 9 12			102	22			UC (1.6)
50										
					- changed to light brown to olive-gray, stiff, few fine-grained subrounded to subangular gravel					
55		4	6 4 5							
60										
					SANDY FAT CLAY (CH), light gray and yellowish brown, wet, soft, little fine to coarse-grained sand, few fine-grained angular gravel			80	45	
65		5	3 2 3							WA (72)
70										
					SHALE AND SILTSTONE, dark gray to black, hard, clay in fractures					
75		6	50/6"							
					Boring was terminated at a depth of approximately 74.5 feet bgs. Groundwater level measured at a depth of approximately 8.5 feet bgs. Switched From hollow stem auger to mud rotary at 10 feet bgs. Hole was backfilled with cement grout after completion of drilling.					
80										

LOG OF BORING B-11

DRILLING DATE: 2/14/14
DRILLING METHOD: 7" HSA
DRILL RIG TYPE: CME 75
HAMMER TYPE: 140-lb, falling 30 inches

SURFACE ELEVATION: -5.0 ft
DATUM: City of San Francisco
LOGGED BY: JF
CHECKED BY: MV

AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					CONCRETE - 3.5 feet thick					
5		1	2 1 10		**CLAYEY SAND WITH GRAVEL (SC)**, black, moist, medium dense, strong creosote odor, wood, "oily" [FILL]					
		2	2 1 1		**FAT CLAY (CH)**, black, wet, very soft, few subangular fine to coarse-grained gravel [YOUNG BAY MUD]					
		3	psh psh							
10		4	psh		- changed to very dark gray, trace gravel					
					Boring was terminated at a depth of approximately 10.5 feet bgs. Groundwater level measured at a depth of approximately 5 feet. Hole was backfilled with cement grout after completion of drilling.					
15										
20										
25										
30										
35										
40										

LBG 30 KK0210-PIER 70-PHASE 2.GPJ 3/21/14

LOG OF BORING B-12	DRILLING DATE: 2/14/14 DRILLING METHOD: 7" HSA DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: -6.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
0					POORLY GRADED SAND (SP), yellowish brown, moist, medium-grained [BEACH SAND ON TOP OF CONCRETE]					
					CONCRETE - 4 feet thick					
4					▽					
5		1	10/4" 50/2"		**POORLY GRADED GRAVEL WITH CLAY (GP-GC)**, brown to gray, wet, very dense [FILL]					
		2	10/6" 50/3"							
10		3	2 1 1		**FAT CLAY WITH GRAVEL (CH)**, black, wet, very soft, little subangular fine to coarse-grained gravel [YOUNG BAY MUD]					
11					Boring was terminated at a depth of approximately 11 feet bgs. Groundwater level measured at a depth of approximately 4 feet. Hole was backfilled with cement grout after completion of drilling.					
15										
20										
25										
30										
35										
40										

LOG OF BORING B-13	DRILLING DATE: 2/14/14 DRILLING METHOD: 7" HSA DRILL RIG TYPE: CME 75 HAMMER TYPE: 140-lb, falling 30 inches	SURFACE ELEVATION: -3.0 ft DATUM: City of San Francisco LOGGED BY: JF CHECKED BY: MV	AGS, Inc. Consulting Engineers

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	ADDITIONAL TESTS
					CONCRETE - 6 feet thick					
5										
					Refusal encountered at a depth of approximately 6 feet bgs. Groundwater not encountered. Hole was backfilled with cement grout after completion of drilling.					
10										
15										
20										
25										
30										
35										
40										

LBG 30 KK0210-PIER 70-PHASE 2.GPJ 3/21/14

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

	Modified California	RV	R-Value
	Standard Penetration Test	SA	Sieve Analysis
	Pitcher Barrel	SW	Swell Test
	NX Core Barrel	TC	Cyclic Triaxial
	Bulk Sample	TX	Unconsolidated Undrained Triaxial
	Sample Attempt with No Recovery	TV	Torvane Shear
COR	Corrosivity	UC	Unconfined Compression
CN	Consolidation	(1.2)	(unconfined compressive strength, tsf)
CP	Compaction	WA	Wash Analysis
DS	Direct Shear	(20)	(with % Passing No. 200 Sieve)
PM	Permeability		Water Level at Time of Drilling
PP	Pocket Penetrometer		Water Level after Drilling(with date measured)

ADDITIONAL TESTS AND KEY TO TEST DATA

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

Geotechnical Study
Pier 70 - Crane Cove Park - Phase II
San Francisco, CA

AGS, Inc.
Consulting Engineers

KK0210	**Date:** May 2014	**PLATE A-2**

PHYSICAL CHARACTERISTICS OF ROCKS
FIELD TERMINOLOGY

HARDNESS		STRENGTH	
VERY SOFT:	EASILY CRUMBLED OR DEFORMED BY HAND.	PLASTIC:	CAN BE DEFORMED BY HAND.
SOFT:	MAY BE BROKEN USING BOTH HANDS OR IF PLASTIC, DEFORMED BY HAND. MAY BE CUT WITH DIFFICULTY WITH KNIFE. EASILY POWDERED WITH PICK. DULL THUD WHEN STRUCK WITH HAMMER.	FRIABLE:	CRUMBLES BY RUBBING WITH FINGERS.
		WEAK:	AN UNFRACTURED OUTCROP OF SUCH MATERIAL WOULD CRUMBLE UNDER LIGHT HAMMER BLOWS.
MODERATELY HARD:	MAY BE SCRATCHED WITH KNIFE TO SHALLOW DEPTH. DULL RING WHEN STRUCK WITH HAMMER.	MODERATELY STRONG:	OUTCROP WOULD WITHSTAND A FEW FIRM HAMMER BLOWS BEFORE BREAKING.
HARD:	CAN BE SCRATCHED WITH DIFFICULTY WITH KNIFE.	STRONG:	OUTCROP WOULD WITHSTAND A FEW HEAVY HAMMER BLOWS, BUT WILL YIELD LARGE FRAGMENTS.
VERY HARD:	CANNOT BE SCRATCHED WITH KNIFE. SHARP RING WHEN STRUCK WITH HAMMER.	VERY STRONG:	OUTCROP WOULD RESIST HEAVY RINGING HAMMER BLOWS AND WILL YIELD, WITH DIFFICULTY, ONLY DUST AND SMALL FRAGMENTS.

WEATHERING

EXTENT	DECOMPOSITION	DISINTEGRATION	DISCOLORATION	FRACTURE CONDITION
SEVERE	MODERATE TO COMPLETE ALTERATION OF MINERALS, FELDSPARS ALTERED TO CLAY, ETC.	GENERALLY FRIABLE, BUT ROCK TEXTURE AND STRUCTURE ARE PRESERVED.	EXTENSIVE AND THOROUGH.	ALL FRACTURES EXTENSIVELY COATED WITH OXIDES, CARBONATES OR CLAY.
MODERATE	SLIGHT ALTERATION OF MINERALS, CLEAVAGE SURFACES LUSTERLESS AND STAINED.	MOST CEMENTATION IS AFFECTED; MAY BE LOCALLY FRIABLE.	MODERATE OR LOCALIZED AND INTENSE.	THIN COATINGS OR STAINS.
SLIGHT	NO MEGASCOPIC ALTERATION OF MINERALS.	LITTLE OF NO EFFECT ON NORMAL CEMENTATION.	SLIGHT TO INTERMITTENT AND LOCALIZED.	FEW STAINS ON FRACTURE SURFACES.
FRESH	UNALTERED, CLEAVAGE SURFACES GLISTENING.	CEMENTATION UNAFFECTED.	NO DISCOLORATION.	NO STAINS.

FRACTURING*

EXTENT	ENGLISH SIZE RANGE	METRIC SIZE RANGE
CRUSHED (MAY CONTAIN CLAY)	LESS THAN 0.05 FT	LESS THAN 1.5 CM
INTENSELY FRACTURED	0.05 TO 0.1 FT	1.5 CM TO 3.0 CM
CLOSELY FRACTURED	0.1 TO 0.5 FT	3.0 CM TO 15 CM
MODERATELY FRACTURED	0.5 TO 1.0 FT	15 CM TO 30 CM
LITTLE FRACTURED	1.0 TO 3.0 FT	30 CM TO 100 CM
MASSIVE	3.0 FT AND LARGER	1.0 M AND LARGER

FRACTURE COATINGS

EXTENT	THICKNESS
UNSTAINED OR CLEAN: NEARLY ALL SURFACES CLEAN.	STAINED: NO PERCEPTIBLE THICKNESS.
SMALL: COVERS LESS THAN 10% OF FRACTURE SURFACE.	THIN: BARELY PERCEPTIBLE.
MODERATE OR PATCHY: COVERS 10% TO 50% OF SURFACE.	MEDIUM: UP TO 2 MM.
EXTENSIVE: COVERS NORE THAN 50% OF SURFACE.	THICK: OVER 2 MM THICK.

STRATIFICATION

STRATIFICATION (OR PARTING)	CROSS-STRATIFICATION	APPROXIMATE THICKNESS**	
		ENGLISH	METRIC
VERY THICK-BEDDED (-PARTED)	VERY THICKLY CROSS-BEDDED	OVER 3 FT	OVER 1 M
THICK-BEDDED (-PARTED)	THICKLY CROSS-BEDDED	1 TO 3 FT	30 TO 100 CM
MEDIUM-BEDDED (-PARTED)	MEDIUM CROSS-BEDDED	4 TO 12 IN	10 TO 30 CM
THIN-BEDDED (-PARTED)	THINLY CROSS-BEDDED	1 TO 4 IN	3 TO 10 CM
VERY THIN-BEDDED (-PARTED)	VERY THINLY CROSS-BEDDED	0.5 TO 1 IN	1 TO 3 CM
LAMINATED (THINLY PARTED)	CROSS-LAMINATED	0.1 TO 0.5 IN	0.3 TO 1.0 CM
THINLY LAMINATED (VERY THINLY PARTED)	THINLY CROSS-LAMINATED	LESS THAN 0.1 IN	LESS THAN 0.3 CM

* JOINTS AND FRACTURES ARE TREATED THE SAME FOR PHYSICAL DESCRIPTION AND BOTH ARE REFERRED TO AS FRACTURES; SIZE RANGE REFERS TO SIZE OF PIECES.
** MASSIVE IF BEDS ARE OVER 10 FT (3 M) THICK.

RECOVERY = RECOVERY RATIO, RECOVERED LENGTH DIVIDED BY LENGTH OF RUN; EXPRESSED AS A PERCENTAGE.

RQD = SUM OF LENGTHS OF FULL DIAMETER, SOUND CORE 4” OR LONGER, DIVIDED BY LENGTH OF RUN; EXPRESSED AS A PERCENTAGE.

Geotechnical Study
Pier 70 - Crane Cove Park
San Francisco, CA

AGS, Inc.
Consulting Engineers

JOB NO: KK0210	January 2011		PLATE A-3

APPENDIX B

GEOTECHNICAL FIELD AND LABORATORY TESTING

B.1 GENERAL

Preliminary visual soil classifications were made by AGS in the field in accordance with ASTM D-2488-93, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Upon completion of drilling, the samples collected from the borings were taken to AGS' laboratory for examination and analyses. The soil classifications were verified by observation of the samples in the laboratory, and a testing program in accordance with ASTM D-2487-93, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System).

Geotechnical field and laboratory tests were performed on selected soil samples in order to evaluate the engineering properties of the materials. The tests performed in AGS' laboratory included moisture content, dry density, grain sieve distribution, passing No. 200, Atterberg limits, and unconfined compressive strength. Samples were submitted to Cooper Laboratory Testing in Palo Alto, California for consolidation, unconsolidated untrained testing, R-value, and corrosion testings. Details of the geotechnical laboratory testing program are included in Appendices B and C.

B.2 FIELD TESTING

The blows required to drive the samplers, using a 140-pound hammer falling 30 inches for an 18-inch penetration, were used to assist in classifying the relative density of cohesionless soil deposits and the stiffness of cohesive soil deposits. Pocket penetrometer test was conducted in the field. Blow counts recorded and pocket penetrometer test conducted by AGS in the field are shown on the Logs of Borings.

B.3 LABORATORY TESTING

The laboratory tests were performed using the techniques and procedures discussed below.

B.3.1 Particle Size

Particle size analyses were conducted on selected samples in accordance with ASTM D-422, Standard Test Method for Particle Size Analysis of Soils or ASTM D-1140, Standard Test

Method for Amount of Material in Soils Finer than the No. 200 (75-µm) Sieve. The results of the particle size and wash analyses are presented in Appendix B, Plates B-1.1 through B-1.3, Particle Size Analysis. The amounts passing the No. 200 sieve are shown on the Logs of Borings.

B.3.2 Moisture and Density Tests

Moisture content and density tests were performed on selected samples to evaluate their consistencies and the moisture variation throughout the explored profile. The moisture content was evaluated in accordance with ASTM D-2216 -92, Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock, and was considered to represent the moisture content of the entire sample for dry density evaluation. The test results are presented on the Logs of Borings at the appropriate sample depth, in Appendix A.

B.3.3 Atterberg Limits

Atterberg limits were evaluated on selected cohesive, fine-grained soil samples to assist in their classification. Liquid limits, plastic limits, and plasticity indices were evaluated in accordance with ASTM D-4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The results of the Atterberg limits tests are included in Appendix B, Plate B-2.1 through B-2.2, Plasticity Chart. Liquid limits and plasticity indices are also shown on the Logs of Borings, in Appendix A.

B.3.4 Consolidation Tests

Consolidation tests were performed on selected undisturbed soil samples by Cooper Testing Labs of Palo Alto, California, to evaluate their consolidation properties. The tests were conducted in accordance with ASTM D2435 Standard Test Method for One-Dimensional Consolidation Properties of Soil. The Consolidation test results are shown in Appendix B, Plates B-3.1 through B-3.2.

B.3.5 Unconfined Compressive Strength Tests

Unconfined compressive strength tests were performed on selected cohesive soil samples at the AGS Laboratory in San Francisco, California, to evaluate their strength characteristics. The tests were conducted in accordance with ASTM D-2166, Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. The unconfined compressive strength test results are shown in Appendix B, Plates B-4.1 through B-4.5.

