Appendix G

Geotechnical Reports

APPENDIX G

GEOTECHNICAL REPORTS

REGIONAL GROUNDWATER STORAGE AND RECOVERY PROJECT

April 2013

INTRODUCTION

This Appendix includes the three geotechnical reports that were prepared for the Regional Groundwater Storage and Recovery (GSR) Project. Due to the length of the appendices for the geotechnical reports, the appendices are not included.

The reports provided in this Appendix include the following:

- *Geotechnical Report – South Westside Groundwater Basin Conjunctive Use Project, April 2009.* This report includes Section 6.3, Densification Improvements, which provides optional construction methodologies for densification of soils. The GSR Project Description does not include use of these optional methodologies and relies instead on appropriate structural design of all structures.

- *Final Geotechnical Report – CUP Well Locations CUP-11A, CUP-23, CUP-36-1, CUP-44-1, and CUP-M-1, South Westside Basin Groundwater Storage and Recovery Project, December 2009*

- *Geotechnical Report – CUP-3A and CUP-7 sites, Regional Groundwater Storage and Recovery Project, November 2011 (Revised January 2012)*

These geotechnical reports utilize a different numbering system for well sites than the EIR. The table below provides the EIR site numbers for each of the site numbers used in the geotechnical reports.

EIR Site Name	Geotechnical Report Site Name
1	3A
2	6
3	5
4	7
5	10A
6	11A
7	18
8	19
9	23
10	22A
11	31
12	36-1
13	41-4
14	44-2
15	44-1
16	M-1
17 (Alternate)	20A
18 (Alternate	22
19 (Alternate)	36-2

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GEOTECHNICAL REPORT – SOUTH WESTSIDE GROUNDWATER BASIN CONJUNCTIVE USE PROJECT, APRIL 2009

GEOTECHNICAL CONSULTANTS, INC.
Geotechnical Engineering • Geology • Hydrogeology

GEOTECHNICAL REPORT
SOUTH WESTSIDE GROUNDWATER BASIN
CONJUNCTIVE USE PROJECT
SAN MATEO COUNTY, CA

April 2009

Prepared for:

Kennedy/Jenks Consultants
303 Second Street, Suite 300 South
San Francisco, CA 94107

Owner:

San Francisco Public Utilities Commission

SF08034

500 Sansome Street, Suite 402 • San Francisco, CA 94111 • (415) 981-9950

TABLE OF CONTENTS

Page

TABLES

FIGURES

PLATES

APPENDIX A – SUPPORTING GEOTECHNICAL DATA

INTRODUCTION

This geotechnical report presents the findings, conclusions, and recommendations of our geotechnical study performed for proposed buildings to facilitate groundwater well stations, and chemical treatment and filtration facilities at five designated sites located in the northern part of San Mateo County, California (Figure 1 – Site Location Map). The proposed wells are part of the South Westside Groundwater Basin Conjunctive Use Project (SWGBCUP), a project being developed through the coordination of the San Francisco Public Utilities Commission (SFPUC) and three partner agencies (California Water Service Company [Cal Water], the City of Daly City and the City of San Bruno). This geotechnical report is being prepared for Kennedy/Jenks Consultants as part of their design services contract with the SFPUC.

We anticipate that the proposed station buildings will typically be constructed with concrete masonry units (CMU), although the material selection will depend on the surrounding structures. The building footprint area for proposed station buildings that house a monitoring well only is approximately 640 square feet. The footprint area for a proposed station building expands to approximately 916 square feet when the building includes chemical treatment facilities in addition to the well. A proposed station building measuring approximately 1,742 square feet is anticipated when the building houses a monitoring well and the facilities for chemical treatment and filtration. Geotechnical recommendations for additional improvements such as new pipeline connections and upgrades, which may require additional geotechnical borings, were not part of our scope of work.

WORK PERFORMED

In accordance with our scope of work as documented in the Subcontract Agreement (Amendment No. 3) with Kennedy/Jenks Consultants, Incorporated (KJ) dated November 17, 2008 and subsequent conversations with personnel from KJ, we have completed the scope of work described below:

1. **Exploratory Drilling.** We explored subsurface conditions by means of drilling one hollow-stem auger boring at each of the five sites designated as CUP-10A, -18, -19, -22A and -41-4. To maintain consistency with the site numbering, our borings have been accordingly labeled as GB-10A, -18, -19, -22A and -41-4 for the subject sites. Boring number, date of drilling, surface elevation and depth are presented for each boring and summarized in Table 1 – Summary of Geotechnical Borings. The surface elevations of the borings were evaluated from topographic maps which were prepared by Chaudhary & Associates from their field surveys in March and September of 2008. The surface elevations presented in this report are approximate. All elevations on Table 1, and referred to throughout this report (unless otherwise noted), are with respect to 1988 North American Vertical Datum (NAVD 88).

FIGURE 1
SITE LOCATION MAP

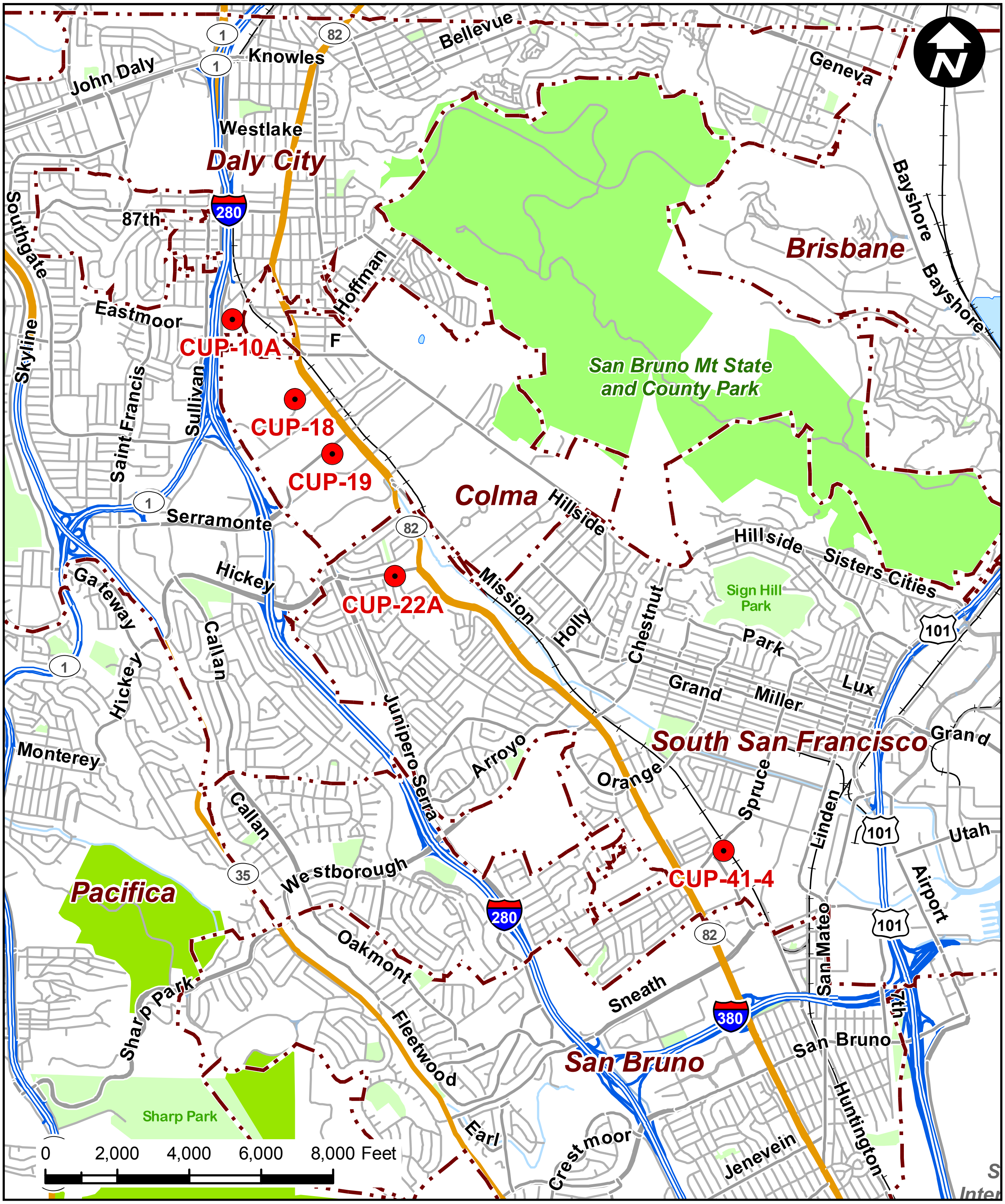


TABLE 1 – SUMMARY OF GEOTECHNICAL BORINGS

Boring	Date Drilled	Approximate Surface Elevation (feet, NAVD 88)	Depth (feet)
GB-10A	12/15/2008	+ 193	30
GB-18	12/15/2008	+ 173	30
GB-19	12/15/2008	+ 112	30.5
GB-22A	12/16/2008	+ 100	30.5
GB-41-4	12/16/2008	(1)	30.5

1. Surface elevation relative to NAVD 88 datum is not available. A preliminary topographic map showing a field survey by Chaudhary & Associates on March 14, 2008 indicates a temporary benchmark was used as a reference.

We visually classified the soil during drilling. We recovered split-spoon (Standard Penetration Test) samples and relatively undisturbed 2 ½ inch diameter sleeve samples using a split-barrel sampler. Selected samples were transferred to a laboratory for testing. The boring locations are shown on Plates 1 through 5 – Boring Location Maps. Boring logs are presented in Appendix A – Supporting Geotechnical Data.

2. **Laboratory Testing.** We performed moisture, density, grain size analysis, Atterberg limits, direct shear and corrosion tests on selected soil samples to measure pertinent index and engineering properties. The laboratory test results are presented on the figures in Appendix A, and on the boring logs on Plates A-1.1 through -1.5.

3. **Engineering Analysis.** We analyzed subsurface conditions and laboratory test results, and reviewed regional and local geology and seismicity. Additionally, we analyzed the following geotechnical parameters:

   - Seismic hazards evaluation including strong ground shaking, liquefaction, seismic and dynamic settlements, and seismically-induced landslides;
   - Seismic design parameters in accordance with the 2006 International Building Code;
   - Bearing capacity (allowable and ultimate) and modulus of subgrade reaction (vertical soil springs) for shallow footings and grade beams, and mat foundations; and
   - Lateral earth pressures (active, passive, at-rest, and seismic increment) and base friction coefficients for restrained and unrestrained walls and/or buried footings.

4. **Report.** We prepared this report presenting our geotechnical findings, conclusions, and recommendations for the proposed improvements at the five subject sites for the SWGBCUP.

FINDINGS

SITE CONDITIONS

The five subject sites are located within the north portion of the South Westside Groundwater Basin in San Mateo County, California. The ground surface along an alignment which roughly transects the five sites, and parallels El Camino Real, generally descends in a northwest-to-southeast direction from elevations of approximately 200 feet to 20 feet above mean sea level for a distance of approximately 4 miles.

The northernmost site CUP-10A is located to the southeast of the intersection between Junipero Serra Boulevard and B Street in Daly City. As indicated on the general layout of the proposed improvements on Plate 1 – Boring Location Map for CUP-10A, the site is located on a relatively flat, abandoned, asphalt paved parking lot. The site is surrounded by parking lots to the south and west, residential/commercial property to the east, and sidewalk abutting B Street to the north. Existing underground water main pipelines (Baden Merced, San Andreas Nos. 2 and 3, Sunset Supply) and proposed connection main and pump-to-waste pipelines are also shown on Plate 1.

Approximately ½ mile to the southeast from CUP-10A, CUP-18 is located to the southwest of the intersection between Colma Boulevard and El Camino Real in the Town of Colma. As indicated on the general layout of the proposed improvements on Plate 2 – Boring Location Map for CUP-18, the site is located on grassy terrain which descends on a mildly sloping (7:1 horizontal to vertical side slope ratio) terrain in a northwest-to-southeast direction. The site is surrounded by a paved turnout for Colma Boulevard to the south, a small maintenance/operations facility building to the west, moderately wooded area to the east, and the Woodlawn Cemetery to the north. Existing underground water main pipelines (Baden Merced, and San Andreas Nos. 2 and 3) and proposed connection main and pump-to-waste pipelines are also shown on Plate 2.

A further 1/3 mile to the southeast from CUP-18, CUP-19 is located to the southwest of the intersection between El Camino Real and Serramonte Boulevard in the Town of Colma. The general layout of the proposed improvements on Plate 3 – Boring Location Map for CUP-19 shows a relatively flat, recently re-graded site which is surrounded to the east by a parking lot for the Kohl's department store, to the west by a concrete retaining wall which retains an automobile dealer parking lot to higher grade, to the north and south by relatively flat, re-graded grounds, and further to the north by Serramonte Boulevard. Existing underground water main pipelines (Baden Merced, and San Andreas Nos. 2 and 3) and proposed connection main and pump-to-waste pipelines are also shown on Plate 3.

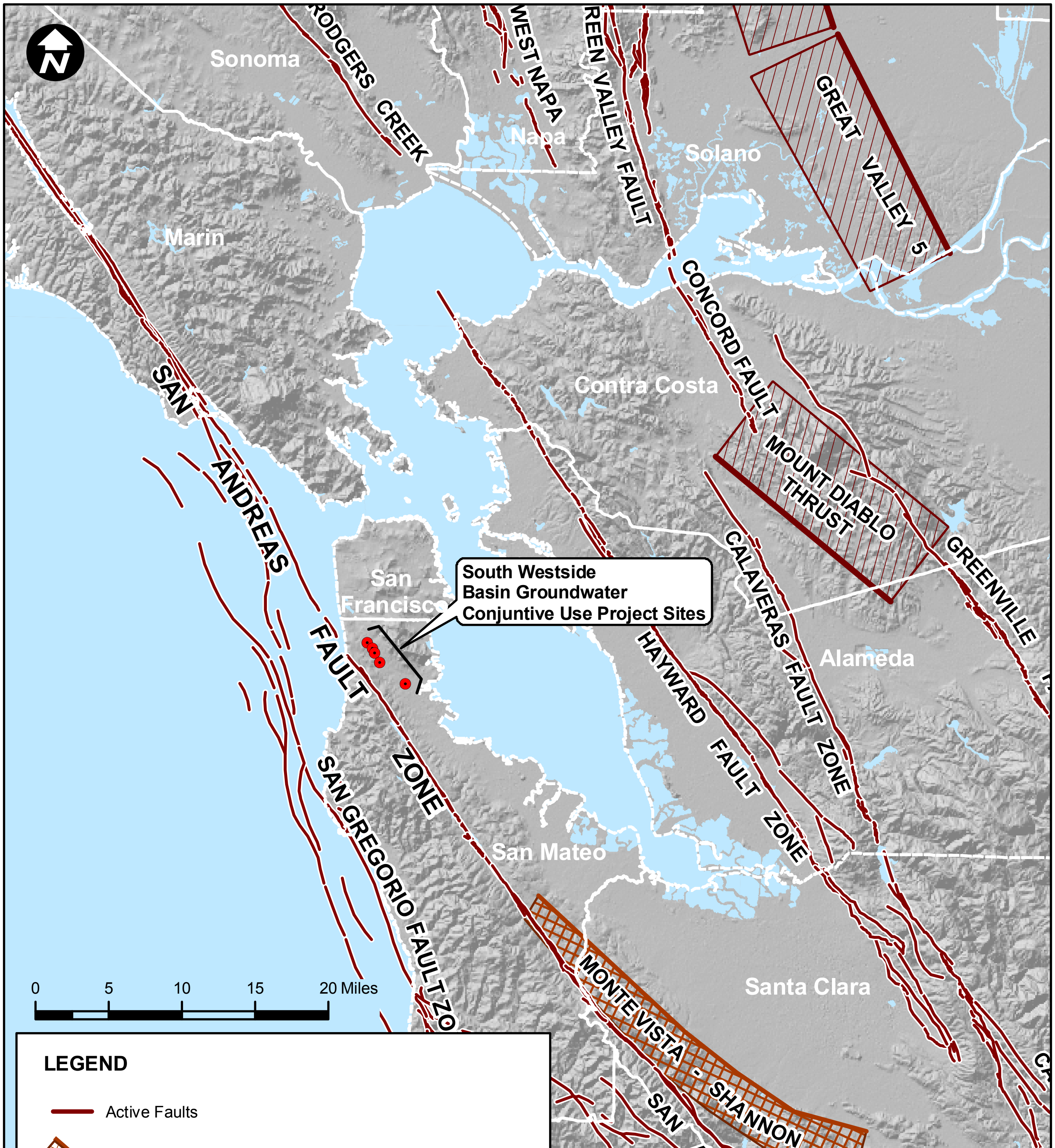
Approximately ¾ mile to the southeast from CUP-19, CUP-22A is located to the southwest of the intersection between Camaritas Avenue and Hickey Boulevard in the City of South San Francisco. The general layout of the proposed improvements on Plate 4 – Boring Location Map for CUP-22A shows a relatively flat, recently re-graded site which is surrounded to the north and east by sidewalks abutting Hickey Boulevard and Camaritas Avenue, to the south and west by relatively flat, recently re-graded grounds, and further to the west by a landscaped slope which ascends to a residential development. Existing underground water main pipelines (Baden Merced, and San Andreas Nos. 2 and 3) and proposed connection main and pump-to-waste pipelines are also shown on Plate 4.

The southernmost site of CUP-41-4 is located approximately 2¼ miles to the southeast from CUP-22A, and is situated to the northeast from the intersection between Huntington Avenue and South Spruce Avenue in South San Francisco. As shown on Plate 5 - Boring Location Map for CUP-41-4, this site is located on relatively flat terrain which is covered with landscaping mulch, lawn and scattered timber logs. The areas surrounding the site are also relatively flat. The site is surrounded to the east by a paved walkway trail which is underlain by the Bay Area Rapid Transit (BART) subway tunnel, to the south by a parking lot for a commercial building, to the west by a two-story commercial office building and its parking lot, and to the north by the sidewalk abutting South Spruce Avenue. Existing underground water main pipelines (Baden Merced, and San Andreas Nos. 2 and 3) and proposed connection main and pump-to-waste pipelines are also shown on Plate 5.

SEISMICITY

The San Francisco Bay Area contains several active faults that could cause strong ground shaking at the project site. Figure 2 – Regional Fault Map shows faults in the vicinity of the subject sites. The San Andreas (1906 Rupture Event and Peninsula Segment) are the nearest active faults and are located within 1.6 miles of the CUP-10A, -18, -19 and -22A sites, and within 2.1 miles of the CUP-41-4 site. The San Andreas is the primary component in a complex system of right-lateral, strike-slip faults; including the San Andreas, San Gregorio-Seal Cove, Hayward, and Calaveras faults; collectively known as the San Andreas fault system. The San Andreas, San Gregorio-Seal Cove, Hayward, and Calaveras faults have produced measurable historic ground motion and movement. The San Andreas fault is capable of producing an earthquake of an estimated maximum magnitude of 7.9. This segment is estimated to have recurrence intervals on the order of 200 years. A summary of nearby faults is presented in Table 2 – Active and Potentially Active Faults.

FIGURE 2
REGIONAL ACTIVE FAULT MAP



LEGEND

Active Faults

Reverse Fault (rectangle represents projection of the fault plane to the surface)

Blind Thrust Faults (faults do not intersect the surface, mapped trace represents projection of upper edge of the fault to surface; rectangle represents projection of the fault plane to the surface)

Fault Data Sources: CGS 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0, Bryant, W. A. (compiler) and Cao, et. al., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, Appendix A - 2002 California Fault Parameters

TABLE 2 – ACTIVE AND POTENTIALLY ACTIVE FAULTS

Fault (Segment or Event)	Distance to Fault (miles)					Estimated Maximum Earthquake Magnitude (1)	Historic Earthquakes (2)	
	CUP-10A	CUP-18	CUP-19	CUP-22A	CUP-41-4		Year	Magnitude
San Andreas							1838	6.8
(1906 rupture)	1.6 (3)	1.6 (3)	1.6 (3)	1.6 (3)	2.1 (3)	7.9 (3)	1898	6.2
(Peninsula)	1.6	1.6	1.6	1.6	2.1	7.2	1906	8.1
(North)	11.2	11.8	19.5	12.9	15.0	7.7	1989	7.1
San Gregorio-Seal Cove							N/A	N/A
(North)	5.5	5.7	5.8	5.8	7.0	7.2		
Hayward							1868	6.8
(North)	17.1	17.1	17.1	17.2	16.5	6.5		
(South)	18.8	18.6	18.5	18.3	17.0	6.7		
Monte Vista-Shannon	20.9	20.4	20.0	19.3	17.1	6.7	N.A.	N.A.
Calaveras							1861	5.3
(North)	26.7	26.6	26.5	26.5	25.5	6.8	1955	5.5
(South)	40.9	40.4	40.1	39.5	37.4	6.2	1979	5.9
							1984	6.1
							2007	5.4

(1) Maximum Moment Magnitude based on California Geological Survey (CGS) fault parameters as updated in 2002 (Cao, et al., 2003), or as suggested by the SFPUC's General Seismic Requirements (SFPUC, 2006).
(2) Historic earthquakes shown may have occurred in other segments of the noted fault.
(3) The 1906 rupture event assumes rupture along the North Coast, Peninsula and Santa Cruz Mountains segments to San Juan Bautista. Maximum magnitude is based on the average 5 m displacement during the 1906 event (WGCEP, 2003; Petersen, et al., 1996).

GEOLOGY

The San Francisco Bay Area is located within the Coast Ranges Geomorphic Province. Past episodes of tectonism have folded and faulted the bedrock, creating the regional topography of the northwest trending ridges and valleys characteristic of the Coast Ranges Geomorphic Province. The San Francisco Bay and vicinity occupy a structurally controlled basin within the province. Late Pleistocene and Holocene sediments (less than 1 million years old) were deposited in the basin as it subsided.

The subject sites at CUP-10A and -18 are located in areas mapped as Colma Formation (Brabb, et al., 1988). Other sedimentary deposits mapped in close proximity to these sites include natural levee deposits, alluvial fan deposits, stream terrace deposits, and Merced Formation. The CUP-19, -22A and -41-4 sites are located in areas mapped as natural levee deposits and Colma Formation. Other sedimentary deposits mapped in close proximity to these sites include historic artificial fill, alluvial fan and stream terrace deposits, and Merced Formation. The geology in the project vicinity is shown on Figure 3 – Regional Geologic Map. Based on a regional geologic study as compiled as a regional geologic cross section of the Westside Basin – Lake Merced (SFPUC, 2008), the Franciscan Complex bedrock is anticipated to be on the order of 600 to 700 feet below ground surface at the subject sites. Geologic maps (Brabb, et al., 1988) describe the identified geologic units as follows:

- **af:** Artificial fill – loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations and thicknesses which may exceed 30 m; some compacted and quite firm, but fill made before 1965 is nearly everywhere not compacted and consists of simply dumped materials.

- **Qhl:** Natural levee deposits (Holocene) – loose, moderately to well-sorted sandy or clayey silt grading to sandy or silty clay; porous and permeable and provide conduits for transport of groundwater. Levee deposits border stream channels, usually both banks, and slope away to flatter floodplains and basins. Abandoned levee systems, no longer bordering stream channels, may be present.

- **Qof:** Older alluvial fan and stream terrace deposits (Pleistocene) – poorly consolidated and poorly indurated well- to poorly-sorted sand and gravel with varying thickness probably less than 30 m.

- **Qc:** Colma Formation (Pleistocene) – yellowish-gray, gray, yellowish-orange and red-brown, friable to loose, fine- to medium-grained arkosic sand with subordinate gravel, silt and clay; total thickness is typically unknown, but may up to 60 m.