B.3.6 Unconsolidated Undrained Triaxial Tests

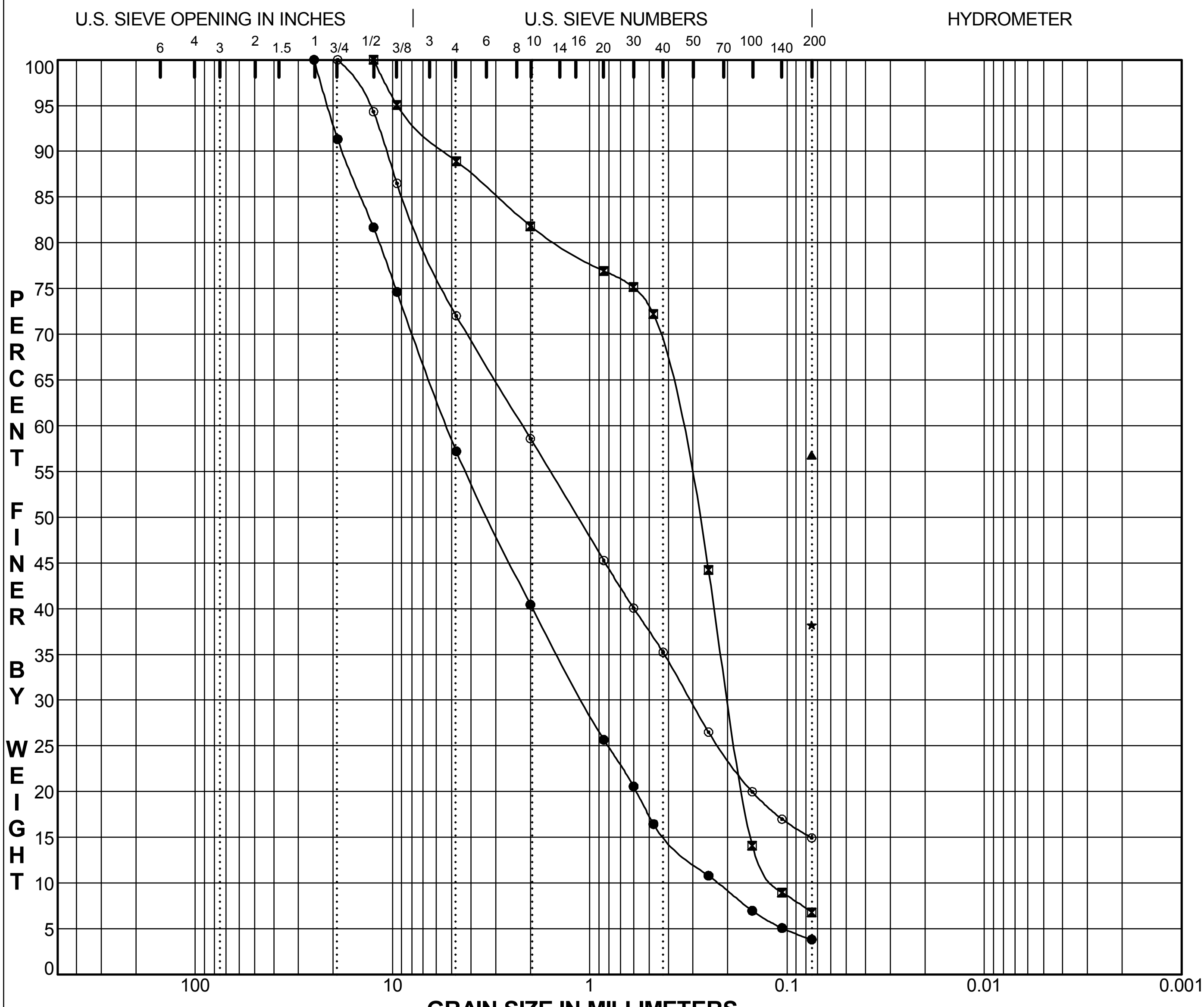
Unconsolidated, undrained triaxial tests were performed on selected cohesive soil samples by Cooper Testing Labs of Palo Alto, California to evaluate their strength characteristics. The tests were conducted in accordance with ASTM D-2850, Standard Test Method for Unconsolidated, Undrained Triaxial Strength of Cohesive Soil. The TXUU test results are shown in Appendix B, Plate B-5.1.

B.3.7 R-value

A Bulk sample of existing soils was collected from upper five feet of Boring B-8 and transported to Cooper Testing Laboratory, Palo Alto, California, for R-value testing. The results of the R-value testing has been shown in this Appendix.

B.3.8 Environmental testing

Environmental testing was completed on a composite soil sample for purposes of waste characterization, off-haul, and disposal. The soil sample was delivered with the appropriate chain of custody forms to Curtis & Tompkins, Ltd., a state licensed analytical testing laboratory located in Berkeley, California. The soil sample was analyzed for Total Purgeable Petroleum Hydrocarbons quantities as Gasoline (TPH-g) by EPA Method 8015B; Methyl-Tertiary-Butyl-Ether (MTBE), Benzene, Toluene, Ethylbenzene and Xylenes (BTEX) by EPA Method 8021B; Total Extractable Petroleum Hydrocarbons (TEH) quantified as Diesel and Motor Oil by EPA Method 8015B; and CA Title 26 Metals by EPA Methods 6010B and 7471. Based on these results, the waste soils were classified non-hazardous for landfill acceptance and disposal.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	SAMPLE SOURCE		CLASSIFICATION	MC%	LL	PL	PI	Cc	Cu
●	B-03 @	13.0'	Well-Graded Sand with Gravel and Concrete (SW)	20				1.00	23.6
⊠	B-03 @	47.0'	Poorly Graded Sand with Clay and Gravel (SP-SC)	40				0.94	3.2
▲	B-03 @	55.5'	Sandy Lean Clay (CL)	18	24	16	8		
★	B-03 @	85.5'	Clayey Sand (SC)	18	37	20	17		
⊙	B-04 @	6.5'	Clayey Sand with Gravel (SC)						

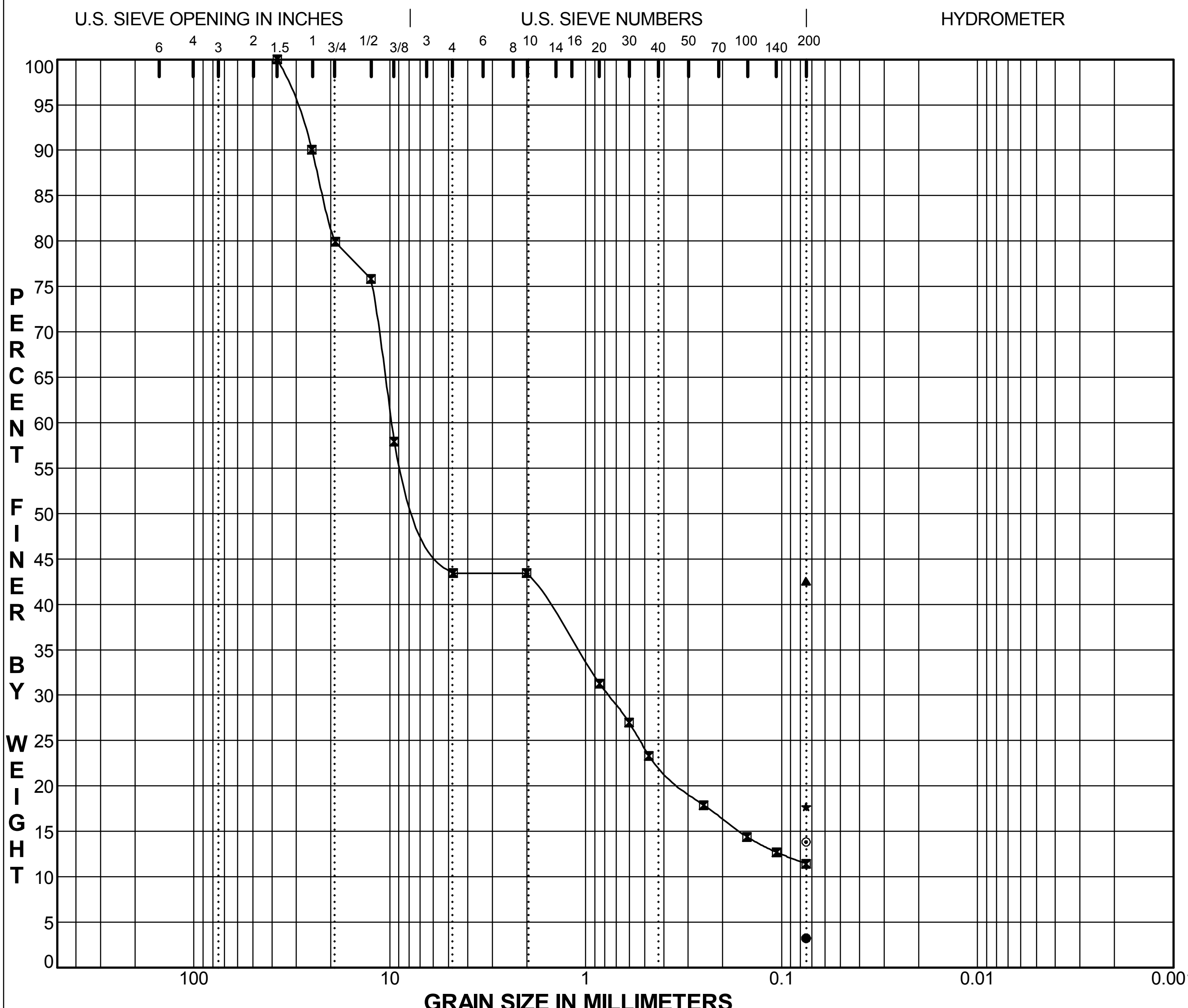
	SAMPLE SOURCE		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-03 @	13.0'	25.00	5.31	1.092	0.2247	42.8	53.4	3.8	
⊠	B-03 @	47.0'	12.50	0.36	0.196	0.1137	11.1	82.1	6.8	
▲	B-03 @	55.5'	0.08				0.0	0.0	56.8	
★	B-03 @	85.5'	0.08				0.0	0.0	38.2	
⊙	B-04 @	6.5'	19.00	2.19	0.309		28.0	57.1	14.9	

PARTICLE SIZE ANALYSIS

Pier 70 - Crane Cove Park
San Francisco, California

AGS, Inc.
Consulting Engineers

JOB NO. KK-0210 | DATE Mar 2014 | PLATE B-1.1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

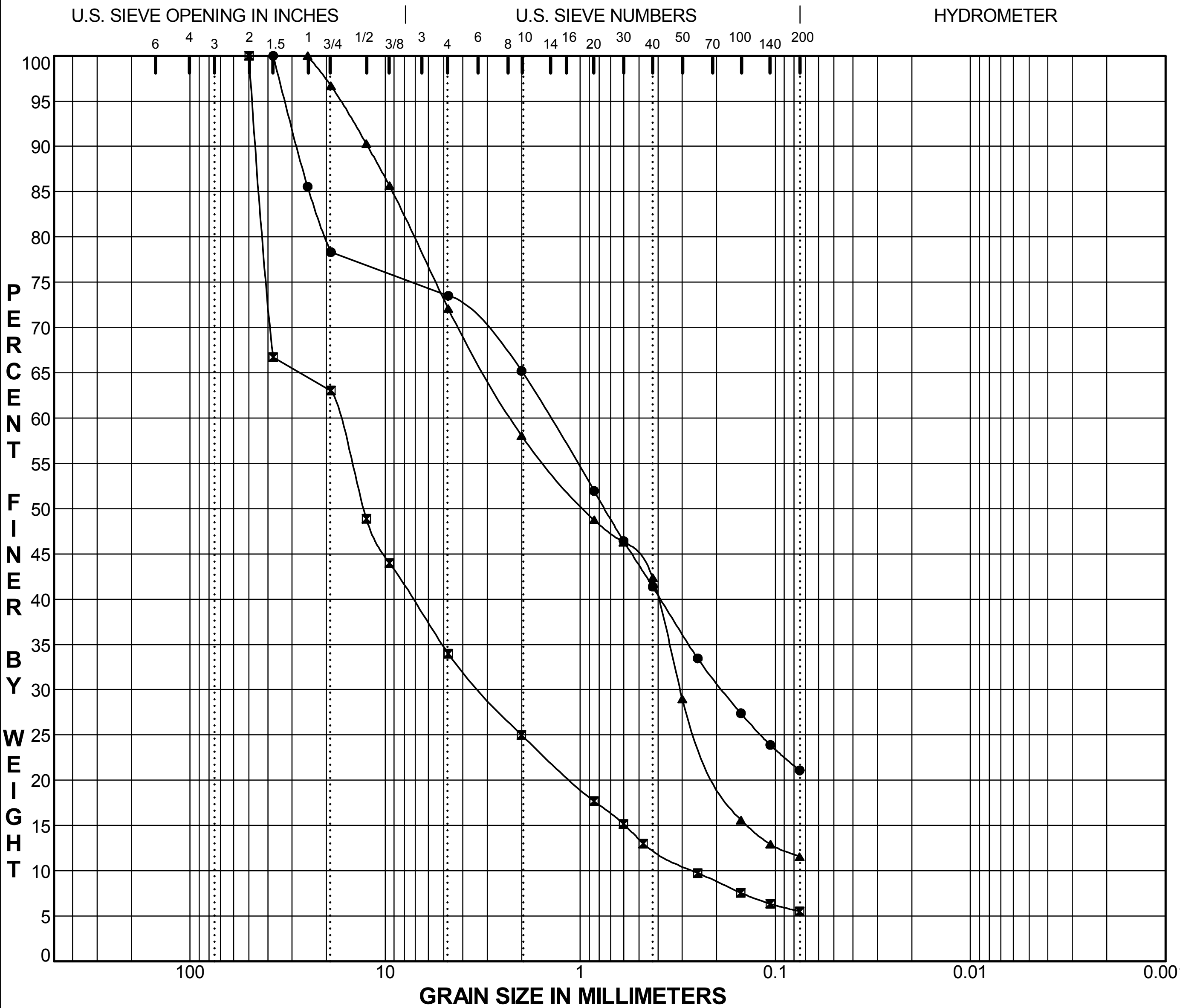
	SAMPLE SOURCE		CLASSIFICATION	MC%	LL	PL	PI	Cc	Cu
●	B-04 @	75.5'	Poorly Graded Sand (SP)	32					
⊠	B-05 @	2.5'	Poorly Graded Sand with Silt (SP-SM)	28				1.18	192.8
▲	B-05 @	64.0'	Silty, Clayey Sand with Shells and Gravel (SC-SM)	35					
★	B-06 @	6.5'	Silty Sand (SM)						
⊙	B-07 @	7.0'	Silty, Clayey Sand with Gravel (SC-SM)	42					

	SAMPLE SOURCE		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-04 @	75.5'	0.08				0.0	0.0	3.2	
⊠	B-05 @	2.5'	37.50	9.81	0.767		56.6	32.0	11.4	
▲	B-05 @	64.0'	0.08				0.0	0.0	42.5	
★	B-06 @	6.5'	0.08				0.0	0.0	17.7	
⊙	B-07 @	7.0'	0.08				0.0	0.0	13.8	

PARTICLE SIZE ANALYSIS
Pier 70 - Crane Cove Park
San Francisco, California

AGS, Inc.
Consulting Engineers

JOB NO. KK-0210 | DATE Mar 2014 | PLATE B-1.2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