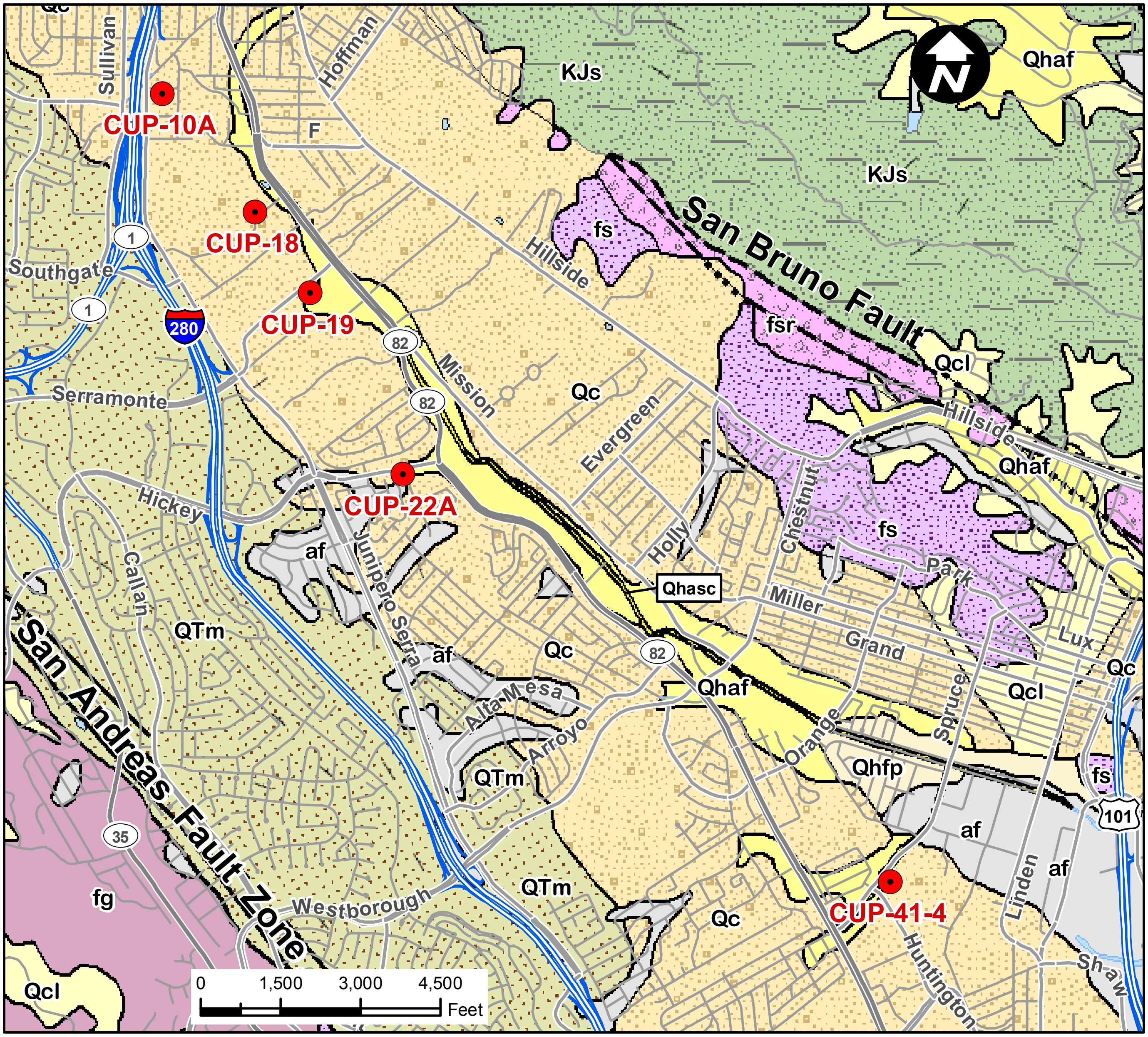
- **QTm:** Merced Formation (lower Pleistocene and upper Pliocene) – medium gray to yellowish gray, yellowish orange, medium- to very fine-grained, poorly indurated to friable sandstone, siltstone, and claystone, with some conglomerate lenses and a few friable beds of white volcanic ash; sandstone is typically silty, clayey, or conglomeratic; fossiliferous conglomerate is well cemented.

- **Qsr:** Slope debris and ravine fill - angular rock fragments in sand, silt, and clay matrix; generally light yellow to reddish brown. Maximum thickness approximately 80 feet.

- **Qd:** Dune sand - clean well-sorted fine to medium sand; yellowish brown to light gray.

- **KJf:** Franciscan Complex – mostly graywacke and shale (fs), and partly unnamed sandstone (KJs); fs consists of greenish gray to buff, fine- to coarse-grained sandstone, with interbedded siltstone and shale; KJs consists of dark gray to yellowish brown graywacke interbedded with shale in approximately equal amounts and resembling fs but the bedding in KJs is better developed.

FIGURE 3
REGIONAL GEOLOGIC MAP



Source: Brabb et. al., 1998, USGS OFR 98-137.

LEGEND

CUP-22A Conjuntive Use Project (CUP) Sites

Geologic Units

af	Artificial Fill
Qhasc	Artificial Stream Channels
Qhfp	Floodplain Deposits
Qhaf	Alluvial Fan and Fluvial Deposits
Qcl	Colluvium
Qc	Colma Formation
QTm	Merced Formation
KJs	Unnamed Sandstone of San Bruno Mtn.
fs	Franciscan Sandstone
fg	Franciscan Greenstone
fsr	Franciscan Melange

Structual Features

- geologic contact
- fault, approx. located
- fault, certain
- fault, concealed

EARTH MATERIALS

The exploratory borings for this investigation at the CUP-10A and -18 sites encountered artificial fill which was underlain by soils of Colma Formation (Qc). An intermediate stratum of natural levee deposits (Qhl) was encountered between the artificial fill and underlying soils of Colma Formation at the CUP-19 and -41-4 sites. At the CUP-22A site, artificial fill was underlain by soils of natural levee deposits to the total depth of exploration.

Artificial Fill. Artificial fill was encountered to depths of approximately 4 to 5 feet in borings GB-10A, -19 and -22A, and approximately 2 feet in borings GB-18 and -41-4. The fill was mainly comprised of light yellowish brown, damp to moist, loose to medium dense, silty fine sand. The origin of this fill at the subject sites of CUP-10A and -18 was likely a result of grading and reuse of on-site, near surface materials of Colma Formation (Qc). The fill at the CUP-19, -22A and -41-4 sites was likely to have originated from on-site, near surface soils of natural levee deposits (Qhl). At the CUP-10A site, the artificial fill was overlain by an asphalt concrete pavement. A surface layer of landscape bark was encountered above the artificial fill at the CUP-41-4 site.

Natural Levee Deposits. At the CUP-19, -22A and -41-4 sites, artificial fill was immediately underlain by soils of the natural levee deposits (Qhl). The thicknesses of the natural levee deposits encountered at the CUP-19 and -41-4 sites are 22, and 15 feet, respectively. The natural levee deposits were underlain by soils of the Colma Formation (Qc). The thickness of the natural levee deposits at the CUP-22A site exceeds 26.5 feet as the bottom contact of the natural levee deposits was not encountered within the total depth of exploration in boring GB-22A. The upper 6 to 8 feet of the soils in the natural levee deposits at the three subject sites consisted of light yellowish to olive brown, damp to moist, loose to medium dense, poorly graded fine sand to silty fine sand. The remaining lower portion of the soils in the natural levee deposits consisted of moist, medium dense to very dense, silty fine sand to sandy silt, and damp to moist, medium stiff to very stiff, sandy clay to clayey sand with some silt. Measured total unit weight ranged from 111 to 131 pounds per cubic feet (pcf), with a moisture content that ranged from 5 to 16 percent.

Colma Formation. Soils of the Colma Formation (Qc) were encountered at the CUP-10A, -18, -19 and -41-4 sites. At the CUP-10A and -18 sites, the soils of Colma Formation were encountered at relatively shallow depths of 5 and 2 feet, respectively, directly underlying the artificial fill. The Colma Formation soils at these two sites consisted of damp to moist, medium dense to very dense, poorly graded fine sand to silty fine sand. At GB-19 and -41-4 sites, the Colma Formation soils, which were encountered at deeper depths of 27 and 17 feet, respectively, were overlain by the natural levee deposits. The Colma Formation soils at these two sites consist of light yellowish to orange brown, moist to wet, dense to very dense, poorly graded fine sand with silt, silty fine sand, and sandy silt. Colma Formation soils at the four sites extended to the total depth of exploration (approximately 30 feet). Measured total unit weight for the Colma Formation soils at the four subject sites ranged from 113 to 129 pcf, with a moisture content ranging from 7 to 17 percent.

GROUNDWATER

Groundwater was not encountered during drilling of our exploratory borings GB-10A, -18, -19 and -22A to the total depths ranging from 30 to 30.5 feet. At GB-41-4, groundwater was encountered during drilling on December 16, 2008 at a depth of 27 feet. A summary of our observed groundwater levels is presented in Table 3 – Observed Groundwater Levels. Seasonal variations are expected to cause fluctuations in groundwater levels.

TABLE 3 – OBSERVED GROUNDWATER LEVELS

Boring	Date of Observation	Depth to Groundwater (feet)
GB-10A	12/15/2008	NE
GB-18	12/15/2008	NE
GB-19	12/15/2008	NE
GB-22A	12/16/2008	NE
GB-41-4	12/16/2008	27

NE = Not encountered.

CONCLUSIONS AND RECOMMENDATIONS

1.0 GENERAL

The following sections provide our conclusions and recommendations for evaluation and design of proposed station buildings at the five subject well sites of CUP-10A, -18, -19, -22A and -41-4. According to the Conceptual Engineering Report (MWH, 2008), station buildings at well sites CUP-10A, -18, -19 and -22A house a well and chemical treatment facilities. The station building at well site CUP-41-4 houses a well and filtration facilities. Based on our findings from our geotechnical field investigation, the CUP-10A and -18 sites are underlain by artificial fill and Colma Formation. Artificial fill at the CUP-22A site is underlain by natural levee deposits. At the CUP-19 and -41-4 sites, an intermediate stratum of natural levee deposits is interbedded between artificial fill and Colma Formation.

We consider the proposed improvements to be geotechnically feasible, provided that our geotechnical recommendations are incorporated into design and construction documents.

2.0 SEISMIC DESIGN CONSIDERATIONS

2.1 **General.** The main seismic hazards at the site are expected to be strong ground shaking and dynamic settlement within isolated zones of loose fill and natural levee deposits. Our seismic design considerations, including fault rupture, ground shaking, liquefaction and dynamic settlement, inundation by tsunamis, seismically-induced landslides, and seismic design with respect to the 2006 International Building Code (which the 2007 California Building Code has adopted) are provided in the following sections.

2.2 **Fault Rupture.** No active or potentially active faults are known to cross the subject sites. Consequently, the hazard posed by ground rupture due to fault offset is considered to be negligible.

2.3 **Ground Shaking**. Strong ground shaking will occur at the site as a result of a moderate to large earthquake occurring on one of the active regional faults. The San Andreas fault is closest to the subject sites (1.6 miles for CUP-10A, -18, -19 and -22A sites; and 2.1 miles for CUP-41-4 site), and therefore has the greatest capability of causing strong ground motions.

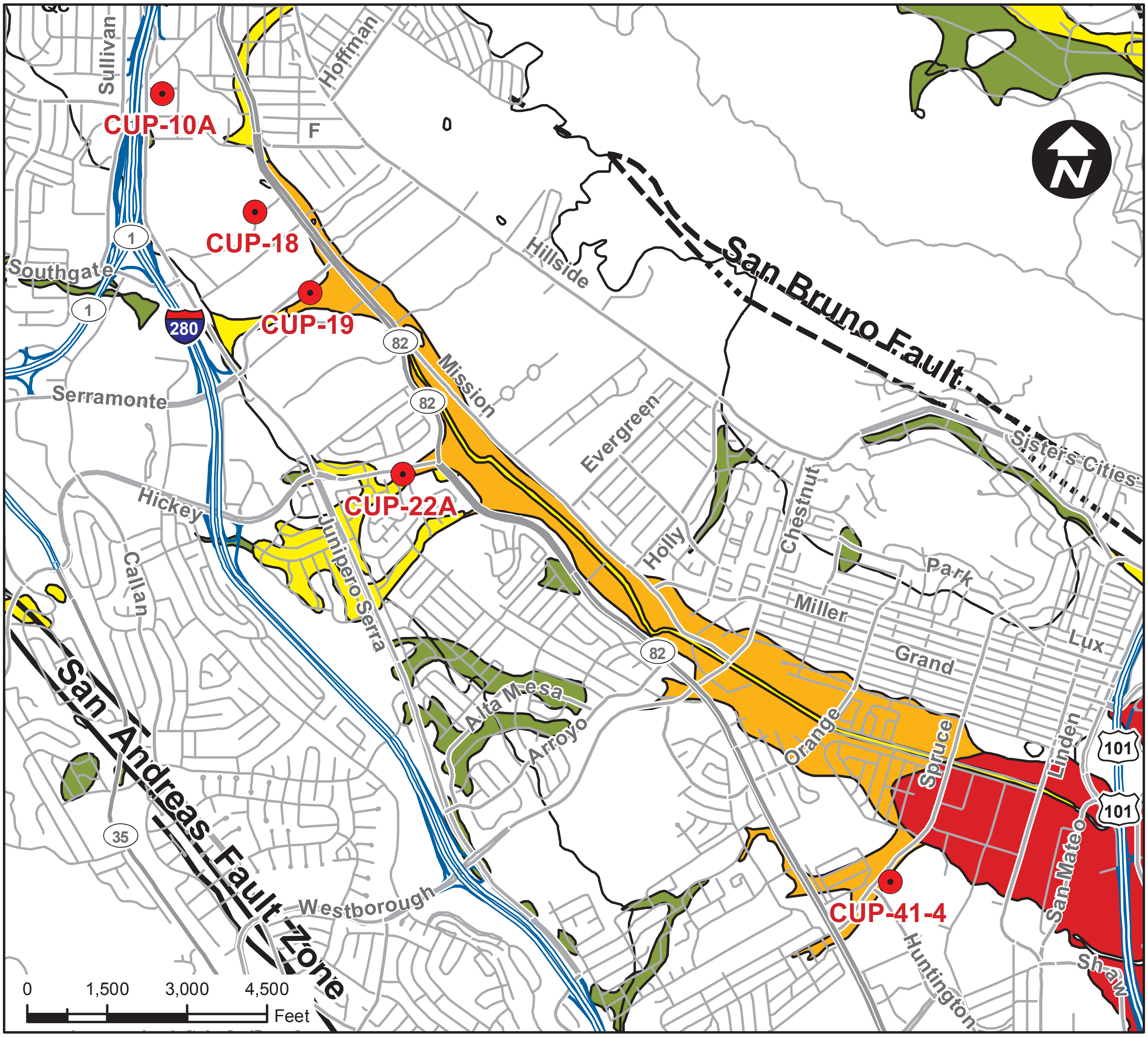
The California Geological Survey (CGS, formerly known as California Division of Mines and Geology) and United States Geological Survey (USGS) completed probabilistic seismic hazard maps in 1996 (Petersen et al., 1996), and subsequently updated fault parameters and revised the maps in 2002 (Cao, et al., 2003). USGS provides a web-based program to evaluate the USGS Probabilistic Uniform Hazard Response Spectra (http://earthquake.usgs.gov/research/hazmaps/design). Based on this data, the PGA at the site is estimated to be 0.71g for an earthquake having a 10 percent probability of exceedance in 50 years.

2.4 Liquefaction and Dynamic Settlement. Liquefaction is a phenomenon wherein a temporary, partial loss of shear strength occurs in a soil due to increases in pore pressure that result from cyclic loading during earthquakes. Saturated, loose to medium dense sands and silty sands are most susceptible to liquefaction. Consequences of liquefaction can include ground settlements, foundation failure, sand boils, and lateral spreading. Dynamic settlement is the densification of saturated and unsaturated soils during strong ground shaking. All soil types are prone to dynamic settlement, though loose, sand and silty sand are most susceptible.

The liquefaction susceptibility, as mapped by Witter et al. (2006), is illustrated on Figure 4 – Liquefaction Susceptibility Map. As can be seen from the figure, well sites at CUP-10A and -18 lie within a zone mapped as having a very low liquefaction susceptibility. The mapped liquefaction susceptibility at sites CUP-10 and -41-4 are moderate, and site CUP-22A lies within a zone mapped between moderate and high liquefaction susceptibility. Because of the regional focus of the liquefaction susceptibility mapping, the data only generally correlates with areas of known liquefaction hazard. The site-specific data from the borings is considered to be more indicative of liquefaction and dynamic settlement hazard. The following paragraphs further describe this hazard based on our subsurface investigation and laboratory testing program.

Due to the absence of groundwater within the 30 feet of total exploration depth for each exploratory boring at the CUP-10A, -18, -19 and -22A sites, and the generally dense nature of the Colma Formation (including the clayey nature of the natural levee deposits at the CUP-22A site) below this depth, liquefaction is not considered to be a significant consideration. Despite the observation of groundwater at a depth of 27 feet at the CUP-41-4 site, liquefaction is also not considered to be a significant consideration because of the dense nature of the Colma Formation encountered at this site. Pore pressure generation and liquefaction may occur in isolated pockets of looser material within the Colma Formation and natural levee deposits. The amount of surface settlement resulting from liquefaction is considered to be negligible at the five subject sites.

FIGURE 4
LIQUEFACTION SUSCEPTIBILITY MAP



Source: Witter, R.C., et. al.., 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California: U.S. Geological Survey Open-File Report 06-1037

LEGEND

CUP-22A Conjuntive Use Project (CUP) Sites

Liquefaction Susceptibility

- Very Low
- Low
- Moderate
- High
- Very High

The amount of dynamic settlement for each site has been evaluated based on an anticipated earthquake event having a 10 percent probability of exceedance in 50 years. Dynamic settlement resulting from strong ground shaking at CUP-10A is estimated at 2 inches due to the loose nature of the artificial fill. At CUP-18, dynamic settlement is estimated at ¼ inch, and is not considered to be significant due to the presence of relatively dense Colma Formation beneath a relatively thin stratum of artificial fill. Dynamic settlement at CUP-19 is estimated at 2 inches, mostly due to a relatively loose layer of poorly graded sand near the upper stratum of natural levee deposits. As a result of a relatively loose layer of silty fine sand within the natural levee deposits, dynamic settlement is estimated at ½ inch for CUP-22A. Dynamic settlement resulting at CUP-41-4 is estimated at 4 inches, and is considered relatively significant due to a loose layer of silty fine sand that spans the upper 6 feet of the natural levee deposits. The hazard posed by dynamic settlement is therefore considered to be low at CUP-18 and -22A, and moderately high at CUP-10A, CUP-19 and -41-4.

2.5 **Inundation by Tsunamis.** Tsunamis are long period waves usually caused by underwater seismic disturbances, volcanic eruptions, or submerged landslides. The disturbance can occur thousands of miles from the San Francisco area, and generate a tsunami wave that affects the site. As tsunami waves approach the coast, they may increase in height to tens of feet.

Flooding due to tsunami is unlikely to occur at CUP-10A, -18, -19 and -22A due to their relatively high ground elevations and distance from the open Northern California coastline. Although CUP-41-4 is located on relatively low lying terrain estimated on the order 25 to 30 feet above Mean Sea Level (MSL), the potential of flooding during a tsunami is unlikely because of the distance to San Francisco Bay.

2.6 **Seismically-Induced Landslides.** Based on the flat topography surrounding the sites of CUP-10A, -22A and -41-4, seismically-induced landslide hazards do not exist at these sites. An elevated automobile dealership parking lot to the west of CUP-19 is not likely to pose seismically-induce landslide hazards because of an existing concrete retaining structure and 30 to 40 feet of setback distance between the retaining wall and proposed station building. At CUP-18 which is located at the foot of a mildly sloping terrain (on the order of 7:1 horizontal to vertical side slope ratio), seismically-induced landslide hazards are considered not likely because of the dense nature of the subsurface soils and absence of shallow groundwater.

2.7 **Seismic Design Parameters.** The proposed improvements may be designed in accordance with the International Building Code Static Force Procedure (ICC, 2006) using the seismic parameters as presented in Table 4 – 2006 International Building Code (IBC) Seismic Design Parameters in developing the site seismic response:

TABLE 4 – 2006 INTERNATIONAL BUILDING CODE SEISMIC DESIGN PARAMETERS

	Site CUP-10A	Site CUP-18	Site CUP-19	Site CUP-22A	Site CUP-41-4
Site Class	C	C	D	D	C
S_s [(1)] at 0.2-second	2.17	2.16	2.16	2.17	2.07
S_1 [(1)] at 1-second	1.22	1.21	1.21	1.22	1.13
Site Coefficient F_a	1.0	1.0	1.0	1.0	1.0
Site Coefficient F_v	1.3	1.3	1.5	1.5	1.3

[(1)] Maximum Considered Earthquake (MCE) Spectral Response Acceleration (in g).

3.0 GROUNDWATER

With the exception of exploratory boring GB-41-4, groundwater was not encountered in the remaining four 30-foot deep exploratory borings. At GB-41-4, groundwater was encountered during drilling at a depth of 27 feet below ground surface. The observation of groundwater at GB-41-4 is consistent with the 1½-mile proximity of the site from the San Francisco Bayshore coastline to the east, and the relatively flat, low lying topography (ground elevations on the order of 25 to 30 feet above mean sea level). It should be noted that groundwater levels are influenced by seasonal variations in precipitation, local irrigation, groundwater pumping and other factors, and are therefore, subject to variation. To account for seasonal variations, we recommend conservative design groundwater levels for structural design purposes as presented in Table 5 – Recommended Design Groundwater Levels. The actual depth to groundwater is expected to be considerably deeper.

Groundwater related design issues such as hydrostatic pressures on shoring elements (if implemented), excavation dewatering, and hydrostatic uplift pressures on the proposed buildings are not anticipated for excavations less than 20 feet below the ground surface at the relatively flat sites of CUP-10A, -19, -22A and -41-4. Due to a sloping terrain at CUP-18, the aforementioned groundwater related issues are not anticipated for excavations less than 15 feet below the ground surface. For excavations exceeding the mentioned depths, the contractor should anticipate groundwater inflow and the need for dewatering.

TABLE 5 – RECOMMENDED DESIGN GROUNDWATER LEVELS

Site Location	Recommended Design Groundwater Depth (feet)
CUP-10A	20
CUP-18	15
CUP-19	20
CUP-22A	20
GB-41-4	20

4.0 EARTHWORK

4.1 **General.** Given the earth materials on the project sites encountered during our exploration, the contractor should be able to carry out planned excavations using conventional heavy equipment.

Evaluation of the presence, or absence, and treatment of hazardous materials was not part of this study. If hazardous materials are encountered during excavation, proper handling and treatment during construction will depend on the contaminant type, concentration, and volatility of the contaminated materials.

General geotechnical considerations for site preparation, excavations, temporary shoring and bracing, engineered fill material, engineered fill placement and compaction, pipe bedding, and utility trench backfill are presented in the following sections.

4.2 **Site Preparation.** Site preparation will consist of demolition, excavation and removal of on-site materials such as pavement, concrete, abandoned utilities, and miscellaneous debris in preparation for the foundation excavations. Any creation of holes from the removal of such materials should be backfilled with engineered fill. Recommendations for engineered fill are provided in Sections 4.5 and 4.6. Also as part of site preparation, the location of active underground utilities should be determined and, if affected by construction activities, should be relocated or protected.

4.3 **Excavations.** We anticipate that excavations for the planned building improvements to extend up to only a few feet below existing ground elevation. Since CUP-18 is located near the foot of mildly sloping terrain, greater excavation may be necessary at this site.

Shallow excavations for the well station buildings will allow for unshored excavations with adequately sloped sidewalls. Vertically shored walls or braced excavations are anticipated where space constraints may not allow for open, sloped excavations. At a minimum, excavations should be constructed in accordance with the current California Occupational Safety and Health Administration (OSHA) regulations (Title 8, California Code of Regulations) pertaining to excavations. Temporary cut slopes are expected to be stable for configurations described in Title 8 for Type C soils and where unsupported should be cut back no steeper than 1 ½ horizontal to 1 vertical. All excavations should be closely monitored during construction to detect any evidence of instability.

Care should be taken when excavating near existing utilities and pipelines. Excavations can undermine support of adjacent existing pipelines and other subsurface structures. We recommend that some form of vertical shoring system be considered for excavated sidewalls that are adjacent to existing pipelines or other known buried adjacent structures.

As indicated in Section 2.4, loose fill soils at CUP-10A and -19 sites, and loose soils in the upper portion of natural levee deposits at CUP-19 and -41-4, may settle excessively during a seismic event, and may require mitigation if the estimated settlements exceed tolerable levels. Some of the near surface loose soils at the five subject sites will likely be removed during excavation for the proposed improvements. If any footings are founded above loose soils, overexcavation of loose soils and replacement with engineered fill may be required. For loose natural levee deposits encountered at depths of 8 to 12 feet at CUP-19, and 2 to 6 feet at CUP-41-4, removal of materials via conventional grading involving earth removal and replacement may not be practical; instead, remediation of loose materials at intermediate depths can be performed using densification improvement methods, as discussed in Section 6.3.

4.4 **Temporary Shoring and Bracing.** The type and design of the shoring will depend on the depth of excavation and excavation-bracing sequence. The shoring and bracing design and installation should be the responsibility of the construction contractor. As a general guideline, construction procedures, excavations, and design and construction of any temporary shoring should comply with the current OSHA Title 8 regulations pertaining to excavations. The shoring and bracing should accommodate surcharge loads that may be imposed by adjacent structures, traffic, or construction activities.

Possible shoring schemes include soldier pile and lagging and steel sheeting, both of which may include internal bracing struts to limit lateral deflections. Such braced and shored excavations will be subjected to lateral earth pressures. Recommended active, at-rest, and passive lateral earth pressures are provided in Section 5.

Horizontal and vertical movements of the ground are possible in the vicinity of the excavations. These movements can generally be reduced to acceptable levels by use of a properly designed and constructed shoring system. Measures should be taken to prevent the loss of sand through the gaps in the shoring or lagging.

4.5 **Engineered Fill Material.** Material for engineered fill should be inorganic, well graded, free of rocks or clods greater than 4 inches in greatest dimension or any other deleterious materials, and have a low potential for expansion. The material should have a liquid limit less than 35, a plasticity index less than 15 and no more than 25 percent passing the No. 200 sieve. Existing on-site soil may be re-used as engineered fill provided it meets the above criteria.

4.6 **Engineered Fill Placement and Compaction.** Engineered fill should be placed in layers no greater than 8 inches in uncompacted thickness, conditioned with water or allowed to dry to achieve a moisture content near optimum, then mechanically compacted to at least 90 percent relative compaction based on ASTM D1557. All engineered fill placed to support footings and the upper 6 inches of engineered fill supporting slabs-on-grade should be mechanically compacted to at least 95 percent relative compaction as determined by ASTM D1557. All compaction should be performed using mechanical compaction means; flooding or jetting should not be used as a means to achieve compaction. The ASTM D1557 laboratory compaction tests should be performed at the time of construction to provide a proper basis for compaction control.