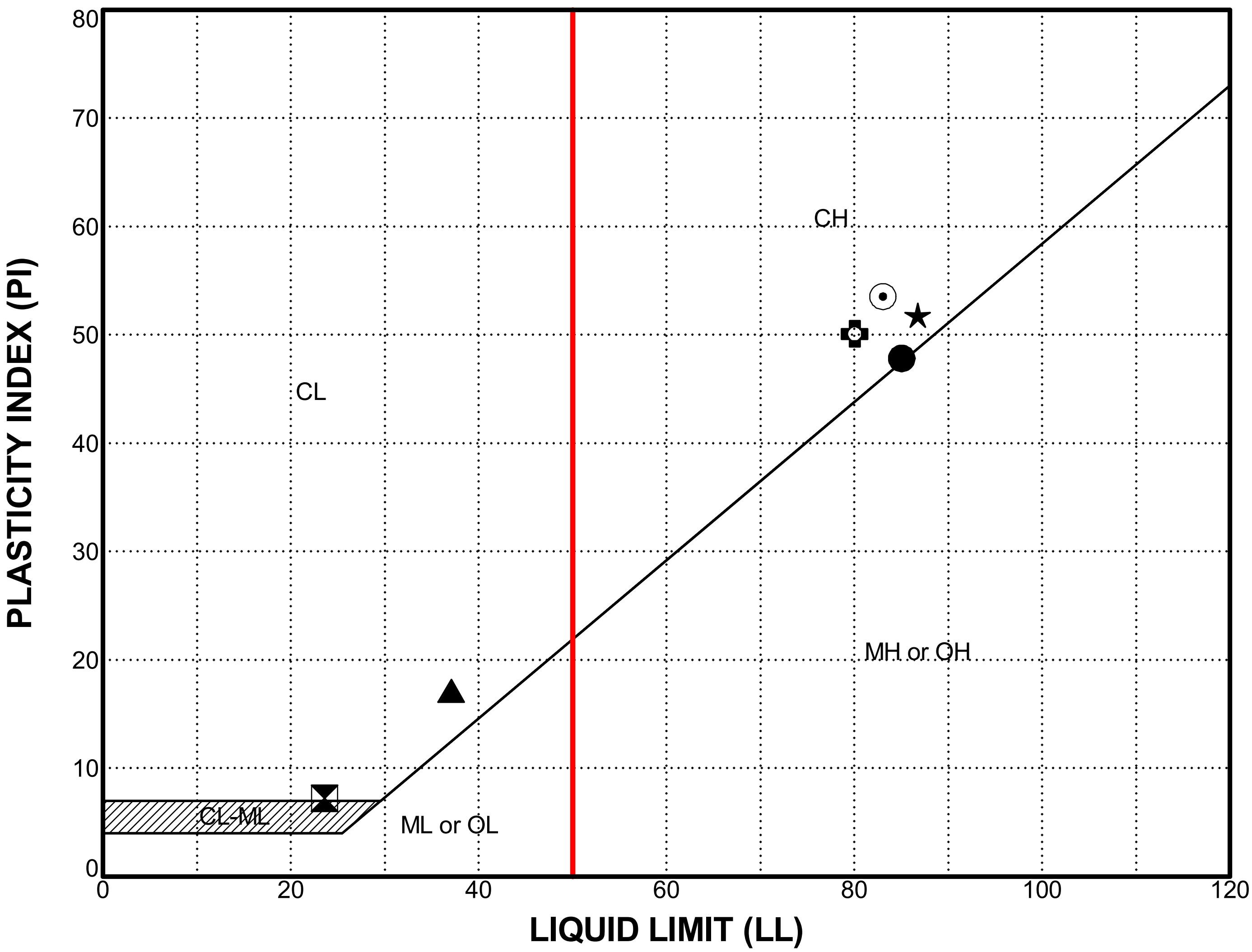
	SAMPLE SOURCE		CLASSIFICATION	MC%	LL	PL	PI	Cc	Cu
●	B-09 @	14.5'	Silty, Clayey Sand with Gravel (SC-SM)	27					
⊠	B-09 @	19.5'	Well-Graded Gravel and Sand with Clay (GW-GC)	16				2.30	66.2
▲	B-09 @	33.5'	Sand with Gravel and Clay (SW-SP)	17				0.84	45.2

	SAMPLE SOURCE		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-09 @	14.5'	37.50	1.43	0.187		26.5	52.4	21.1	
⊠	B-09 @	19.5'	50.00	17.37	3.239	0.2625	66.0	28.5	5.5	
▲	B-09 @	33.5'	25.00	2.26	0.308		27.9	60.5	11.6	

PARTICLE SIZE ANALYSIS
Pier 70 - Crane Cove Park
San Francisco, California

AGS, Inc.
Consulting Engineers

JOB NO.	KK-0210	DATE	Mar 2014	PLATE B-1.3

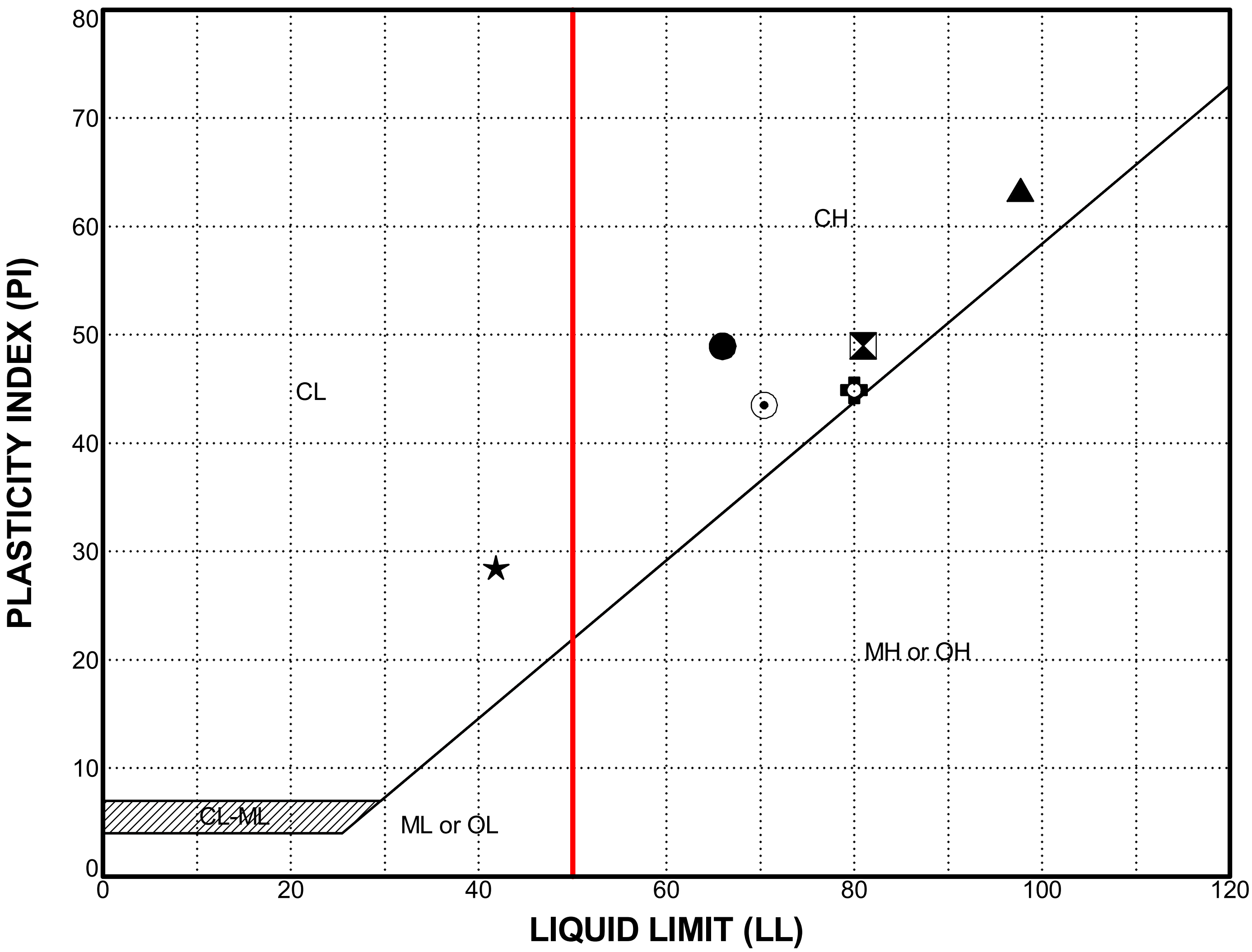


SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
● B-03 @ 22.0'	Fat Silty Clay (CH)	85	37	48	
⧗ B-03 @ 55.5'	Sandy Lean Clay (CL)	24	16	8	57
▲ B-03 @ 85.5'	Clayey Sand (SC)	37	20	17	38
★ B-04 @ 18.0'	Fat Silty Clay (CH)	87	35	52	
⊙ B-05 @ 31.0'	Fat Silty Clay (CH)	83	30	53	
✿ B-06 @ 16.0'	Fat Silty Clay (CH)	80	30	50	

PLASTICITY CHART
Pier 70 - Crane Cove Park
San Francisco, California

AGS, Inc.
Consulting Engineers

JOB NO.	**KK-0210**	**DATE**	**Mar 2014**	**PLATE B-2.1**



SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
● B-06 @ 42.0'	Fat Silty Clay (CH)	66	17	49	
⧗ B-06 @ 70.5'	Fat Silty Clay (CH)	81	32	49	
▲ B-07 @ 17.0'	Fat Silty Clay with Sand (CH)	98	34	64	
★ B-07 @ 37.5'	Lean Clay with Sand and Gravel (CL)	42	13	29	
⊙ B-10 @ 20.0'	Fat Silty Clay (CH)	70	27	43	
✿ B-10 @ 65.0'	Sandy Fat Clay (CH)	80	35	45	

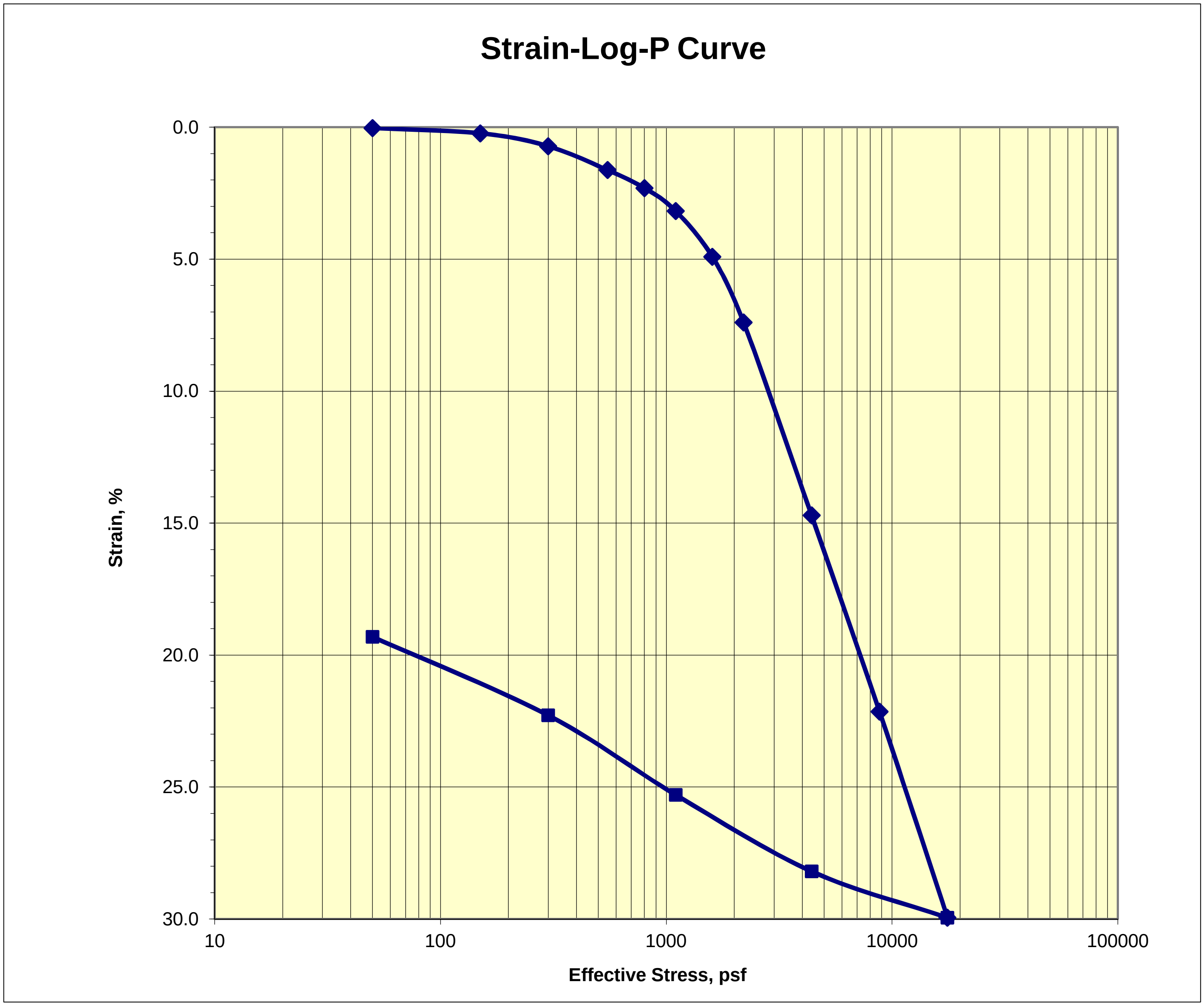
PLASTICITY CHART
Pier 70 - Crane Cove Park
San Francisco, California

AGS, Inc.
Consulting Engineers

JOB NO.	KK-0210	DATE	Mar 2014	PLATE B-2.2

Consolidation Test
ASTM D2435

Job No.:	041-106	**Boring:**	B-3-4	**Run By:**	MD
Client:	AGS	**Sample:**		**Reduced:**	PJ
Project:	KK0210-Phase 2	**Depth, ft.:**	20-22(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Dark Greenish Gray CLAY			**Date:**	3/19/2014

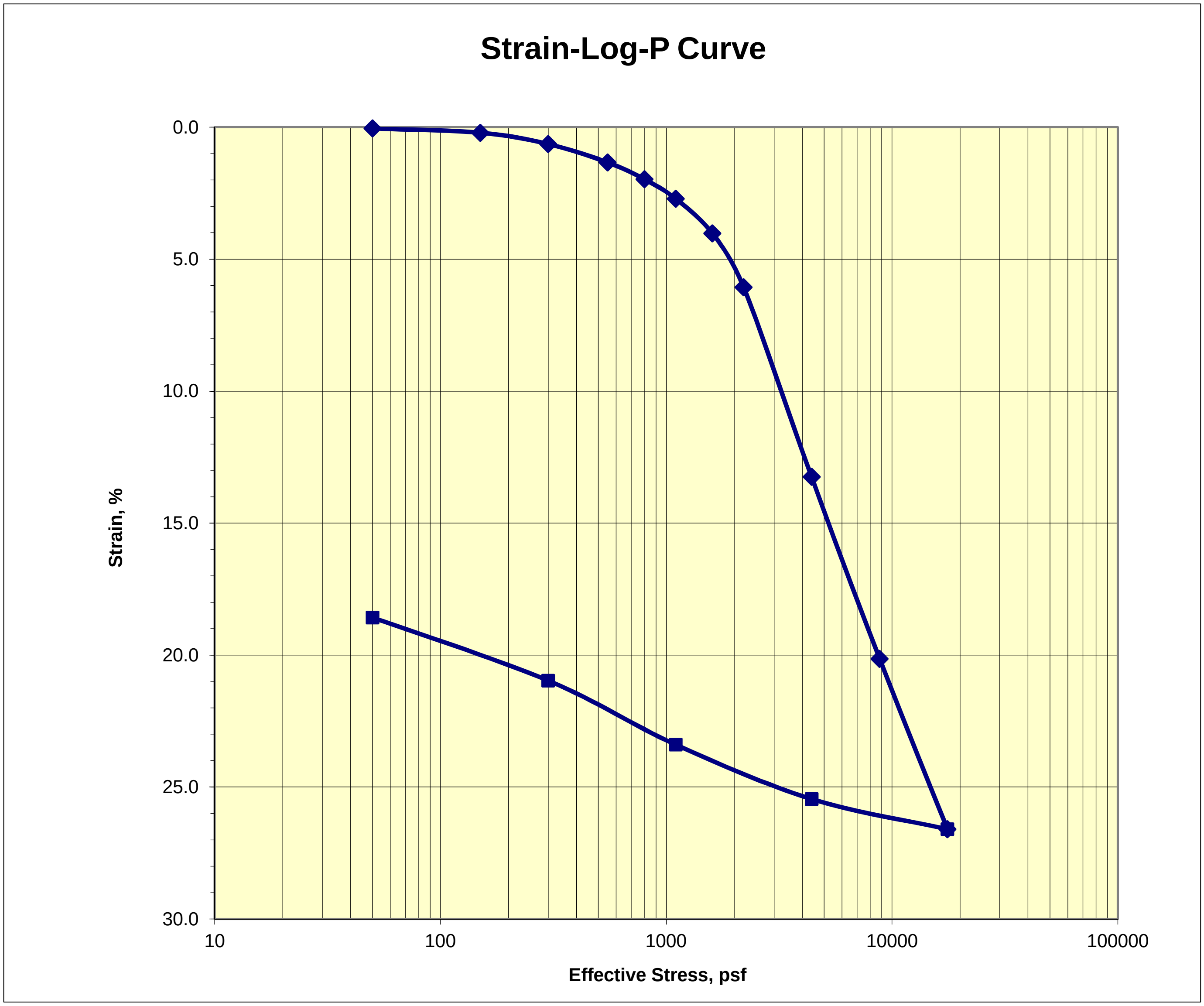


Assumed Gs 2.8	Initial	Final
Moisture %:	77.3	56.0
Dry Density, pcf:	55.0	68.0
Void Ratio:	2.181	1.569
% Saturation:	99.3	100.0