4.7 **Pipe Bedding for Small Diameter Pipes.** Pipe bedding should consist of well-graded sand or a sand-gravel mixture. Maximum gravel size should be ½ inch and the bedding material should have less than 12 percent passing the No. 200 sieve. Uniformly graded material such as pea gravel should not be used as pipe bedding material. Pipe bedding should have a minimum thickness of 6 inches beneath the pipe and 6 inches above the pipe. If soft or otherwise unsuitable soils are exposed in the bottom of the trench excavation, the necessity of over-excavation should be evaluated by the project geotechnical engineer. All pipe bedding should be placed to achieve uniform contact with the pipe and a minimum relative compaction of 90 percent per ASTM D1557.

4.8 **Utility Trench / Pipe Backfill.** Utility and pipe trenches may be backfilled above the pipe zone with excavated on-site soils, provided they meet the gradation requirements of engineered fill. The backfill material should be placed in layers no greater than 8 inches in uncompacted thickness, moisture conditioned or allowed to dry to achieve a moisture content near optimum, then mechanically compacted to at least 90 percent relative compaction based on ASTM D1557. The upper 2 feet should be compacted to at least 95 percent relative compaction in areas where structural or traffic loads are anticipated.

5.0 LATERAL EARTH PRESSURES

5.1 **Active Earth Pressure.** Active earth pressures are imposed by the soil on walls that are unrestrained so that the top of the wall is free to translate or rotate at least 0.004H, where H is the height of the wall. The active earth pressure may be calculated using a design equivalent fluid pressure (EFP) for each of the subject sites as indicated in Table 6.1 – Active Earth Pressures.

TABLE 6.1 – ACTIVE EARTH PRESSURES

Site Location	CUP-10A	CUP-18	CUP-19	CUP-22A	CUP-41-4
Active EFP [(1)] (pcf)	30	30	35	35	35

1. EFP assumes that excavations do not extend below the groundwater table.

5.2 **At-Rest Earth Pressure.** At-rest pressures should be used for design of walls that are restrained such that the deflections required to develop active earth pressures cannot occur or are undesirable. The at-rest earth pressures may be calculated using a design EFP for each of the subject sites as indicated in Table 6.2 – At-Rest Earth Pressures.

TABLE 6.2 – AT-REST EARTH PRESSURES

Site Location	CUP-10A	CUP-18	CUP-19	CUP-22A	CUP-41-4
At-Rest EFP [(1)] (pcf)	50	50	55	55	55

1. EFP assumes that excavations do not extend below the groundwater table.

5.3 **Seismic Earth Pressure.** In addition to the active and at-rest pressures, retaining walls should be designed to consider additional earth pressures due to earthquake loading. The increment in earth pressure due to seismic loading, for both restrained and unrestrained below-grade walls, may be calculated using an inverted triangular distribution with the pressure at the top of the wall equal to a design earth pressure (EP) of 30H, wherein H is the height of the wall in feet, and diminishes to zero at the base of the wall, as indicated in Table 6.3 – Seismic Earth Pressures.

TABLE 6.3 – SEISMIC EARTH PRESSURES

Site Location	CUP-10A	CUP-18	CUP-19	CUP-22A	CUP-41-4
Seismic EP [(1)] at Top of Wall (psf)	30 H [(2)]	30 H [(2)]	30 H [(2)]	30 H [(2)]	30 H [(2)]

1. EFP assumes that excavations do not extend below the groundwater table.
2. H is the height of the wall in feet, and diminishes to zero at the base of the wall.

5.4 **Passive Earth Pressure.** Lateral loads on structures can be resisted by passive pressures that develop against the sides of below-grade structures such as walls or footings. The passive pressure depends on the lateral displacement of the wall or footing. In accordance with FEMA 356 (FEMA, 2000), the ultimate passive pressure is mobilized at a displacement of approximately 6 percent of the wall height. The ultimate passive earth pressure may be calculated using a design EFP that corresponds to the ultimate EFP as long as the structure can be mobilized to such level of displacement and still does not exceed the allowable displacement of the structure. Oftentimes, the displacement to achieve ultimate passive earth pressures exceeds the allowable displacement of the structure. Consequently, a design EFP needs to be reduced when the allowable displacement of the structure is less than 6 percent of the wall height. For displacements of approximately 0.8 and 3 percent of the wall height, the design EFP may be reduced to 50 and 85 percent of the ultimate EFP. Passive pressures computed using these design EFPs may be combined with the base friction mobilized at the concrete-soil interface to resist lateral loading (see Section 5.5). The passive earth pressures may be computed using the following design EFPs as indicated in Table 6.4 – Passive Earth Pressures:

TABLE 6.4 – PASSIVE EARTH PRESSURES

Site Location	CUP-10A	CUP-18	CUP-19	CUP-22A	CUP-41-4
Passive Ultimate EFP (1) at 6% Wall Height Displacement (pcf)	390	390	425	425	360
Passive EFP (1) at 3% Wall Height Displacement (pcf)	330	330	360	360	305
Passive EFP (1) at 0.8% Wall Height Displacement (pcf)	195	195	215	215	180

1. EFP assumes that excavations do not extend below the groundwater table.

5.5 **Base Friction.** A coefficient of friction of 0.4 may be used for estimating the resistance due to base friction. The coefficient should be multiplied by the dead load only. The passive earth pressure and base friction mobilized at the concrete-subgrade interface may be combined to resist lateral loading.

6.0 FOUNDATIONS

6.1 **Subgrade Preparation.** Subgrades to new shallow foundations for the proposed structures should be prepared to provide a flat, relatively dry, and firm working surface. If any unsuitable materials, such as, soft clays or silts, soils containing organic material, debris or other deleterious materials are encountered at subgrade, they should be over-excavated and restored to grade with engineered fill in accordance with Sections 4.5 and 4.6. The fill soils encountered in our exploratory borings were suitable for support of the proposed improvements provided the upper 12 inches are scarified, moisture conditioned, and recompacted. We recommend that the upper 12 inches of subgrade be scarified, moisture conditioned to near optimum moisture content, and compacted in accordance with Sections 4.5 and 4.6. The subgrade should be free of loose debris and ponded water prior to placing reinforcing steel and concrete.

6.2 Shallow Foundation Alternatives. A shallow foundation system is suitable for support of the proposed improvements at the subject sites. Alternatives for shallow foundation systems include grade beams / shallow footings, mat foundations, and post-tensioned foundations.

Grade Beams / Shallow Footings: Based on the findings from our subsurface evaluation and laboratory testing, the ultimate bearing capacity of soils below new footings within the footprint of proposed buildings varies according the geotechnical characteristics of soils encountered at each subject site. We recommend an ultimate bearing capacity of 10,000 pounds per square foot (psf) for soils below new footings at the CUP-10A, -18 and -19 sites, 11,000 psf for CUP-22A, and 7,600 psf for CUP-41-4. Settlement of footings to attain these ultimate bearing capacities are expected to be on the order of about 2 inches, and could be significantly more as the ultimate bearing capacity is exceeded. To limit foundation settlements to less than ½ inch for dead and live loads and less than 1 inch for total loads including wind and seismic, the allowable bearing capacities provided in Table 7 – Allowable Bearing Capacities of Grade Beams and Shallow Footings may be used.

TABLE 7 – ALLOWABLE BEARING CAPACITIES OF GRADE BEAMS AND SHALLOW FOOTINGS

Sites	Load Combination	Allowable Bearing Capacity
CUP-10A CUP-18 CUP-19	Dead Load Dead + Live Load Dead + Live + Wind or Seismic Loads	3,300 psf 3,800 psf 5,000 psf
CUP-22A	Dead Load Dead + Live Load Dead + Live + Wind or Seismic Loads	3,600 psf 4,100 psf 5,400 psf
CUP-41-4	Dead Load Dead + Live Load Dead + Live + Wind or Seismic Loads	2,500 psf 3,000 psf 3,800 psf

Allowable bearing capacities recommended herein are applicable to newly constructed footings with widths of at least 18 inches and footing embedment of at least 24 inches below lowest adjacent grade.

A static modulus of subgrade reaction of 60 pounds per cubic inch (pci) may be used in order to develop soil springs below the foundation elements. For the lateral resistance of grade beams and footings, the geotechnical design parameters provided in the Lateral Earth Pressures section may be used.

As discussed in Section 2.4, dynamic settlements of up to approximately ½ inch may affect the CUP-18 and -22A sites during an earthquake event. The remaining three sites are more susceptible to significant dynamic settlements during an earthquake event. Larger dynamic settlements, on the order of 2 inches at CUP-10A and CUP-19, and 4 inches at CUP-41-4, are anticipated during an earthquake event if these sites are not mitigated. These dynamic settlements are in addition to the settlements estimated for the building loads described above. Long-term consolidation settlements are not likely due to the granular nature of much of the subsurface soils, and the stiffness and overconsolidation of clayey soils.

***Mat Foundations*:** Effects from differential dynamic settlements at the CUP-10A, -19 and -41-4 sites may be limited by supporting the structures at these sites on structurally rigid mat foundations. A mat foundation is a large concrete slab, designed by a structural engineer for specific use, to interface one or more columns or pieces of equipment with the foundation soil. It may encompass the entire foundation footprint or only a portion. The mat contact stresses are generally lower than other shallow foundation types due to distribution of stress over a larger area and stress compensation from excavated soil. Thickness and reinforcement of the mat foundation should be in accordance with the recommendations of a structural engineer. The appropriate allowable contact pressure(s) beneath the mat foundations will vary with their size, shape, and other factors. To limit foundation static settlements to less than ½ inch for dead and live loads and less than 1 inch for total loads including wind and seismic, the contact pressure beneath the mats should not exceed the allowable bearing capacities as recommended in Table 7 – Allowable Bearing Capacities for Grade Beams and Shallow Footings. Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the mat. A design coefficient of subgrade reaction, $\mathbf{K_{v1}}$, of 260 kips per cubic foot (kcf) in compacted fill soils may be used for evaluating such deflections at the subject sites. This value is based on a square foot area and should be adjusted for the planned mat size. The coefficient of subgrade reaction, $\mathbf{K_B}$, for a mat of a specific dimension may be evaluated using the following equation:

$$\mathbf{K_{B}, = K_{v1} [(B+1)/2B]^2 [(1+0.5(B/L)/1.5]}$$

where **B** is the width and **L** is the length of the foundation measured in feet.

Mat foundations bearing on fill may be designed using a coefficient of friction of 0.4 (total frictional resistance equals coefficient of friction times the dead load). The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed two-thirds of the total allowable resistance. For mat foundations, we recommend a passive resistance value of 300 psf per foot of depth, with a value not to exceed 3,000 psf. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

***Post-Tensioned Foundations*:** Effects from differential dynamic settlements at the CUP-10A, -19 and -41-4 sites may be limited through the application of post-tensioning in reinforcing, and hence, increasing the structural rigidity of grade beams / shallow footings. Thickness and reinforcement of a post-tensioned foundation should be in accordance with the recommendations of a structural engineer.

6.3 **Densification Improvements.** Dynamic settlements of loose granular soils at CUP-10A, -19, and -41-4 are anticipated during an earthquake event if these sites are not mitigated. An estimate of the amount of dynamic settlement and the depth to the zone of susceptible soils are provided in Table 8 - Densification Improvements to Mitigate Dynamic Settlements. If the structures cannot be designed to withstand this amount of settlement, densification may be an option to improve susceptible soils. Due to the existing pipelines at the sites, it may be difficult to improve the soils without causing settlement of the pipelines or otherwise damaging them. Once the site layouts are finalized and the existing pipelines accurately located, we can provide further recommendations regarding densification improvements.

TABLE 8 – DENSIFICATION IMPROVEMENTS TO MITIGATE DYNAMIC SETTLEMENTS

Site Location	CUP-10A	CUP-18	CUP-19	CUP-22A	CUP-41-4
Estimated Dynamic Settlement (inches)	2	¼	2	½	4
Improvement Depth of Loose Granular Soils (feet)	5±	-- (3)	12±	-- (3)	12±
Potential Method(s) of Improvement (1)	RAP RIC OR (2)	-- (3)	RAP RIC	-- (3)	RAP RIC

1. Densification improvement methods are denoted by RAP for Rammed Aggregate Piers and RIC for Rapid Impact Compaction.
2. For the CUP-10A site, conventional method of overexcavation and recompaction (OR) of loose granular soils is also a viable alternative to the above densification improvement methods.
3. Densification improvements are not necessary because the potential for dynamic settlement is low at CUP-18 and -22A.