PLATE B-3.1

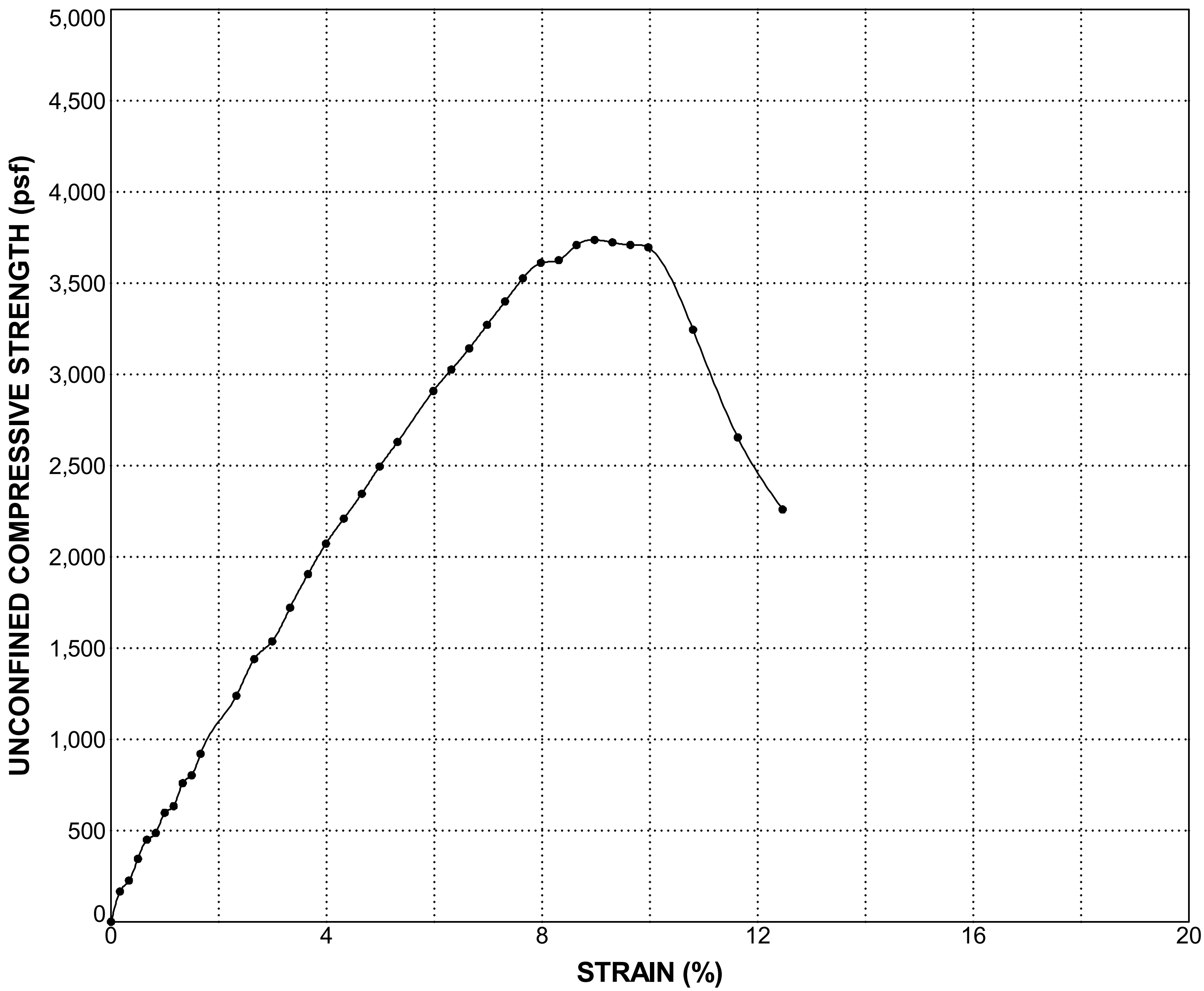
Consolidation Test
ASTM D2435

Job No.:	041-106	**Boring:**	B-10-2	**Run By:**	MD
Client:	AGS	**Sample:**		**Reduced:**	PJ
Project:	KK0210-Phase 2	**Depth, ft.:**	18-20	**Checked:**	PJ/DC
Soil Type:	Dark Greenish Gray CLAY w/ shells			**Date:**	3/19/2014



Assumed Gs 2.7	Initial	Final
Moisture %:	60.7	42.4
Dry Density, pcf:	63.6	78.6
Void Ratio:	1.651	1.145
% Saturation:	99.2	100.0

PLATE B-3.2



Sample Source	Classification	Type of Test		Ultimate Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
● B-03 @ 55.5'	Sandy Lean Clay (CL)			3737	9	110	18.1

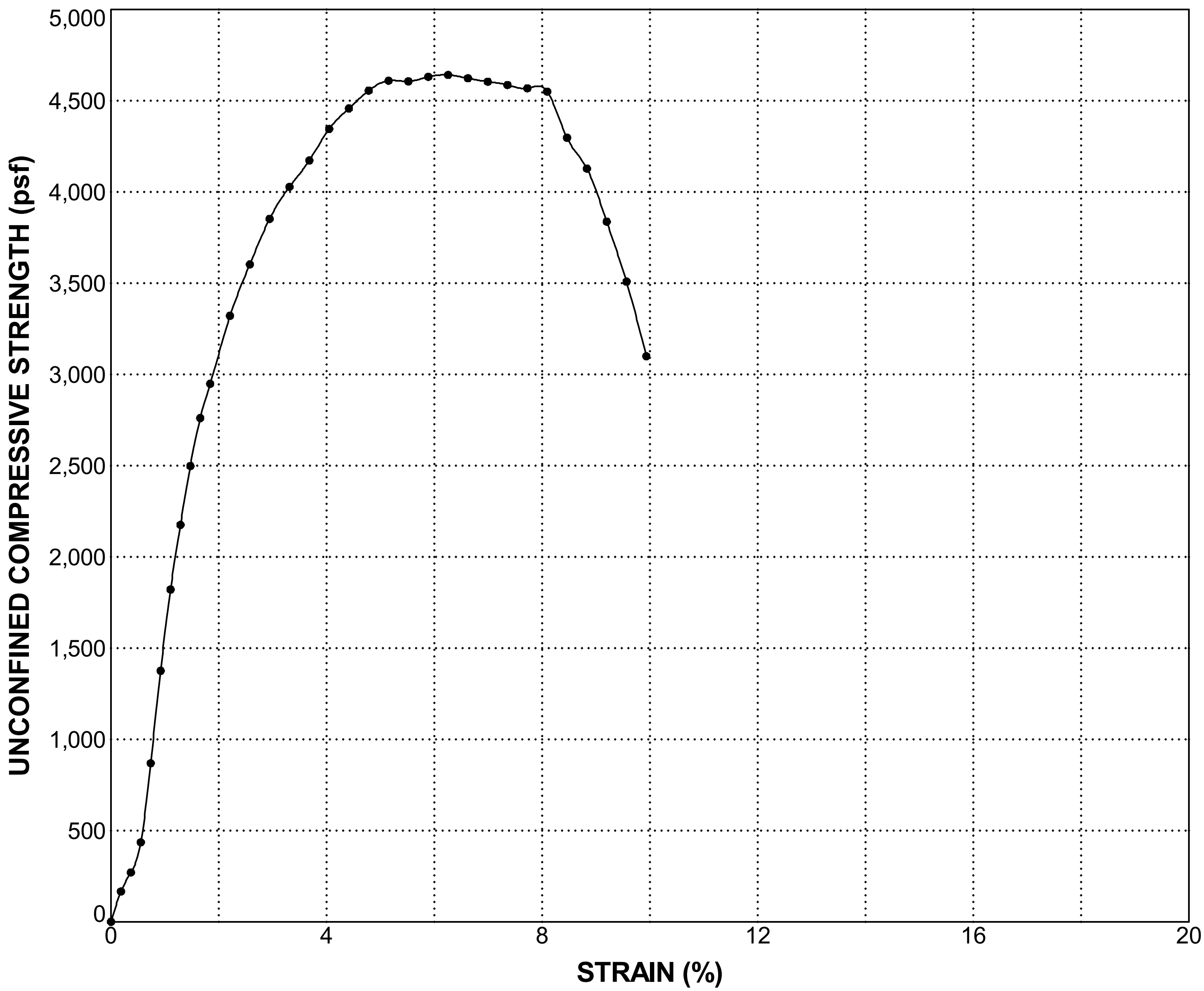
UC = Unconfined Compression

UNCONFINED COMPRESSIVE STRENGTH

Pier 70 - Crane Cove Park
San Francisco, California

AGS, Inc.
Consulting Engineers

JOB NO.	KK-0210	DATE	Mar 2014	PLATE B-4.1



Sample Source	Classification	Type of Test		Ultimate Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
● B-03 @ 85.5'	Clayey Sand (SC)			4641	6	108	17.6

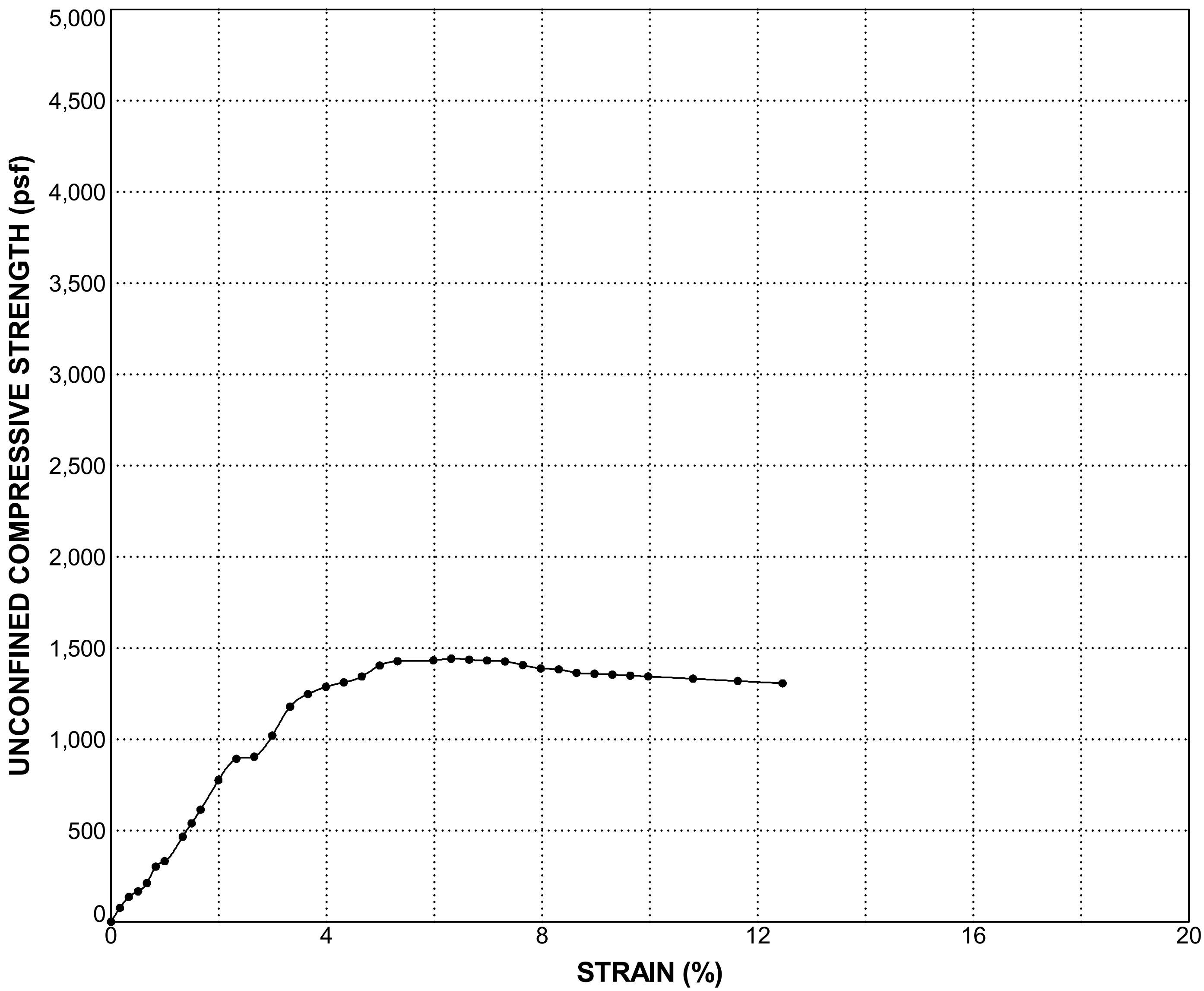
UC = Unconfined Compression

UNCONFINED COMPRESSIVE STRENGTH

Pier 70 - Crane Cove Park
San Francisco, California

AGS, Inc.
Consulting Engineers

JOB NO.	KK-0210	DATE	Mar 2014	PLATE B-4.2



Sample Source	Classification	Type of Test		Ultimate Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
● B-05 @ 64.0'	Silty, Clayey Sand with Shells and Gravel (SC-SM)			1442	6	110	18.1