The loose granular soils at CUP-10A can be mitigated by overexcavation and recompaction. Loose granular soils as encountered in the upper natural levee deposits at CUP-19 and -41-4 are susceptible to dynamic settlements on the order of 2 and 4 inches, respectively, if they are left unmitigated. Since such susceptible materials were encountered at intermediate depths within the upper 12 feet and 8 feet at GB-19 and -41-4, densification improvements and/or intermediate foundation systems may be preferable and more feasible than earth grading involving mass excavation and recompaction of loose materials, or a deep foundation system. Intermediate foundations such as Rammed Aggregate Piers (RAP) and Rapid Impact Compaction (RIC) may be suitable in mitigating the potential for post-earthquake dynamic settlements of loose materials at CUP-19 and -41-4.

RAP is constructed by either replacement (drilling a cavity) or displacement (driving a mandrel) to the depth of treatment, and ramming select aggregate in thin lifts to form compacted aggregate "bulbs" and densified materials surrounding the aggregate (Farrell, et al., 2004 and 2008; Majchrzak, et al., 2004). While the replacement process allows better quality control through visual inspection of drill spoils, the displacement approach eliminates spoils and is suitable for granular materials. Predrilled shafts are typically 24, 30, 33 and 36 inches in diameter. The ramming equipment typically consists of 18- to 27-ton hydraulic excavators equipped with 2,000- to 4,000-pound hydraulic break hammers and specially modified beveled tampers. The hydraulic hammer typically delivers 1 to 2 million ft-lbs of ramming energy per minute to the beveled tamper at 300 to 500 blows per minute. The ramming action increases the lateral stress in the surrounding soil and increases stiffness of the stabilized composite soil mass. The beveled tamper densifies and embeds the crushed aggregate laterally into the sidewalls of the shaft. Densification in both vertical and lateral (radial) directions enhances shear strength, bearing capacity and stiffness of the mitigated soil mass. RAP is typically effective for intermediate treatment depths up to 30 feet. When RAP aggregate is extracted from locally recycled concrete or any of the materials approved by the US Green Building Council (USGBC), points can be earned toward a Green Building certification in accordance with the Leadership in Energy and Environmental Design (LEED) rating system.

RIC is economically viable in recompacting loose materials at intermediate depths beyond practical/feasible reach of conventional mass grading. Similar to the ground improvement principles for RAP, RIC increases bearing capacity, controls dynamic settlement, and reduces potential for liquefaction by increasing density and strength of loose materials within the treatment depth (Kristiansen, 2004; TerraSystems, Inc., undated). RIC, which was originally developed by the British Sheet Piling, Limited in collaboration with the British Ministry of Defence, is an improvement on the process of Deep Dynamic Compaction (DDC) for many applications. Excavator mounted equipment provides controlled impact compaction of the earth by dropping a 7.5-ton weight approximately 4 feet onto a 5-foot diameter tamper at a rate of 40 to 60 times a minute. The energy transfer of RIC to the ground is relatively efficient because its tamper stays in contact with the ground during the impacting sequence. Densification of underlying loose materials is sustained from repeated dynamic impact energy imparted from the compaction tamper. Depth of impact is typically on the order of 10 feet to 20 feet. Treatment depth diminishes with increasing presence of fines in the subsurface materials. It is advantageous to perform RIC after stripping and limited removal of shallow overburden fill.

Quality assurance of the remediation program, which consists of post-treatment density evaluation, is an integral part of the acceptance testing program. Cone penetration testing (CPT) is typically used in providing continuous measurement of the soil density of the improved site.

6.4 **Floor Slabs.** Slabs-on-grade should be supported on a 12-inch thick mat of compacted, engineered fill. Material for engineered fill and compaction requirements are presented in Sections 4.5 and 4.6. For moisture-sensitive flooring, floor slabs resting on soil should be underlain, at a minimum, by a capillary break system. We recommend 6 inches of clean coarse sand or pea gravel. When floor dampness is a concern, such as at CUP-41-4 where elevated moisture content was observed in the near surface soils, floor slabs should be underlain by a vapor barrier and capillary break system. We recommend a system consisting of a 10-mil polyethylene (or equivalent) membrane placed over 6 inches of clean coarse sand or pea gravel. The exposed subgrade should be moistened just prior to the placement of the capillary break system. A sand layer above the moisture barrier to aid in concrete curing should be evaluated by the structural engineer. The slab underlayment including the capillary break can be taken as part of the 12-inch thick pad of compacted, engineered fill described above. Flooring and waterproofing consultants should be consulted for additional slab waterproofing recommendations.

7.0 CORROSION

Schiff Associates performed corrosivity laboratory tests on one soil sample for each of the five subject sites. Their laboratory results are included in Appendix A – Supporting Geotechnical Data. They performed the following tests:

- Resistivity (As-Received and Saturated),
- pH,
- Electrical Conductivity,
- Chemical Analyses of Cations (e.g. Calcium, Magnesium, Sodium)
- Chemical Analyses of Anions (e.g. Carbonate, Bicarbonate, Chloride, Sulfate)
- Chemical Analyses of Ammonium
- Chemical Analyses of Nitrate
- Chemical Analyses of Sulfide
- Oxidation-reduction (Redox) Potential

Electrical resistivities indicate soils are mildly corrosive to ferrous metals. The soil pH values were near neutral. The soluble salt contents of the samples were low, and on-site soils present a negligible sulfate exposure to concrete structures.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 **Existing Underground Utilities.** A number of underground water main pipelines pass beneath and in the vicinity of the proposed sites, including the Baden Merced, California Water Main, Daly City Water Main, San Andreas No. 2, San Andreas No. 3, San Bruno Water Main and Sunset Supply pipelines. Other existing subsurface lines include the SFPUC transmission lines, sanitary sewer and storm sewer lines. Some of these utilities were located and marked prior to our subsurface investigation so that we would not damage them during drilling.

The City may consider remarking these utilities prior to construction of the improvements so they remain visible during earthwork and construction of the subject improvements. Any excavations made adjacent to existing utilities should be backfilled with on-site or imported soil to at least 90 percent relative compaction as evaluated by ASTM D 1557.

8.2 **Vibration and Noise Control During Densification Improvements.** Peak soil particle velocities generated from vibrations during either RAP or RIC will vary with soil type, and will increase as the degree of compaction achieved increases. A test section using the proposed method of densification should be performed prior to production to establish a safe working distance from adjacent vibration-sensitive structures. For protection of existing sensitive underground water main pipelines near the proposed building footprint from ground-borne vibrations induced by either RAP or RIC, the use of open excavated cut-off trenches may be considered in attenuating densification-induced vibrations.

The level of air-borne noise generated by the RAP and RIC equipment in an open site, as well as a hearing protection zone, needs to be evaluated as part of the construction considerations.

8.3 **Surface Drainage.** Proper surface drainage is essential for satisfactory site performance. Positive drainage should be provided and maintained to direct surface water away from building foundations and other site improvements. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet or greater away from the foundations, flatwork, and tops of slopes. Runoff should then be directed by the use of swales or pipes into a collective drainage system. Surface water should not be allowed to pond adjacent to footings. We further recommend that the proposed structure be equipped with appropriate roof gutters and downspouts. Downspouts should discharge to a system of closed pipes that transport the collected water to a suitable discharge facility. We recommend that drought tolerant vegetation be used for site landscaping. Irrigation should be kept at levels just sufficient to maintain plant vigor.

9.0 CLOSURE

The conclusions and recommendations presented herein are professional opinions based on geotechnical and geologic data and the project as described. A review by this office of any foundation, excavation, grading plans and specifications, or other work product that relies on the content of this report, together with the opportunity to make supplemental recommendations is considered an integral part of this study. Should unanticipated conditions come to light during project development or should the project change from that described, we should be given the opportunity to review our recommendations.

The findings and professional opinions presented in this report are presented within the limits prescribed by the client, in accordance with generally accepted professional engineering and geologic practices. There is no other warranty, either express or implied, regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

Submitted by:
GEOTECHNICAL CONSULTANTS, INC.

Nick S. Ng, P.E., G.E.
Geotechnical Engineer, GE 2831

REGISTERED PROFESSIONAL ENGINEER
DERON VAN HOFF
No. 2575
Exp. 12/31/10
GEOTECHNICAL
STATE OF CALIFORNIA

4/2/09

Deron J. Van Hoff, P.E., G.E.
Geotechnical Engineer, GE 2575

REFERENCES

Blake, T.F., 2000, "EQFAULT, A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration Using Three-Dimensional California Faults as Earthquake Sources."

Brabb, E.E., Graymer, R.W., and Jones, D.L., 1998, Geology of the onshore part of San Mateo County, California: A Digital Database, U.S. Geological Survey Open-File Report 98-137.

California Geological Survey (CGS), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0, Bryant, W. A. (compiler): http://www.consrv.ca.gov/CGS/information/publications/QuaternaryFaults_ver2.htm.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, "The Revised 2002 California Probabilistic Seismic Hazard Maps," June.

Chaudhary & Associates, Incorporated, 2008a, Topographic Maps for Sites 10A, 18, 19 and 22A, South Westside Groundwater Conjunctive Use Project, September.

Chaudhary & Associates, Incorporated, 2008b, Topographic Maps for Site 41-4, South Westside Groundwater Conjunctive Use Project, March.

Coduto, D.P., 1998, Geotechincal Engineering: Principles and Practice, Prentice Hall.

Department of Conservation, Division of Mines and Geology, 1997, "Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California," March.

Farrell, T., Taylor, A., 2004, "Rammed Aggregate Pier Design and Construction in California – Performance, Constructability, and Economics," SEAOC 2004 Convention Proceedings.

Farrell, T., FitzPatrick, B., Kenney, W., 2008, "Uplift Testing of Rammed Aggregate Pier Systems," ASCE Geo Institute Geotechnical Earthquake Engineering and Soil Dynamics IV Proceedings, May.

Federal Emergency Management Agency (FEMA), 2000, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings", FEMA 356, November.

Frankel, A., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1996, National Seismic Hazard Maps: Documentation, June 1996, USGS Open File Report 96-532, July 19.

International Code Council (ICC), 2006, 2006 International Building Code.

Kristiansen, H., and Davise, M., 2004, "Ground Improvement Using Rapid Impact Compaction," 13th World Conference on Earthquake Engineering, Vancouver, British Columbia, Canada, Paper No. 496, August.

Majchrzak, M., Lew, Marshall, Sorensen, K., and Farrell, T., 2004, "Settlement of Shallow Foundations Constructed Over Reinforced Soil: Design Estimates vs. Measurements, "Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering, New York, NY, April.

Montgomery Watson Harza, Incorported (MWH), 2007, Draft Conceptual Engineering Report, CS-826 South Westside Groundwater Basin Conjunctive Use Program, Prepared for the San Francisco Public Utilities Commission, Illustrated Figures, November.

Montgomery Watson Harza, Incorported (MWH), 2008, Final Conceptual Engineering Report, CS-826 South Westside Groundwater Basin Conjunctive Use Project, Prepared for the San Francisco Public Utilities Commission, November.

Naval Facilities Engineering Command (NAVFAC), 1982a, "Soil Mechanics," Design Manual 7.1, May (Revalidated September, 1986).

Naval Facilities Engineering Command (NAVFAC), 1982b, "Foundations and Earth Structures," Design Manual 7.2, May (Revalidated September, 1986).

Petersen, M.D., Bryant, W.A., Cramer, C.H., Cao, T., Reichle, M., Frankel, A.D., Lienkaemper, J.J., McCrory, P.A., and Schwartz, D.P., 1996, Probabilistic Seismic Hazard Assessment for the State of California, CDMG Open-File Report 96-08. http://www.consrv.ca.gov/cgs/rghm/psha/ofr9608/Pages/Index.aspx

Pradel, D., 1998, "Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 124, No. 4, pp. 364 – 368, and Volume 124, No. 10, p. 1048.

San Francisco Public Utilities Commission (SFPUC) – Engineering Management Bureau, 2006, "General Seismic Requirements for Design of New Facilities and Upgrade of Existing Facilities," Engineering Standard General Seismic Design Requirements, EMB Document WSIP/CSP-001, August 15 (Updated with Addenda 1 and 2, August 21, 2007).

San Francisco Public Utilities Commission (SFPUC), 2008, 2007 Annual Groundwater Monitoring Report, Westside Basin, San Francisco and San Mateo Counties, California, April 25.

Seed, H.B. and Idriss, I.M., 1971, “Simplified Procedure for Evaluating Soil Liquefaction during Earthquakes,” Journal of Geotechnical Engineering Division, ASCE, Volume 97, No. 9, pp. 1249-1273.

Seed, H.B., Tokimatsu, K., Harder, L.F., and Chung, R.M., 1985, “Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations,” Journal of Geotechnical Engineering, ASCE, Volume 111, No. 12, pp. 1425 – 1445.

Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, J.D., Kayen, R.E., and Faris, A., 2003, “Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework,” 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Keynote Presentation, H.M.S. Queen Mary, Long Beach, California, 71 pp., April 30.

TerraSystems, Incorporated, Undated, “Rapid Impact Compaction, Another Form of Dynamic Compaction?,” Terra Notes, A Ground Improvement Update from TerraSystems, Lovettsville, Virginia.