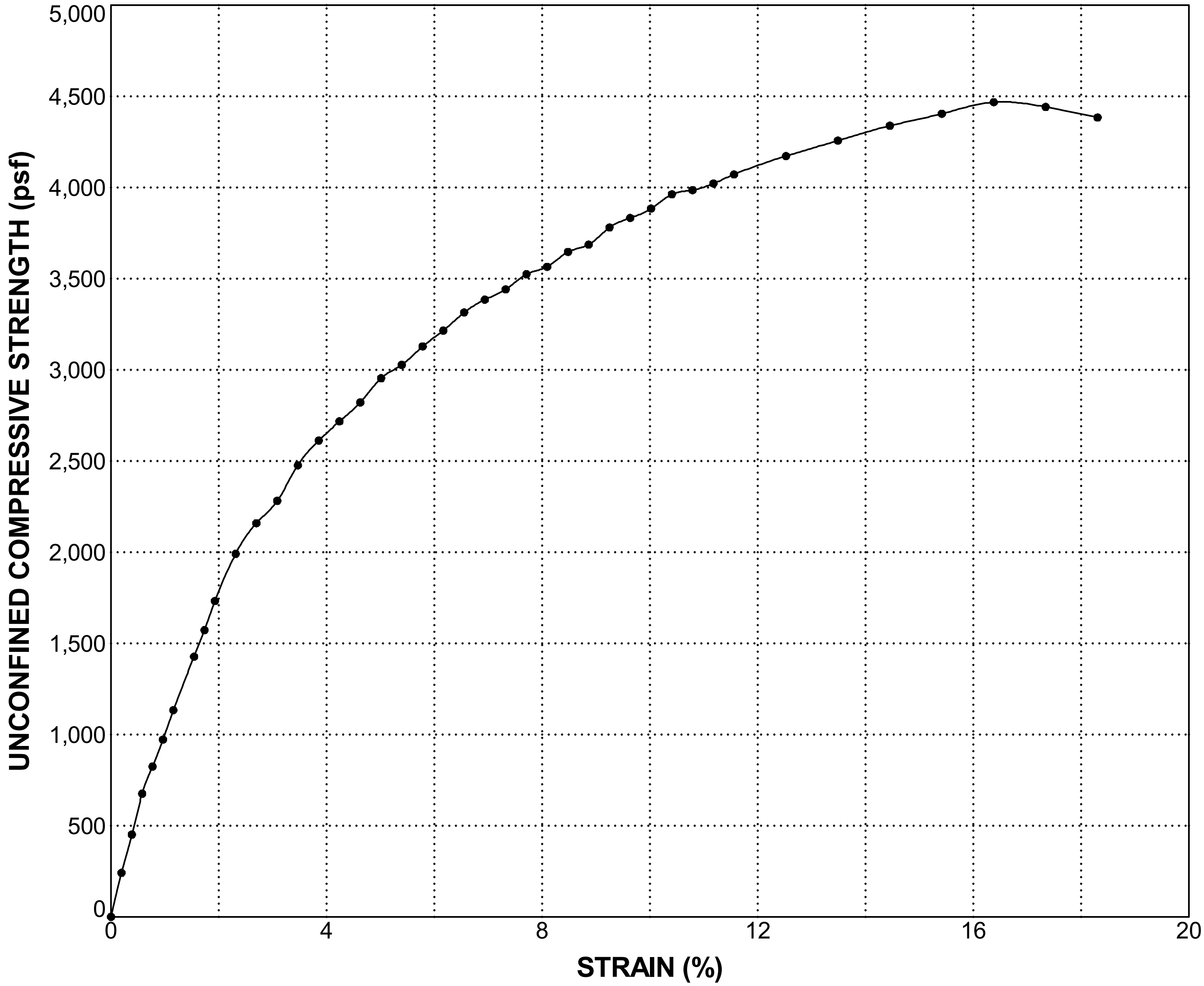
UC = Unconfined Compression

UNCONFINED COMPRESSIVE STRENGTH

Pier 70 - Crane Cove Park
San Francisco, California

AGS, Inc.
Consulting Engineers

JOB NO. KK-0210	DATE Mar 2014		PLATE B-4.3



Sample Source	Classification	Type of Test		Ultimate Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
● B-07 @ 37.5'	Lean Clay with Sand and Gravel (CL)			4376	15	119	19.1

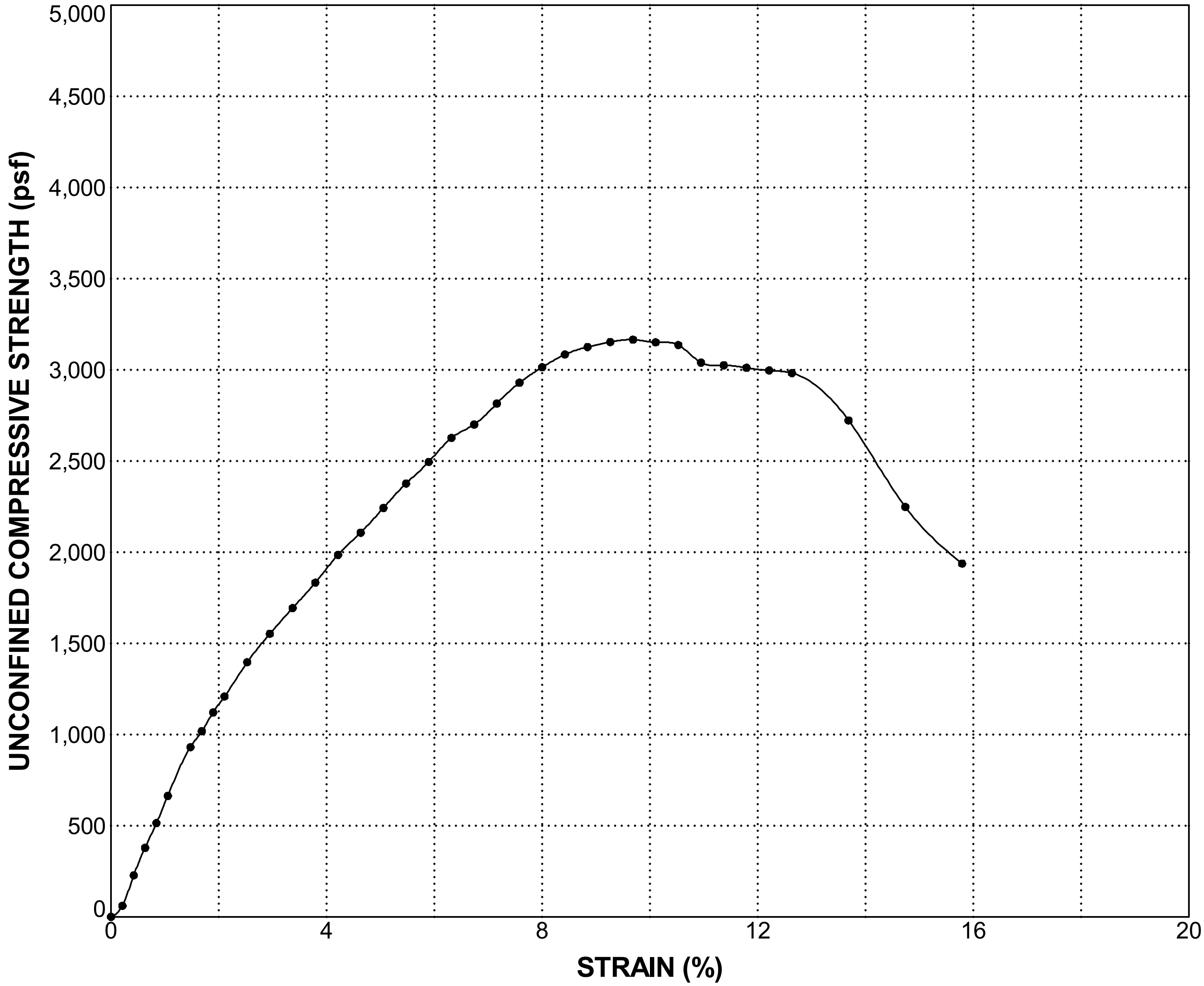
UC = Unconfined Compression

UNCONFINED COMPRESSIVE STRENGTH

Pier 70 - Crane Cove Park
San Francisco, California

AGS, Inc.
Consulting Engineers

JOB NO.	KK-0210	DATE	Mar 2014	PLATE B-4.4



Sample Source	Classification	Type of Test		Ultimate Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
● B-10 @ 45.5'	Sandy Lean Clay (CL)			3165	10	102	22.2

UC = Unconfined Compression

UNCONFINED COMPRESSIVE STRENGTH

Pier 70 - Crane Cove Park
San Francisco, California

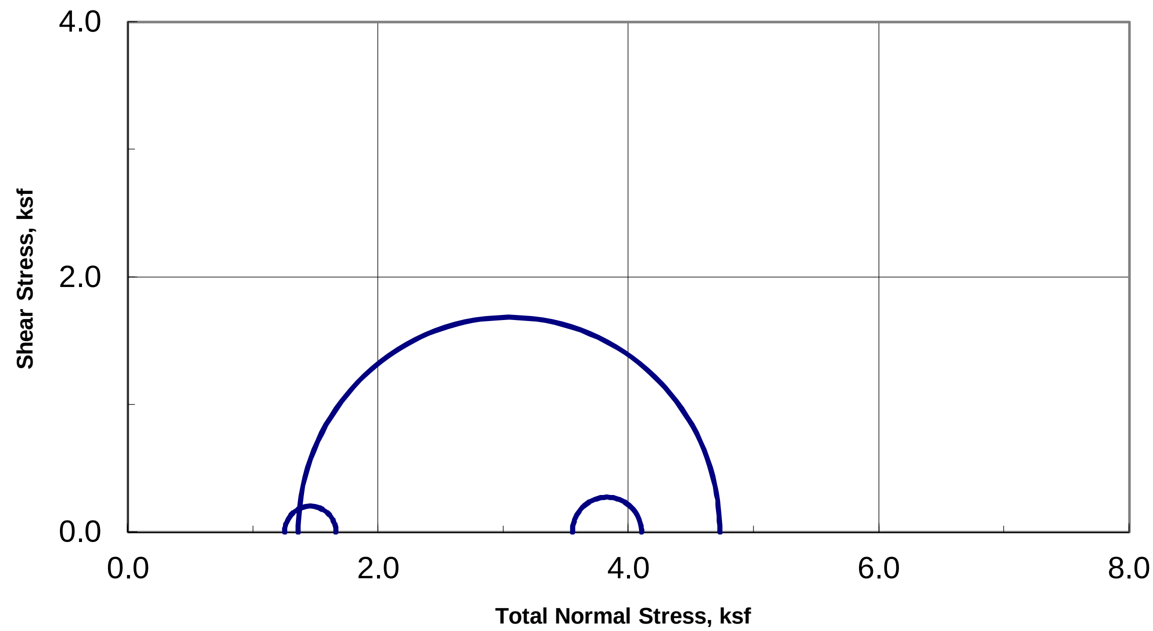
AGS, Inc.
Consulting Engineers

JOB NO.	KK-0210	DATE	Mar 2014	PLATE B-4.5

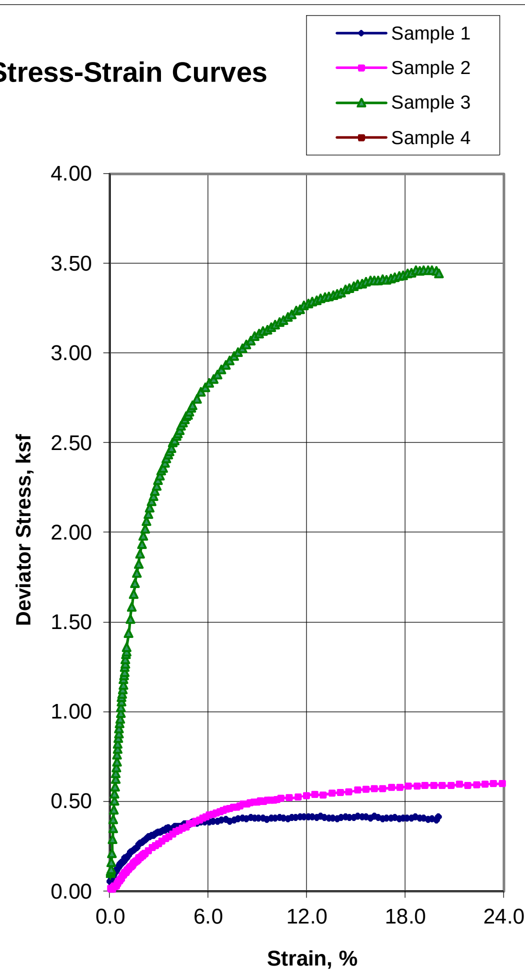
Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test

ASTM D2850



Stress-Strain Curves



Sample Data				
	1	2	3	4
Moisture %	81.0	24.0	24.6	
Dry Den,pcf	52.4	49.9	100.5	
Void Ratio	2.215	2.380	0.677	
Saturation %	98.8	27.2	98.1	
Height in	6.00	1.93	5.99	
Diameter in	2.86	3.99	2.89	
Cell psi	8.7	24.7	9.4	
Strain %	15.00	15.00	15.00	
Deviator, ksf	0.412	0.553	3.373	
Rate %/min	1.00	2.07	1.00	
in/min	0.060	0.040	0.060	
Job No.:	041-106			
Client:	AGS			
Project:	KK0210-Phase 2			
Boring:	B-5-4	B-5-9	B-8-5	
Sample:				
Depth ft:	15-17(Tip-3")	84-84.5	19-21(Tip-6")	

Visual Soil Description	
Sample #	
1	Very Dark Bluish Gray CLAY, trace Sand
2	Olive Clayey SAND
3	Greenish Gray Sandy CLAY
4	

PLATE 5-1.1

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

<u>APPENDIX C</u>

CORROSIVITY TESTING

C.1 CORROSIVITY TESTING

Representative soil samples from the borings were selected and transferred to Cooper Testing Laboratory for analysis and evaluation of the corrosivity potential to buried construction materials such as metal and concrete.

Two (2) samples were selected for testing including resistivity, chlorides, sulfate, pH, Redox, and Sulfide. Tests were performed in accordance with California Test Methods 417, 422, and 532. Test results are presented in this report.

Corrosivity Tests Summary

CTL # 041-106 **Date:** 3/5/2014 **Tested By:** PJ **Checked:** PJ
Client: AGS, Inc. **Project:** Pier 70 - Crane Cove Park- Phase 2 **Proj. No:** KK0210- Phase 2
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
B-4-1	-	2-3.5	-	-	4,208	5	16	0.0016	7.7	499	22	-	8.2	Yellowish Brown GRAVEL w/ Silt & Sand
B-5-2	-	2.5-4	-	-	68	5,431	527	0.0527	6.7	-117	22	-	27.9	Black GRAVEL w/ Silt & Sand

APPENDIX D

LIQUEFACTION ANALYSES

D.1 GENERAL

Soil liquefaction is a phenomenon in which saturated (submerged) cohesionless soils experience a temporary loss of strength due to the build-up of excess pore water pressure during cyclic seismic loadings. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements. Soils most susceptible to liquefaction are loose, clean, saturated, and uniformly graded, fine-grained sands, which lie within about 50 feet of the ground surface. Saturated loose silty and clayey sands may also liquefy during strong ground shaking.

This appendix presents the results of our liquefaction potential evaluation for the proposed improvement discussed in the main text. The liquefaction potential evaluation was based on the results of our field exploration program, whereby blow counts were recorded by driving the California Modified (MC) and Standard Penetration Test (SPT) samplers. The blow counts shown on these plates were corrected for various factors, as discussed below, and used in the liquefaction analyses.

The design earthquakes evaluated were a Maximum Moment Magnitude event of M7.9 on the San Andreas Fault. The San Andreas Fault located about 12 kilometers southwest of the site. Our evaluations were made using the liquefaction evaluation procedure developed by National Center for Earthquake Engineering (NCEE), 1996; Youd and Idriss 2001, and Idriss and Boulanger (2008) based on liquefaction observation in previous earthquakes.

A comprehensive collection of site conditions at various locations where some evidence of liquefaction was known to have or to have not taken place was collected by Seed and others (1984). These data on sandy soils with a fines content less than 5 percent under magnitude 7.5 earthquake conditions was presented as relationships between field values of average cyclic stress ratio, τ_{av}/σ'_o (where: τ_{av} = average horizontal shear stress induced by an earthquake; and σ'_o = initial effective overburden pressure on the soil element), and the SPT blow counts corrected for certain effects. For an earthquake of magnitude 7.9, the cyclic shear stress ratio necessary to cause liquefaction in Seed's curve was corrected to account for the earthquake magnitude or duration effect (Idriss, 1996).

For the first step in estimating liquefaction potential, the measured SPT blow counts should be corrected for various factors using the method proposed by NCEER (1996). The raw SPT blow count, N, is corrected to obtain the modified penetration resistance, $(N_1)_{60}$. The modified penetration resistance is computed as follows:

$$(N_1)_{60} = N \times C_m \times C_z \times C_h \times C_s \times C_n \qquad \text{(D.1)}$$

where:

N: raw SPT or Modified California blow count (blows/ft);

C_m: a factor to correct for the larger size of the Modified California sampler. Raw blow counts using a Modified California sampler were multiplied by 0.61;

C_z: a factor that depends on the length of the drive rods; the following C_z factors may be used for various depths:

Depth	C_z
20 < x <30 ft	1.0
13 < x <20 ft	0.95
10 < x <13 ft	0.85
<10 ft	0.75

C_h: a factor that accounts for the hammer efficiency used in the field, where the blow count is multiplied by a factor of 0.9;

C_s: a factor that depends on the sampling tube; for a split-spoon sampler without liner (ID = 1.5" and OD = 2.0"), the following C_s factors may be used:

Raw Blow Count, N	C_S
< 10	1.0
> 10	1.2

C_n: a factor that depends on the effective overburden pressure at the depth when the penetration test was conducted.

As presented by NCEE (1996), another correction factor, $\delta(N_1)_{60}$, should be added to $(N_1)_{60}$ to account for fine contents as follows:

$$(N_1)_{60\,cs} = (N_1)_{60} + \delta(N_1)_{60} \qquad \text{(D.2)}$$

$$\delta(N_1)_{60} = EXP(1.63+9.7/(FC(\%)+0.01)-(15.7/(FC(\%)+0.01))\text{^}2)$$

Where FC is the percent fine content.

The average cyclic stress ratio (CSR), τ_{av}/σ'_o, at a specific depth can be estimated from dynamic site response analyses. It also can be estimated with reasonable accuracy from the following equation as discussed by Seed and Idriss (1982).

$$\tau_{av}/\sigma'_o=0.65 \times a_{max}/g \times \sigma_o/\sigma'_o \times r_d \qquad \text{(D.3)}$$

where:

a_{max}: maximum acceleration at the ground surface;

σ_o: total overburden pressure;

σ_o': effective overburden pressure; and

r_d: a stress reduction factor.

Based on the magnitude of the design earthquake, and the peak ground acceleration generated by that earthquake, the cyclic stress ratio was calculated using Equation D.3. The cyclic stress ratio was then corrected to account for an earthquake magnitude other than 7.5. The resulting curve of the threshold earthquake, together with a plot of cumulative liquefaction and seismically-induced settlement versus depth for Borings SAB-1, SAB-2, and SAB-3 was generated, as shown on Plates D-1 through D-3.

Equation D.4 is used to estimate corrected cyclic resistance ration (CRR).

$$CRR_R = CRR_M * MSF * K_s \qquad \text{(D.4)}$$

Where:

Magnitude scaling factor (MSF) is used to adjust the CRR to a common value of M_w=7.5, because the CRR depends on the number of loading cycles, which correlates with M_w (Seed et al. 1975b). MSF is calculated using equation D.5.

$$MSF = 6.9*EXP(-MCE/4)-0.058) \geq 1.8 \qquad \text{(D.5)}$$

Overburden correction factor (K_s) was introduced by Seed (1983) to adjust the CRR. Overburden correction factor can be estimated using equation D.6.

$$K_s= 1-(1/(18.9-2.55*SQRT((N_1)_{60\ cs} \geq 37))*LN((\sigma_0'/21.09)/101)) \geq 1.1 \qquad \text{(D.6)}$$

The derived correlation between CRR and penetration resistances is expressed via following equations developed by Idriss and Boulanger (2008).

For $(N_1)_{60\,cs} < 37.5$, (D.7)

$CRR_M = EXP((N_1)_{60\,cs}/14.1+((N_1)_{60\,cs}/126)^2-((N_1)_{60\,cs}/23.6)^3+((N_1)_{60\,cs}/25.4)^4-2.8)$

And For $((N_1)_{60\,cs} \geq 37.5$, $CRR_M = 2$ (D.8)

Finally CRR can be estimated using equation D.9.

$CRR = MIN(2, CRR_R)$ (D.9)

The factor of safety against liquefaction can be estimated using equation D.10.

$FS_{liq} = CRR/CSR$ (D.10)

D.2 SEISMICALLY-INDUCED SETTLEMENT

For coarse-grained soils such as sand and gravel with various amount of silt and clay, AGS used a liquefaction evaluation approach developed over the years by Seed and his co-authors.

For fine-grained soils such as silt and clay, there are currently two screening procedures. Both approaches are based on modified Chinese Criteria for liquefaction evaluation of fine-grained soils. The first approach was developed by Bray and Sancio (2006), and another approach was developed by Boulanger and Idriss (2006). The Bray and Sancio (2006) criteria state that a soil is:

a) Susceptible to liquefaction if w_c/**LL > 0.85**, **PI < 12,** and **LL<37**

b) Moderately susceptible to liquefaction if **0.8 <**w_c**/LL< 0.9** and **12< PI <18**

c) Not susceptible to liquefaction if w_c**/LL< 0.8** and **PI >18**

where w_c is water content, **LL** is Liquid Limit, and **PI** is Plasticity Index. The criteria presented by Boulanger and Idriss (2006) state that a soil is

a) **sand-like if PI < 7**

b) **clay-like if PI > 7**

where sand-like soils are susceptible to liquefaction, and clay-like soils should be evaluated using Boulanger and Idriss (2004) criteria based on the cyclic triaxial shear testing. The method of analyses is based on the relationship proposed by Tokimatsu and Seed (1987), and Ishihara and Yoshimine (1990), Zhang (2004) with scale effect modification.

Based on the results of AGS' evaluation and the estimated thicknesses of the liquefiable soils, the estimated seismically-induced settlement of the project area would be as shown in Tables D-1 to D-8.

General Data

S	Ground Slope / Liquefiable Layer Slope	0.1
L	Horizontal Distance from the Toe of a Free Face (ft)	50
H	Free Face Height (ft)	8
Gw	Depth to groundwater	2
Mw	Maximum Moment Magnitude	7.9
PGA	Peak Ground Acceleration	0.36

Table D-1 - Seismically-Induced Settelment & Lateral Deformation Calculation - B-3

SAMPLE DEPTH (feet)	ROD LENGTH (feet)	ROD LENGTH (m)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (N) (blows per foot)	SOIL TYPE	PI Clay/Silt	Su Clay/Silt (psf)	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	TOTAL OVER-BURDEN (psf)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)
12.75	16.75	5.1054	7	MC	22	SW			4	Y	N	120	70	1530	670.8	859.2	41.1494
18	22	6.7056	7	MC	2	CH	50	4000	90	N	N	90	70	2002.5	998.4	1004.1	48.0891
44.75	48.75	14.859	7	MC	15	SP			7	N	N	120	70	5212.5	2667.6	2544.9	121.882
46.25	50.25	15.3162	7	SPT	6	SP			7	N	N	120	70	5392.5	2761.2	2631.3	126.02
50																	

SAMPLE DEPTH (feet)	CORRECTION FACTORS C_b Borehole Size Factor	C_n Depth Reduction Factor	C_m MC Factor	C_R Drill Rod Length Factor	C_h Hammer Efficiency Factor	C_s Unlined SPT Factor	PENETRATION RESISTANCE N_{60}	MODIFIED PENETRATION RESISTANCE $(N_1)_{60}$	ΔN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND $(N_1)_{60CS}$	STRESS REDUCTION COEFFICIENT (r_d)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (K_s)	CRR STATIC SHEAR CORRECTION (K_a)	CORRECTED CRR (CCRR) Sand	CORRECTED CRR (CCRR) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
12.75	1.15	1.54	0.61	0.85	1.17	1.00	15.30	23.6	0.0	23.63	0.98	0.41	0.258	0.00	1.10	1.00	0.261	0.00	0.90	-	0.631	YES, LIQ.	0.63
18	1.15	1.67	0.61	0.95	1.17	1.00	1.55	2.6	5.5	8.11	0.97	0.45	NA	2.87	1.06	1.00	NA	3.05	0.90	6.75	NA	NO, NOT LIQ.	6.75
44.75	1.15	0.91	0.61	1.00	1.17	1.00	12.28	11.1	0.1	11.28	0.88	0.42	0.112	0.00	0.98	1.00	0.127	0.00	0.90	-	0.266	YES, LIQ.	0.27
46.25	1.15	0.88	1.00	1.00	1.17	1.00	8.05	7.1	0.1	7.24	0.88	0.42	0.088	0.00	0.98	1.00	0.100	0.00	0.90	-	0.210	YES, LIQ.	0.21
50																							

Liquefaction Settlement / Lateral Displacement Index and Settlement (B-1)

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatso	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatso	Liquefaction Settlement (ft) Ishihara	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	ΔH (ft)	ΔLDIi (ft)	Vertical Reconsol. Strain ε_v	Liquefaction Settlement (ft) Ishihara
12.75	YES, LIQ.	18.00	1.20	2.15	0.22	0.39	0.10	0.31450352	0.10	9.00	0.934	0.020	0.18
18.00	NO, NOT LIQ.	26.00	3.00	0.00	0.00	0.00	0.50	0.94272477	0.00	26.75	0.000	0.000	0.00
44.75	YES, LIQ.	1.50	3.00	3.50	0.05	0.05	0.41	0.88304214	0.41	1.50	0.617	0.035	0.05
46.25	YES, LIQ.	4.50	3.00	4.50	0.14	0.20	0.50	0.94740531	0.50	3.75	1.875	0.044	0.17

	Tokimatso	Ishihara
Total Settlement (ft)	0.40	0.64
Total Settlement (in)	4.75	7.70

Settlement		LDI	Lateral Displacement			
			Sloping		Free Face	
ft	in		ft	in	ft	in
0.64	7.70	3.426	1.0	12.3	4.7	56.9

Assumptions:

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

General Data

S	Ground Slope / Liquefiable Layer Slope	0.1
L	Horizontal Distance from the Toe of a Free Face (ft)	40
H	Free Face Height (ft)	12
Gw	Depth to groundwater	2
Mw	Maximum Moment Magnitude	7.9
PGA	Peak Ground Acceleration	0.36

Table D-2 - Seismically-Induced Settelment & Lateral Deformation Calculation - B-4

SAMPLE DEPTH (feet)	ROD LENGTH (feet)	ROD LENGTH (m)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (N) (blows per foot)	SOIL TYPE	PI Clay/Silt	Su Clay/Silt (psf)	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	TOTAL OVER-BURDEN (psf)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS C_b Borehole Size Factor	C_n Depth Reduction Factor	C_m MC Factor	C_R Drill Rod Length Factor	C_h Hammer Efficiency Factor	C_s Unlined SPT Factor	PENETRATION RESISTANCE N_{60}	MODIFIED PENETRATION RESISTANCE $(N_1)_{60}$
2.75	6.75	2.0574	7	SPT	10	GP			15	Y	N	125	70	343.75	46.8	296.95	14.2217	1.15	1.70	1.00	0.75	1.17	1.00	10.06	17.1
5.75	9.75	2.9718	7	SPT	3	SC			15	N	N	120	70	703.75	234	469.75	22.4976	1.15	1.70	1.00	0.75	1.17	1.00	3.02	5.1
10.5	14.5	4.4196	7	SPT	50	GW			15	Y	N	130	70	1321.25	530.4	790.85	37.876	1.15	1.29	1.00	0.85	1.17	1.00	57.02	73.8
17	21	6.4008	7	MC	2	CH	52	4000	90	N	N	90	70	1906.25	936	970.25	46.4679	1.15	1.70	0.61	0.95	1.17	1.00	1.55	2.6
50	54	16.4592																			1.00				

SAMPLE DEPTH (feet)	ΔN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND $(N_1)_{60CS}$	STRESS REDUCTION COEFFICIENT (r_d)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (K_s)	CRR STATIC SHEAR CORRECTION (K_a)	CORRECTED CRR (CCRR) Sand	CORRECTED CRR (CCRR) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
2.75	3.3	20.37	1.00	0.27	0.208	0.00	1.10	1.00	0.210	0.00	0.90	-	0.767	YES, LIQ.	0.77
5.75	3.3	8.39	1.00	0.35	0.106	0.00	1.10	1.00	0.107	0.00	0.90	-	0.304	YES, LIQ.	0.30
10.5	3.3	77.07	0.99	0.39	1.979	0.00	1.10	1.00	2.000	0.00	0.90	-	2.000	NO, NOT LIQ.	2.00
17	5.5	8.16	0.97	0.45	NA	2.97	1.07	1.00	NA	3.17	0.90	7.10	NA	NO, NOT LIQ.	7.10
50															

Liquefaction Settlement / Lateral Displacement Index and Settlement (B-1)

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatso	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatso	Liquefaction Settlement (ft) Ishihara	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	ΔH (ft)	ΔLDIi (ft)	Vertical Reconsol. Strain ε_v	Liquefaction Settlement (ft) Ishihara
2.75	YES, LIQ.	4.00	1.40	2.04	0.06	0.08	0.15	0.49820748	0.08	2.88	0.231	0.023	0.07
5.75	YES, LIQ.	6.00	3.00	4.50	0.18	0.27	0.50	0.93988259	0.50	4.75	2.375	0.041	0.20
10.50	NO, NOT LIQ.	7.00	0.05	0.00	0.00	0.00	0.00	-3.9296827	0.00	6.50	0.000	0.000	0.00
17.00	NO, NOT LIQ.	43.00	3.00	0.00	0.00	0.00	0.50	0.94225481	0.00	33.00	0.000	0.000	0.00
50.00													
			Total Settlement (ft)		0.24	0.35							
			Total Settlement (in)		2.83	4.22							

Settlement		LDI	Lateral Displacement			
			Sloping		Free Face	
ft	in		ft	in	ft	in
0.35	4.22	2.606	0.8	9.4	6.0	71.6

Assumptions:

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

General Data

S	Ground Slope / Liquefiable Layer Slope	0.1
L	Horizontal Distance from the Toe of a Free Face (ft)	30
H	Free Face Height (ft)	10
Gw	Depth to groundwater	2
Mw	Maximum Moment Magnitude	7.9
PGA	Peak Ground Acceleration	0.36