Terzaghi, K., Peck, R.B., and Mesri, G., 1996, Soil Mechanics in Engineering Practice, Third Edition, Wiley-Interscience.

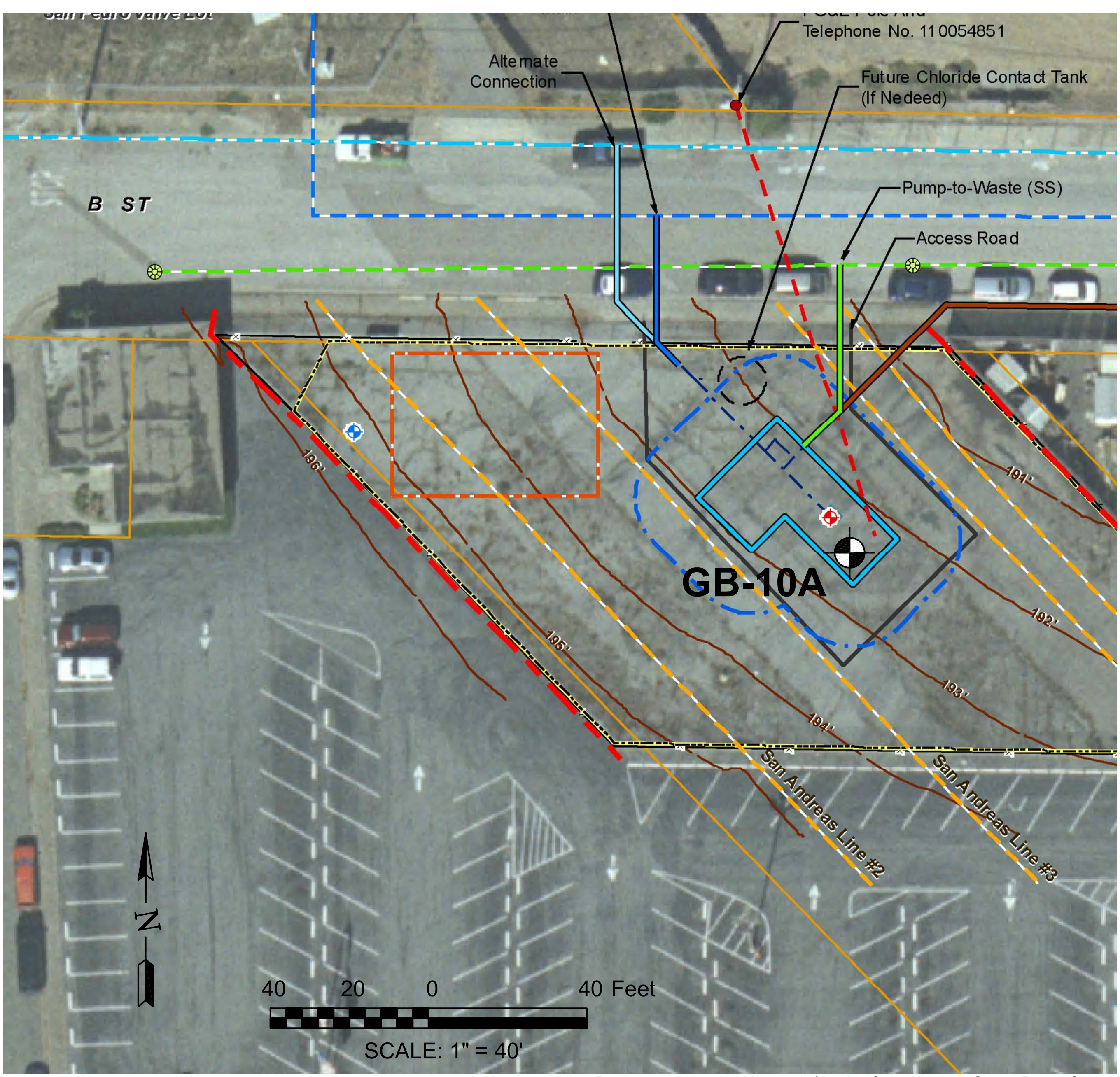
Tokimatsu, K. and Seed, H.B., 1987, “Evaluation of Settlements in Sands Due to Earthquake Shaking,” American Society of Civil Engineers, Journal of Geotechnical Engineering, Vol. 118.

United States Geological Survey (USGS), 1993, San Francisco South Quadrangle, California, 7.5-Minute Series (Topographic), Scale 1:24,000.

United States Geological Survey (USGS), 2008, Seismic Hazard Curves, Response Parameters and Design Parameters: Earthquake Ground Motion Parameters, A Java-based Application, Version 5.0.9., http://earthquake.usgs.gov/research/hazmaps/design, October 6.

Witter, R.C., Knudsen, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., Randolph, C.E., Brooks, S.K. and Gans, K.D., 2006, “Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California,” U.S. Geological Survey OFR 2006-1037, Scale 1:200,000. http://pubs.usgs.gov/of/2006/1037/.

Working Group on California Earthquake Probabilities (WGCEP), 2003, “Earthquake Probabilities in the San Francisco Bay Region: 2002–2031,” USGS Open-File Report 03-214. http://pubs.usgs.gov/of/2003/of03-214/.



Base map source: Kennedy/Jenks Consultants, Serra Bowl, Colma, CUP-10A location map, dated 2/2/2009.

LEGEND

- Test Well
- Monitoring Well
- GB-10A Geotechnical Boring by Geotechnical Consultants, Inc. in December 2008.
- ConstructionArea
- StagingArea Boundary
- Construction Area-16ft Building Buffer
- BuildingOutline
- AccessRoad
- Proposed Connection Main
- Alternate Connection
- Pump-to-Waste (SS)
- Pump-to-Waste (SD)
- RightOfWay
- Fence
- Topography
- Parcels - San Mateo County
- Transmission Line - SFPUC
- PG&E Pole
- Underground Electrical
- Water - CalWater
- Water - DalyCity
- Stormdrain Catch Basin - DalyCity
- Stormdrain Manhole - DalyCity
- Stormdrain - DalyCity
- Sanitary Sewer Manhole - DalyCity
- Sanitary Sewer - DalyCity

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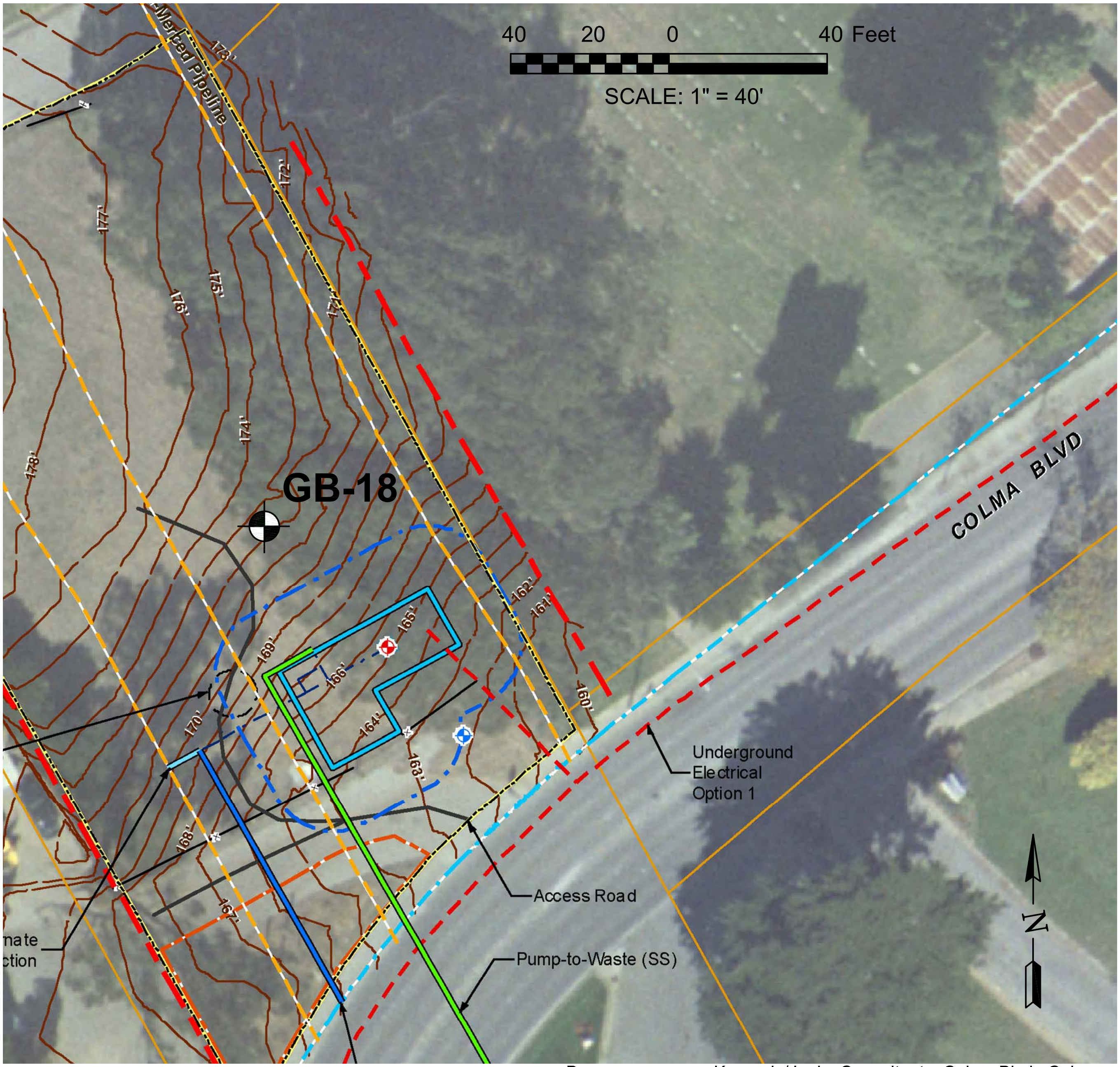
BORING LOCATION MAP FOR SITE CUP-10A	PLATE 1
SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
APRIL 2009	SF08034



Base map source: Kennedy/Jenks Consultants, Colma Blvd., Colma, CUP-18 location map, dated 2/2/2009.

LEGEND

- Test Well
- Monitoring Well
- GB-18 Geotechnical Boring by Geotechnical Consultants, Inc. in December 2008.
- ConstructionArea
- StagingArea Boundary
- Construction Area-16ft Building Buffer
- BuildingOutline
- AccessRoad
- Proposed Connection Main
- Alternate Connection
- Pump-to-Waste (SS)
- Pump-to-Waste (SD)
- RightOfWay
- Fence
- Topography
- Parcels - San Mateo County
- Transmission Line - SFPUC
- PG&E Pole
- Underground Electrical
- Water - CalWater
- Water - DalyCity
- Stormdrain Catch Basin - DalyCity
- Stormdrain Manhole - DalyCity
- Stormdrain - DalyCity
- Sanitary Sewer Manhole - DalyCity
- Sanitary Sewer - DalyCity