Table D-3 - Seismically-Induced Settelment & Lateral Deformation Calculation - B-5

SAMPLE DEPTH (feet)	ROD LENGTH (feet)	ROD LENGTH (m)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (N) (blows per foot)	SOIL TYPE	PI Clay/Silt	Su Clay/Silt (psf)	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	TOTAL OVER-BURDEN (psf)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)
1.75	5.75	1.7526	7	MC	12	GC			11	Y	N	120	70	210	0	210	10.0575
3.25	7.25	2.2098	7	SPT	6	GC			11	Y	N	120	70	390	78	312	14.9425
8.75	12.75	3.8862	7	SPT	5	GC			11	Y	N	120	70	1050	421.2	628.8	30.1149
15	19	5.7912	7	MC	2	CH	53	4000	90	N	N	90	70	1612.5	811.2	801.3	38.3764
50	54	16.4592															

SAMPLE DEPTH (feet)	CORRECTION FACTORS C_b Borehole Size Factor	C_n Depth Reduction Factor	C_m MC Factor	C_R Drill Rod Length Factor	C_h Hammer Efficiency Factor	C_s Unlined SPT Factor	PENETRATION RESISTANCE N_{60}	MODIFIED PENETRATION RESISTANCE $(N_1)_{60}$	ΔN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND $(N_1)_{60CS}$	STRESS REDUCTION COEFFICIENT (r_d)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (K_s)	CRR STATIC SHEAR CORRECTION (K_a)	CORRECTED CRR (CCRR) Sand	CORRECTED CRR (CCRR) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
1.75	1.15	1.70	0.61	0.75	1.17	1.00	7.37	12.5	1.6	14.13	1.00	0.23	0.147	0.00	1.10	1.00	0.149	0.00	0.90	-	ABOVE G.W.	ABOVE G.W.	0.00
3.25	1.15	1.70	1.00	0.75	1.17	1.00	6.04	10.3	1.6	11.88	1.00	0.29	0.130	0.00	1.10	1.00	0.132	0.00	0.90	-	0.445	YES, LIQ.	0.44
8.75	1.15	1.70	1.00	0.80	1.17	1.00	5.37	9.1	1.6	10.74	0.99	0.39	0.122	0.00	1.10	1.00	0.123	0.00	0.90	-	0.315	YES, LIQ.	0.32
15	1.15	1.70	0.61	0.85	1.17	1.00	1.39	2.4	5.5	7.88	0.98	0.46	NA	3.59	1.08	1.00	NA	3.89	0.90	8.47	NA	NO, NOT LIQ.	8.47
50				1.00							0.86												

Liquefaction Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatso	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatso	Liquefaction Settlement (ft) Ishihara
1.75	ABOVE G.W.	2.00	2.20	2.76	0.00	0.00
3.25	YES, LIQ.	6.00	3.00	3.50	0.18	0.21
8.75	YES, LIQ.	7.00	3.00	3.50	0.21	0.25
15.00	NO, NOT LIQ.	35.00	3.00	0.00	0.00	0.00
50.00						
				Total Settlement (ft)	0.39	0.46
				Total Settlement (in)	4.68	5.46

Lateral Displacement Index and Settlement (B-1)

SAMPLE DEPTH (feet)	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	ΔH (ft)	ΔLDIi (ft)	Vertical Reconsol. Strain ε_v	Liquefaction Settlement (ft) Ishihara
1.75	0.30	0.78865007	0.00	1.63	0.000	0.000	0.00
3.25	0.39	0.86596959	0.39	5.50	2.120	0.034	0.19
8.75	0.44	0.89717385	0.44	6.25	2.728	0.036	0.22
15.00	0.50	0.9445258	0.00	35.00	0.000	0.000	0.00

Settlement		LDI	Lateral Displacement			
			Sloping		Free Face	
ft	in		ft	in	ft	in
0.46	5.46	4.848	1.5	17.5	12.1	144.9

Assumptions:

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

General Data

S	Ground Slope / Liquefiable Layer Slope	0.1
L	Horizontal Distance from the Toe of a Free Face (ft)	30
H	Free Face Height (ft)	10
Gw	Depth to groundwater	2
Mw	Maximum Moment Magnitude	7.9
PGA	Peak Ground Acceleration	0.36

Table D-4 - Seismically-Induced Settelment & Lateral Deformation Calculation - B-5

SAMPLE DEPTH (feet)	ROD LENGTH (feet)	ROD LENGTH (m)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (N) (blows per foot)	SOIL TYPE	PI Clay/Silt	Su Clay/Silt (psf)	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)
4.75	8.75	2.667	7	SPT	13	SP			18	Y	N	115	70
15	19	5.7912	7	MC	2	CH	50	4000	90	N	N	90	70
50													

SAMPLE DEPTH (feet)	TOTAL OVER-BURDEN (psf)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS C_b Borehole Size Factor	C_n Depth Reduction Factor	C_m MC Factor	C_R Drill Rod Length Factor	C_h Hammer Efficiency Factor	C_s Unlined SPT Factor	PENETRATION RESISTANCE N_{60}	MODIFIED PENETRATION RESISTANCE $(N_1)_{60}$	ΔN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND $(N_1)_{60CS}$
4.75	546.25	171.6	374.65	17.943	1.15	1.70	1.00	0.75	1.17	1.00	13.08	22.2	4.1	26.33
15	1468.75	811.2	657.55	31.4919	1.15	1.70	0.61	0.85	1.17	1.00	1.39	2.4	5.5	7.88
50														

SAMPLE DEPTH (feet)	STRESS REDUCTION COEFFICIENT (r_d)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (K_σ)	CRR STATIC SHEAR CORRECTION (K_α)	CORRECTED CRR (CCRR) Sand	CORRECTED CRR (CCRR) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
4.75	1.00	0.34	0.322	0.00	1.10	1.00	0.325	0.00	0.90	-	0.945	YES, LIQ.	0.95
15	0.98	0.51	NA	4.38	1.10	1.00	NA	4.81	0.90	9.44	NA	NO, NOT LIQ.	9.44
50													

Liquefaction Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatso	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatso	Liquefaction Settlement (ft) Ishihara
4.75	YES, LIQ.	10.00	1.10	0.82	0.11	0.08
15.00	NO, NOT LIQ.	40.00	3.00	0.00	0.00	0.00
			Total Settlement (ft)		0.11	0.08
			Total Settlement (in)		1.32	0.98

Lateral Displacement Index and Settlement (B-1)

Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	ΔH (ft)	ΔLDIi (ft)	Vertical Reconsol. Strain ε_v	Liquefaction Settlement (ft) Ishihara
0.08	0.14976835	0.04	7.50	0.296	0.009	0.07
0.50	0.9445258	0.00	35.00	0.000	0.000	0.00

Settlement		LDI	Lateral Displacement			
			Sloping		Free Face	
ft	in		ft	in	ft	in
0.11	1.32	0.296	0.1	1.1	0.7	8.8

Assumptions:

Sample Depth is to top of sample layer.
Fines content estimated based on soil classification where no test data available.
Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.
Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

General Data

S	Ground Slope / Liquefiable Layer Slope	0.1
L	Horizontal Distance from the Toe of a Free Face (ft)	660
H	Free Face Height (ft)	10
Gw	Depth to groundwater	2
Mw	Maximum Moment Magnitude	7.9
PGA	Peak Ground Acceleration	0.36

Table D-5 - Seismically-Induced Settelment & Lateral Deformation Calculation - B-7

SAMPLE DEPTH (feet)	ROD LENGTH (feet)	ROD LENGTH (m)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (N) (blows per foot)	SOIL TYPE	PI Clay/Silt	Su Clay/Silt (psf)	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)
3	7	2.1336	7	SPT	11	SC			12	N	N	120	70
9.75	13.75	4.191	7	SPT	6	SC			14	N	N	120	70
15	19	5.7912	7	MC	2	CH	64	4000	90	N	N	90	70
37.75	41.75	12.7254	7	MC	13	CL	29	1500	75	N	N	115	70
49.75	53.75	16.383	7	MC	29	GC			15	Y	N	120	70
50	54	16.4592											

SAMPLE DEPTH (feet)	TOTAL OVER-BURDEN (psf)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS C_b Borehole Size Factor	C_n Depth Reduction Factor	C_m MC Factor	C_R Drill Rod Length Factor	C_h Hammer Efficiency Factor	C_s Unlined SPT Factor	PENETRATION RESISTANCE N_{60}	MODIFIED PENETRATION RESISTANCE $(N_1)_{60}$	ΔN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND $(N_1)_{60CS}$	STRESS REDUCTION COEFFICIENT (r_d)	CYCLIC STRESS RATIO (CSR)
3	360	62.4	297.6	14.2529	1.15	1.70	1.00	0.75	1.17	1.00	11.07	18.8	2.1	20.89	1.00	0.28
9.75	1170	483.6	686.4	32.8736	1.15	1.70	1.00	0.85	1.17	1.00	6.84	11.6	2.9	14.54	0.99	0.39
15	1642.5	811.2	831.3	39.8132	1.15	1.70	0.61	0.85	1.17	1.00	1.39	2.4	5.5	7.88	0.98	0.45
37.75	4258.75	2230.8	2027.95	97.124	1.15	1.02	0.61	1.00	1.17	1.00	10.64	10.9	5.6	16.42	0.91	0.45
49.75	5698.75	2979.6	2719.15	130.227	1.15	0.90	0.61	1.00	1.17	1.00	23.73	21.4	3.3	24.65	0.86	0.42
50								1.00							0.86	

SAMPLE DEPTH (feet)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (K_s)	CRR STATIC SHEAR CORRECTION (K_a)	CORRECTED CRR (CCRR) Sand	CORRECTED CRR (CCRR) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
3	0.215	0.00	1.10	1.00	0.217	0.00	0.90	-	0.758	YES, LIQ.	0.76
9.75	0.151	0.00	1.10	1.00	0.152	0.00	0.90	-	0.382	YES, LIQ.	0.38
15	NA	3.46	1.08	1.00	NA	3.74	0.90	8.29	NA	NO, NOT LIQ.	8.29
37.75	NA	0.53	1.01	1.00	NA	0.54	0.90	1.20	NA	NO, NOT LIQ.	1.20
49.75	0.244	0.00	0.96	1.00	0.282	0.00	0.90	-	0.575	YES, LIQ.	0.58
50											

Liquefaction Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatso	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatso	Liquefaction Settlement (ft) Ishihara
3.00	YES, LIQ.	9.00	1.40	2.04	0.13	0.18
9.75	YES, LIQ.	6.00	2.20	2.76	0.13	0.17
15.00	NO, NOT LIQ.	22.00	3.00	0.00	0.00	0.00
37.75	NO, NOT LIQ.	9.00	1.90	0.49	0.00	0.00
49.75	YES, LIQ.	4.00	1.20	2.15	0.05	0.09
50.00						
			Total Settlement (ft)		0.31	0.44
			Total Settlement (in)		3.67	5.22

Lateral Displacement Index and Settlement (B-1)

Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	ΔH (ft)	ΔLDIi (ft)	Vertical Reconsol. Strain ε_v	Liquefaction Settlement (ft) Ishihara
0.14	0.47001704	0.08	4.88	0.389	0.022	0.11
0.29	0.77295798	0.29	5.25	1.518	0.029	0.15
0.50	0.9445258	0.00	22.75	0.000	0.000	0.00
0.24	0.69323962	0.00	12.00	0.000	0.000	0.00
0.09	0.253407	0.09	0.25	0.023	0.019	0.00

Settlement		LDI	Lateral Displacement			
			Sloping		Free Face	
ft	in		ft	in	ft	in
0.44	5.22	1.931	0.6	7.0	0.4	4.9

Assumptions:

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

General Data

S	Ground Slope / Liquefiable Layer Slope	0.1
L	Horizontal Distance from the Toe of a Free Face (ft)	660
H	Free Face Height (ft)	10
Gw	Depth to groundwater	2
Mw	Maximum Moment Magnitude	7.9
PGA	Peak Ground Acceleration	0.36

Table D-6 - Seismically-Induced Settelment & Lateral Deformation Calculation - B-7

SAMPLE DEPTH (feet)	ROD LENGTH (feet)	ROD LENGTH (m)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (N) (blows per foot)	SOIL TYPE	PI Clay/Silt	Su Clay/Silt (psf)	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	TOTAL OVER-BURDEN (psf)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS: C_b Borehole Size Factor	C_n Depth Reduction Factor	C_m MC Factor	C_R Drill Rod Length Factor	C_h Hammer Efficiency Factor	C_s Unlined SPT Factor
4	8	2.4384	7	SPT	13	SC			12	N	N	120	70	480	124.8	355.2	17.0115	1.15	1.70	1.00	0.75	1.17	1.00
5.75	9.75	2.9718	7	MC	11	SC			20	N	N	120	70	690	234	456	21.8391	1.15	1.70	0.61	0.75	1.17	1.00
7.25	11.25	3.429	7	SPT	7	SC			20	N	N	120	70	870	327.6	542.4	25.977	1.15	1.70	1.00	0.80	1.17	1.00
19.75	23.75	7.239	7	MC	2	CH	29	4000	90	N	N	90	70	1995	1107.6	887.4	42.5	1.15	1.70	0.61	0.95	1.17	1.00
21.25	25.25	7.6962	7	SPT	23	CL		3000	75	N	N	110	70	2160	1201.2	958.8	45.9195	1.15	1.34	1.00	0.95	1.17	1.00
50	54	16.4592																			1.00		

SAMPLE DEPTH (feet)	PENETRATION RESISTANCE N_{60}	MODIFIED PENETRATION RESISTANCE $(N_1)_{60}$	ΔN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND $(N_1)_{60CS}$	STRESS REDUCTION COEFFICIENT (r_d)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (K_s)	CRR STATIC SHEAR CORRECTION (K_a)	CORRECTED CRR (CCRR) Sand	CORRECTED CRR (CCRR) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
4	13.08	22.2	2.1	24.31	1.00	0.32	0.272	0.00	1.10	1.00	0.275	0.00	0.90	-	0.860	YES, LIQ.	0.86
5.75	6.75	11.5	4.5	15.96	1.00	0.35	0.163	0.00	1.10	1.00	0.164	0.00	0.90	-	0.461	YES, LIQ.	0.46
7.25	7.51	12.8	4.5	17.25	0.99	0.37	0.174	0.00	1.10	1.00	0.176	0.00	0.90	-	0.468	YES, LIQ.	0.47
19.75	1.55	2.6	5.5	8.16	0.96	0.51	NA	3.24	1.08	1.00	NA	3.49	0.90	6.88	NA	NO, NOT LIQ.	6.88
21.25	29.32	39.2	5.6	44.75	0.96	0.51	NA	2.25	1.10	1.00	NA	2.48	0.90	-	NA	YES, LIQ.	0.00
50					0.86												

Liquefaction Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatso	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatso	Liquefaction Settlement (ft) Ishihara
4.00	YES, LIQ.	5.00	1.20	1.62	0.06	0.08
5.75	YES, LIQ.	1.50	1.90	2.76	0.03	0.04
7.25	YES, LIQ.	8.50	1.70	2.76	0.14	0.23
19.75	NO, NOT LIQ.	6.00	3.00	0.00	0.00	0.00
21.25	YES, LIQ.	2.00	0.05	1.30	0.00	0.03
50.00						
				Total Settlement (ft)	0.23	0.38
				Total Settlement (in)	2.81	4.60

Lateral Displacement Index and Settlement (B-1)

SAMPLE DEPTH (feet)	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	ΔH (ft)	ΔLDIi (ft)	Vertical Reconsol. Strain ε_v	Liquefaction Settlement (ft) Ishihara
4.00	0.10	0.27371713	0.05	2.88	0.142	0.012	0.03
5.75	0.25	0.71391507	0.25	1.50	0.372	0.027	0.04
7.25	0.22	0.65525986	0.22	12.50	2.694	0.026	0.32
19.75	0.50	0.94225481	0.00	1.50	0.000	0.000	0.00
21.25	0.00	-1.169366	0.00	28.75	0.000	0.000	0.00

Settlement		LDI	Lateral Displacement			
			Sloping		Free Face	
ft	in		ft	in	ft	in
0.38	4.60	3.209	1.0	11.6	0.7	8.1

Assumptions:

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

General Data

S	Ground Slope / Liquefiable Layer Slope	0.1
L	Horizontal Distance from the Toe of a Free Face (ft)	450
H	Free Face Height (ft)	12
Gw	Depth to groundwater	2
Mw	Maximum Moment Magnitude	7.9
PGA	Peak Ground Acceleration	0.36

Table D-7 - Seismically-Induced Settelment & Lateral Deformation Calculation - B-9

SAMPLE DEPTH (feet)	ROD LENGTH (feet)	ROD LENGTH (m)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (N) (blows per foot)	SOIL TYPE	PI Clay/Silt	Su Clay/Silt (psf)	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	TOTAL OVER-BURDEN (psf)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)
4	8	2.4384	7	SPT	6	SC			23	N	N	115	70	460	124.8	335.2	16.0536
8.75	12.75	3.8862	7	SPT	10	SC			23	N	N	115	70	1006.25	421.2	585.05	28.0196
13.75	17.75	5.4102	7	SPT	3	SC			21	N	N	115	70	1581.25	733.2	848.05	40.6154
18.75	22.75	6.9342	7	MC	6	GC			6	Y	N	120	70	2181.25	1045.2	1136.05	54.4085
23.75	27.75	8.4582	7	SPT	10	GC			6	Y	N	120	70	2781.25	1357.2	1424.05	68.2016
28.5	32.5	9.906	7	SPT	33	SC			10	N	N	130	70	3398.75	1653.6	1745.15	83.58
32.75	36.75	11.2014	7	SPT	25	SC			12	N	N	130	70	3951.25	1918.8	2032.45	97.3396
38.75	42.75	13.0302	7	SPT	45	CI	50	4000	75	N	N	120	70	4671.25	2293.2	2378.05	113.891
50	54	16.4592															

SAMPLE DEPTH (feet)	CORRECTION FACTORS: C_b Borehole Size Factor	C_n Depth Reduction Factor	C_m MC Factor	C_R Drill Rod Length Factor	C_h Hammer Efficiency Factor	C_s Unlined SPT Factor	PENETRATION RESISTANCE N_{60}	MODIFIED PENETRATION RESISTANCE $(N_1)_{60}$	ΔN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND $(N_1)_{60CS}$	STRESS REDUCTION COEFFICIENT (r_d)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (K_s)	CRR STATIC SHEAR CORRECTION (K_a)	CORRECTED CRR (CCRR) Sand	CORRECTED CRR (CCRR) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
4	1.15	1.70	1.00	0.75	1.17	1.00	6.04	10.3	4.9	15.15	1.00	0.32	0.156	0.00	1.10	1.00	0.157	0.00	0.90	-	0.485	YES, LIQ.	0.49
8.75	1.15	1.70	1.00	0.80	1.17	1.00	10.73	18.2	4.9	23.13	0.99	0.40	0.249	0.00	1.10	1.00	0.252	0.00	0.90	-	0.625	YES, LIQ.	0.62
13.75	1.15	1.70	1.00	0.85	1.17	1.00	3.42	5.8	4.6	10.45	0.98	0.43	0.118	0.00	1.09	1.00	0.121	0.00	0.90	-	0.277	YES, LIQ.	0.28
18.75	1.15	1.47	0.61	0.95	1.17	1.00	4.66	6.8	0.0	6.87	0.97	0.43	0.092	0.00	1.05	1.00	0.097	0.00	0.90	-	0.212	YES, LIQ.	0.21
23.75	1.15	1.22	1.00	0.95	1.17	1.00	12.75	15.6	0.0	15.60	0.95	0.44	0.152	0.00	1.05	1.00	0.161	0.00	0.90	-	0.348	YES, LIQ.	0.35
28.5	1.15	1.06	1.00	0.95	1.17	1.00	42.06	44.4	1.1	45.55	0.94	0.43	1.905	0.00	1.06	1.00	2.000	0.00	0.90	-	2.000	NO, NOT LIQ.	2.00
32.75	1.15	1.01	1.00	1.00	1.17	1.00	33.54	34.0	2.1	36.04	0.92	0.42	1.266	0.00	1.01	1.00	1.390	0.00	0.90	-	2.000	NO, NOT LIQ.	2.00
38.75	1.15	0.97	1.00	1.00	1.17	1.00	60.38	58.5	5.6	64.06	0.90	0.42	NA	1.21	0.97	1.00	NA	1.17	0.90	2.82	NA	NO, NOT LIQ.	2.82
50				1.00							0.86												

Liquefaction Settlement / Lateral Displacement Index and Settlement (B-1)

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatso	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatso	Liquefaction Settlement (ft) Ishihara	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	ΔH (ft)	ΔLDIi (ft)	Vertical Reconsol. Strain ε_v	Liquefaction Settlement (ft) Ishihara
4.00	YES, LIQ.	8.00	1.90	2.76	0.15	0.22	0.27	0.74827173	0.27	4.38	1.185	0.029	0.12
8.75	YES, LIQ.	5.00	1.20	2.15	0.06	0.11	0.11	0.34351177	0.11	5.00	0.555	0.020	0.10
13.75	YES, LIQ.	4.00	3.00	3.50	0.12	0.14	0.45	0.90403768	0.45	5.00	2.252	0.036	0.18
18.75	YES, LIQ.	6.00	4.00	4.50	0.24	0.27	0.50	0.9475684	0.50	5.00	2.500	0.046	0.23
23.75	YES, LIQ.	3.00	1.90	2.76	0.06	0.08	0.26	0.72934728	0.26	4.75	1.225	0.028	0.13
28.50	NO, NOT LIQ.	6.00	0.05	0.00	0.00	0.00	0.00	-1.232631	0.00	4.25	0.000	0.000	0.00
32.75	NO, NOT LIQ.	3.00	0.05	0.00	0.00	0.00	0.02	-0.5106724	0.00	6.00	0.000	0.000	0.00
38.75	NO, NOT LIQ.	8.00	0.05	0.00	0.00	0.00	0.00	-2.7728731	0.00	11.25	0.000	0.000	0.00
50.00													

	Tokimatso	Ishihara
Total Settlement (ft)	0.63	0.82
Total Settlement (in)	7.55	9.85

Settlement		LDI	Lateral Displacement			
			Sloping		Free Face	
ft	in		ft	in	ft	in
0.82	9.85	7.717	2.3	27.8	2.5	30.6

Assumptions:

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

General Data

S	Ground Slope / Liquefiable Layer Slope	0.1
L	Horizontal Distance from the Toe of a Free Face (ft)	330
H	Free Face Height (ft)	12
Gw	Depth to groundwater	2
Mw	Maximum Moment Magnitude	7.9
PGA	Peak Ground Acceleration	0.36

Table D-8 - Seismically-Induced Settelment & Lateral Deformation Calculation - B-10

SAMPLE DEPTH (feet)	ROD LENGTH (feet)	ROD LENGTH (m)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (N) (blows per foot)	SOIL TYPE	PI Clay/Silt	Su Clay/Silt (psf)	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	TOTAL OVER-BURDEN (psf)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)
7.75	11.75	3.5814	7	SPT	5	SC			12	Y	N	120	70	930	358.8	571.2	27.3563
18.75	22.75	6.9342	7	MC	2	CH	43	4000	90	N	N	90	70	1920	1045.2	874.8	41.8966
44.75	48.75	14.859	7	MC	18	CL		2000	75	N	N	115	70	4910	2667.6	2242.4	107.395
50																	

SAMPLE DEPTH (feet)	CORRECTION FACTORS C_b Borehole Size Factor	C_n Depth Reduction Factor	C_m MC Factor	C_R Drill Rod Length Factor	C_h Hammer Efficiency Factor	C_s Unlined SPT Factor	PENETRATION RESISTANCE N_{60}	MODIFIED PENETRATION RESISTANCE $(N_1)_{60}$	ΔN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND $(N_1)_{60CS}$	STRESS REDUCTION COEFFICIENT (r_d)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (K_σ)	CRR STATIC SHEAR CORRECTION (K_α)	CORRECTED CRR (CCRR) Sand	CORRECTED CRR (CCRR) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
7.75	1.15	1.70	1.00	0.80	1.17	1.00	5.37	9.1	2.1	11.20	0.99	0.38	0.125	0.00	1.10	1.00	0.127	0.00	0.90	-	0.331	YES, LIQ.	0.33
18.75	1.15	1.70	0.61	0.95	1.17	1.00	1.55	2.6	5.5	8.16	0.97	0.50	NA	3.29	1.08	1.00	NA	3.54	0.90	7.14	NA	NO, NOT LIQ.	7.14
44.75	1.15	0.97	0.61	1.00	1.17	1.00	14.73	14.3	5.6	19.86	0.88	0.45	NA	0.64	0.99	1.00	NA	0.64	0.90	-	NA	YES, LIQ.	0.00
50																							

Liquefaction Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatso	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatso	Liquefaction Settlement (ft) Ishihara
7.75	YES, LIQ.	16.00	3.00	3.50	0.48	0.56
18.75	NO, NOT LIQ.	26.00	3.00	0.00	0.00	0.00
44.75	YES, LIQ.	8.00	1.70	2.76	0.14	0.22
			Total Settlement (ft)		0.62	0.78
			Total Settlement (in)		7.39	9.37

Lateral Displacement Index and Settlement (B-1)

Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	ΔH (ft)	ΔLDIi (ft)	Vertical Reconsol. Strain ε_v	Liquefaction Settlement (ft) Ishihara
0.42	0.88529277	0.42	9.38	3.891	0.035	0.33
0.50	0.94225481	0.00	26.00	0.000	0.000	0.00
0.16	0.52541055	0.00	5.25	0.000	0.000	0.00

Settlement		LDI	Lateral Displacement			
			Sloping		Free Face	
ft	in		ft	in	ft	in
0.78	9.37	3.891	1.2	14.0	1.6	19.8

Assumptions:

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

APPENDIX E

SHAKE ANALYSES

In this appendix, we present the results of the site-specific ground motion evaluation for the subject project.

E.1 SITE CHARACTERIZATION

The site is classified as Type F based on Table 16-13-5.2 of the 2010 ASCE 7-10.

E.2 DEVELOPING TARGET UNIFORM HAZARD SPECTRA

In accordance with Section 21 of the ASCE 7-10, a maximum considered earthquake (MCE) response spectrum was developed for bedrock. A probabilistic seismic hazard analysis (PSHA) was used to estimate the target uniform hazard spectra (UHS) for the design basis seismic event discussed above. This analysis involves the selection of an appropriate predictive relationship to estimate the ground motion parameters, and, through probabilistic methods, determination of spectral accelerations.

Attenuation Relationship

For horizontal ground motions, we selected relationships presented by Abrahamson and Silva (2008), Boore and Atkinson (2008) Campbell and Bozorgnia (2008), and Chiou and Youngs (2008). These predictive relationships are pertinent to shallow crustal earthquakes in active tectonic regions such as the project area.

The PSHA results were obtained by taking an average of the hazard results from each set of attenuations relationships used for horizontal ground motions. The uncertainty in the predicted ground motion is taken into consideration by including a magnitude dependent standard error in the probabilistic analysis.

Peak Ground Acceleration

SHAKE2000 software program was used in our seismic analyses and the calculated peak ground horizontal acceleration for the alluvium soil condition is presented in Section 3.2.2 of the main text.

E.3 DEVELOPING RESPONSE SPECTRUM CURVE

Selecting Representative Input Ground Motions

Three sets of time histories were selected from the PEER Strong Motion Database as references based on the closest similarity of seismological and geological features. Detailed information of the selected time histories is provided in the following Table.

Detailed Information of Selected Reference Records

Earthquake	Station	Style of Faulting	Magnitude (Mw)	Distance to Fault Rupture (km)
Loma Prieta 10/17/1989,	Dumbarton Bridge	Strike Slip	6.95	20
Loma Prieta 10/17/1989,	Lexington Dam	Strike Slip	6.95	5
Loma Prieta 10/17/1989,	Palo Alto	Strike Slip	6.95	17

The above selected horizontal motions were spectrally matched to the target horizontal UHS.

SHAKE 2000 software program was used to generate time-histories for the site. AGS performed two geotechnical soil borings at the site in 2011. Subsurface condition were described in the main text. The subsurface was divided into thirteen sublayers with maximum thickness of 5 feet and extrapolated to about 60 feet to bedrock.

Material property models developed by Seed et al. (1986) for fill, Darendeli (2001) for Younger bay Mud and Older Bay Mud, and Schnabel (1973) for rock were used in our Shake analysis. Plates E-1 and E-2 shows input soil parameters and material models used for Bay Mud.

Digital Data of Spectrum-Matched Acceleration Time Histories

The original response spectrum acceleration for Loma Prieta earthquake at Lexington Dam Station is shown on Plate E-3.

REFERENCES

Abrahamson, N., & Silva, W. 2008. Summary of the Abrahamson & Silva NGA ground-motion relations. Earthquake Spectra, 24(1), 67–97.

Abrahamson, N., & Silva, W. 2009 (Aug). Errata for "Summary of the Abrahamson and Silva NGA ground-motion relations" by Abrahamson, N. A. and W. J. Silva. Published on PEER NGA website.

Boore, D. M., & Atkinson, G. M. 2008. Ground-motion prediction equations for the average horizontal component of PGA, PGV, and 5%-damped PSA at spectral periods between 0.01s and 10.0s. Earthquake Spectra, 24(1), 99–138.

Campbell, K. W., & Bozorgnia, Y. 2008. NGA ground motion model for the geometric mean horizontal component of PGA, PGV, PGD and 5% damped linear elastic response spectra for periods ranging from 0.01 to 10 s. Earthquake Spectra, 24(1), 139–171.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J. (2003), The Revised 2002 California Probabilistic Seismic Hazards Maps, California Geological Survey, June 2003. Available at website

http:llwww.consrv.ca.govICGS/rghmlpshalfault parameters/pdf/2002, CA Hazard Maps.pdf.

Chiou, B. S.-J., & Youngs, R. R. 2008. An NGA model for the average horizontal component of peak ground motion and response spectra. Earthquake Spectra, 24(1), 173–215.

Darendeli, M.B. (2001), Development of a new Family of Normalized Modulus Reduction and Material Damping Curves, Ph.D. Dissertation. UTAM.

Risk Engineering (2005), EZ-FRISK, Version 7.12 (http:l/www.riskenq.com).

Shake 2000, Version 3.4.0 (2007)