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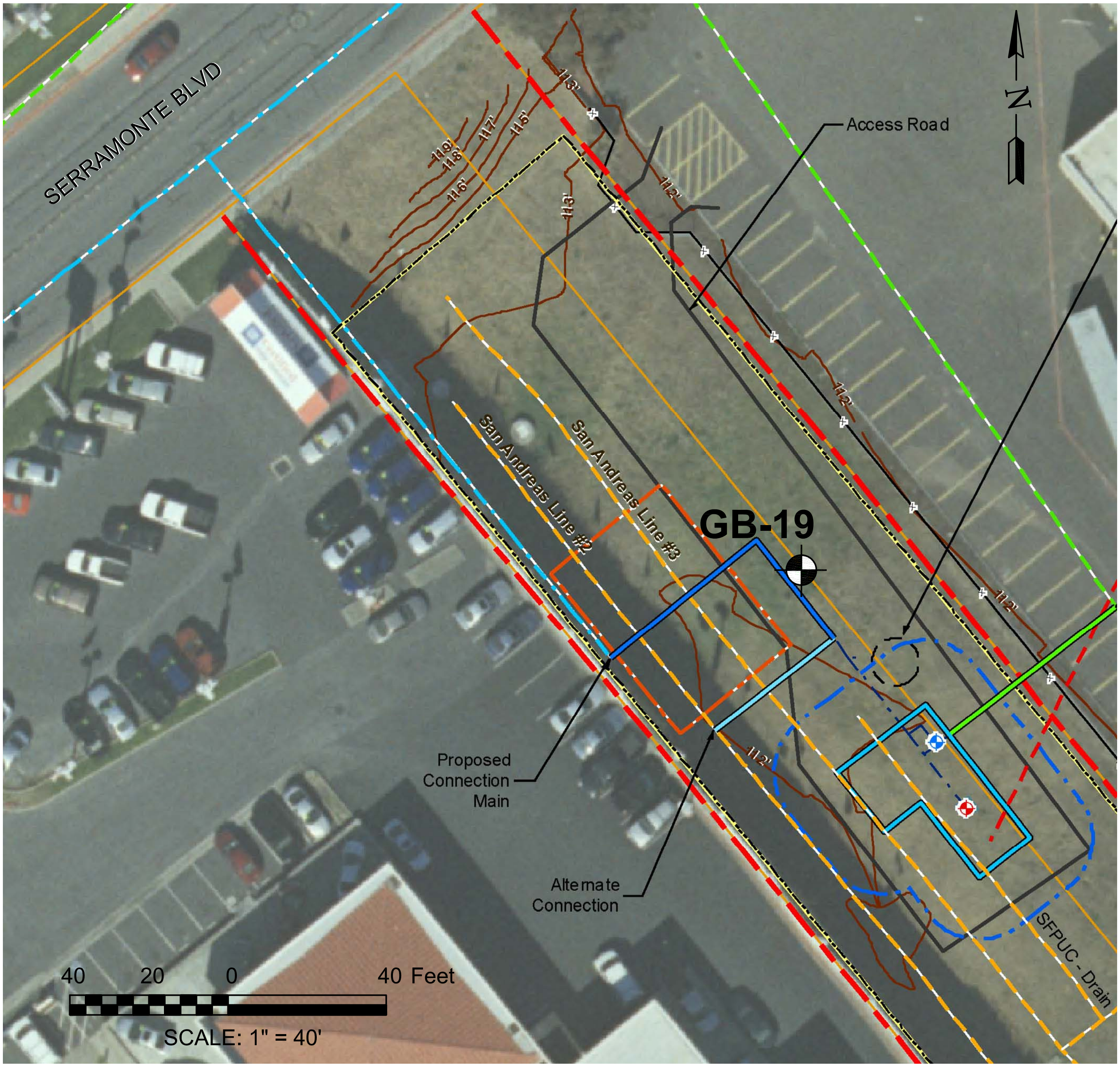
BORING LOCATION MAP FOR SITE CUP-18	PLATE 2
SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
APRIL 2009	SF08034



Base map source: Kennedy/Jenks Consultants, ROW AR Serramonte Blvd., Colma, CUP-19 location map, dated 2/2/2009.

LEGEND

- Test Well
- Monitoring Well
- GB-19 Geotechnical Boring by Geotechnical Consultants, Inc. in December 2008.
- ConstructionArea
- StagingArea Boundary
- Construction Area-16ft Building Buffer
- BuildingOutline
- AccessRoad
- Proposed Connection Main
- Alternate Connection
- Pump-to-Waste (SS)
- Pump-to-Waste (SD)
- RightOfWay
- Fence
- Topography
- Parcels - San Mateo County
- Transmission Line - SFPUC
- PG&E Transformer
- Underground Electrical
- Water - CalWater
- Water - DalyCity
- Stormdrain Catch Basin - DalyCity
- Stormdrain Manhole - DalyCity
- Stormdrain - DalyCity
- Sanitary Sewer Manhole - DalyCity
- Sanitary Sewer - DalyCity

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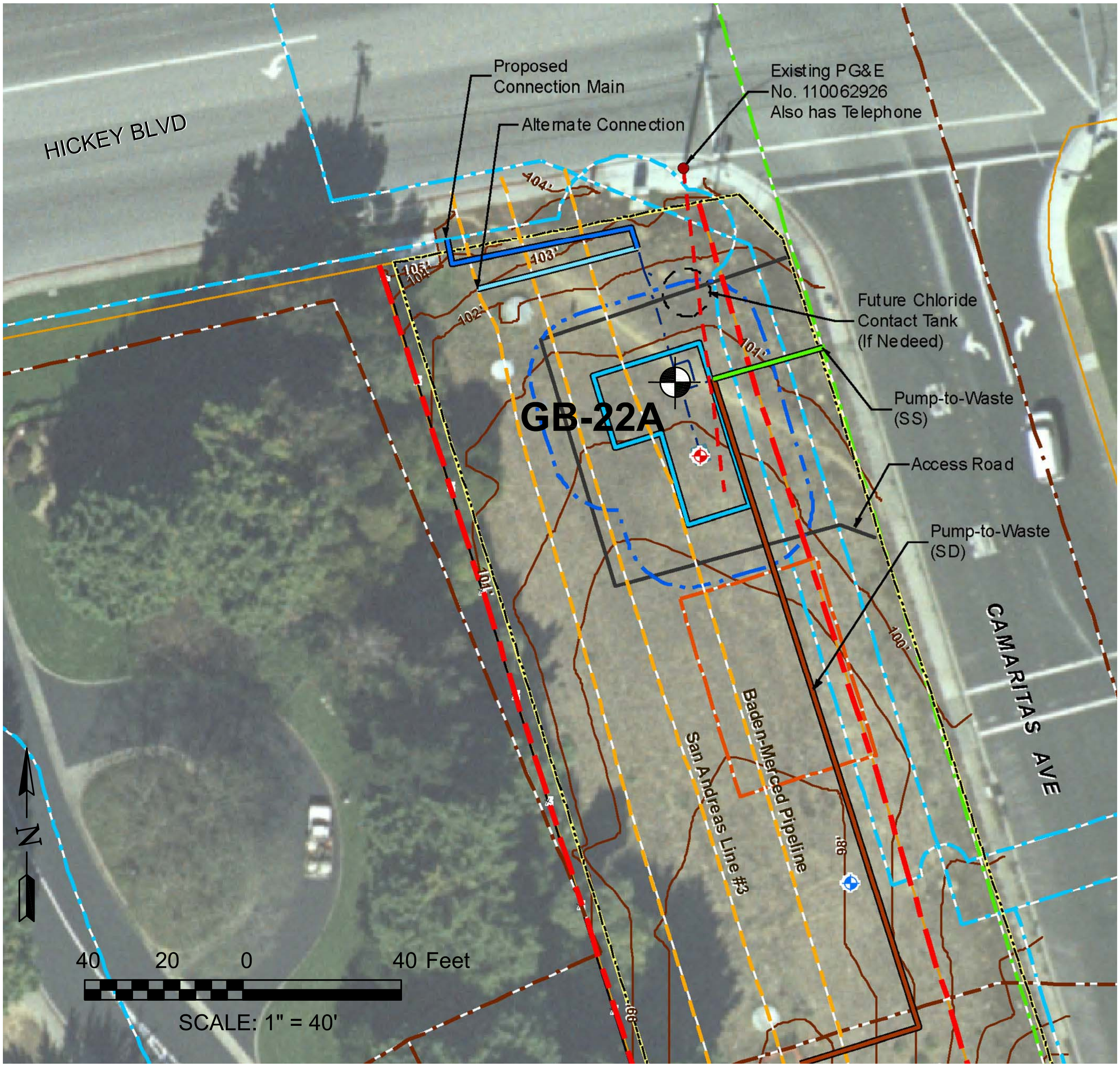
BORING LOCATION MAP FOR SITE CUP-19	PLATE 3
SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
APRIL 2009	SF08034

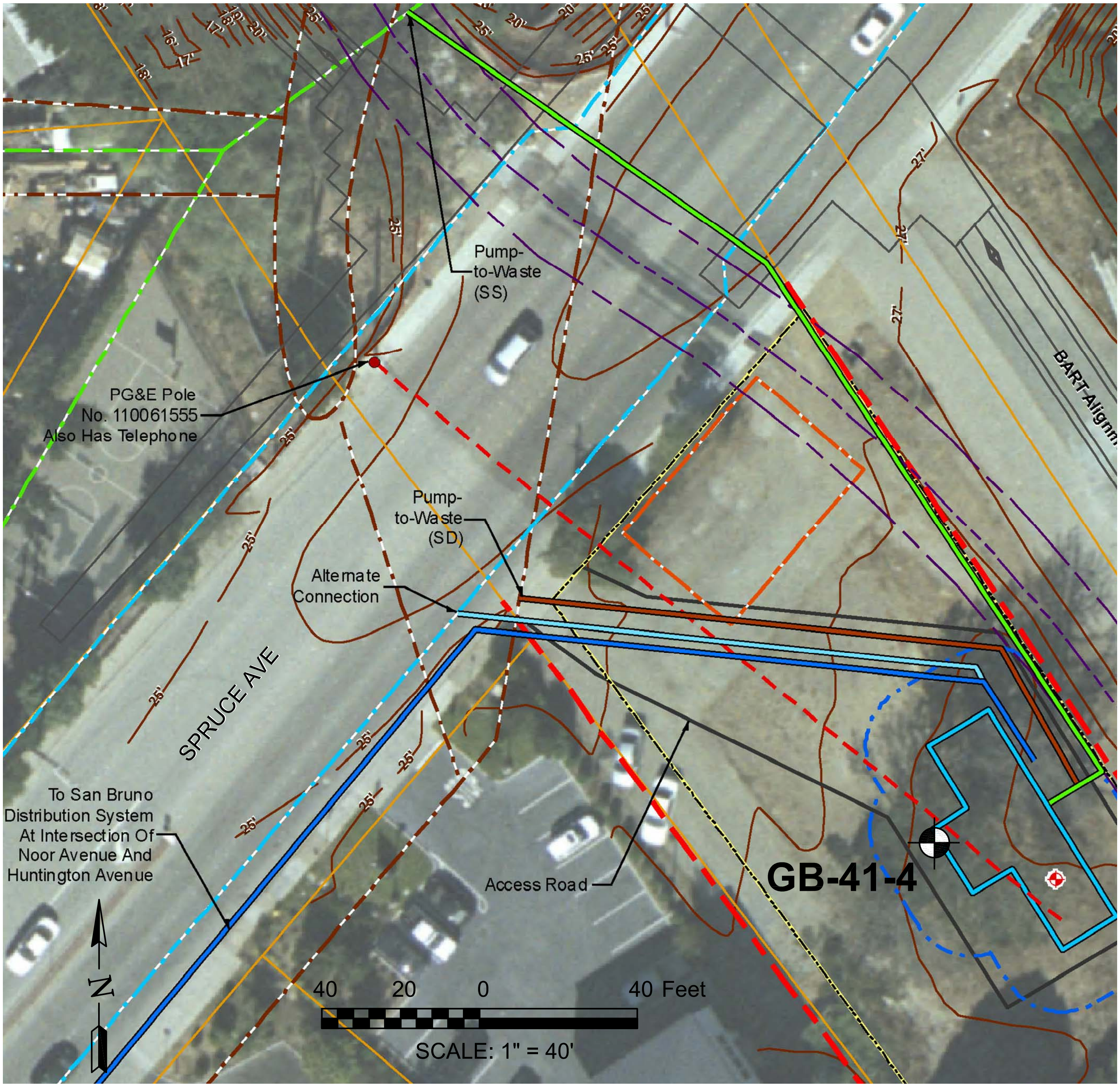


Base map source: Kennedy/Jenks Consultants, Hickey Blvd., South San Francisco, CUP-22A location map, dated 2/2/2009.

LEGEND

- Test Well
- Monitoring Well
- GB-22A Geotechnical Boring by Geotechnical Consultants, Inc. in December 2008.
- ConstructionArea
- StagingArea Boundary
- Construction Area-16ft Building Buffer
- BuildingOutline
- AccessRoad
- Proposed Connection Main
- Alternate Connection
- Pump-to-Waste (SS)
- Pump-to-Waste (SD)
- RightOfWay
- Fence
- Topography
- Parcels - San Mateo County
- Transmission Line - SFPUC
- PG&E Pole
- Underground Electrical
- Water - CalWater
- Stormdrain Manhole - South San Francisco
- Stormdrain - South San Francisco
- Sanitary Sewer - South San Francisco

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome St., Suite 402 San Francisco, CA 94111	**BORING LOCATION MAP FOR SITE CUP-22A**	**PLATE 4**
	SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
	APRIL 2009	**SF08034**



Base map source: Kennedy/Jenks Consultants, SSF Linear Park, San Bruno, CUP-41-4 location map, dated 2/2/2009.

LEGEND

- Test Well
- GB-41-4 Geotechnical Boring by Geotechnical Consultants, Inc. in December 2008.
- ConstructionArea
- StagingAreaBoundary
- Construction Area-16ft Building Buffer
- BuildingOutline
- AccessRoad
- Proposed Connection Main
- Alternate Connection
- Pump-to-Waste (SS)
- Pump-to-Waste (SD)
- RightOfWay
- Fence
- Topography
- Parcels - San Mateo County
- Transmission Line - SFPUC
- PG&E Pole
- Underground Electrical
- Water - CalWater
- Stormdrain Manhole - South San Francisco
- Stormdrain - South San Francisco
- Sanitary Sewer - South San Francisco

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome St., Suite 402 San Francisco, CA 94111	**BORING LOCATION MAP FOR SITE CUP-41-4**	**PLATE 5**
	SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
	APRIL 2009	**SF08034**

FINAL GEOTECHNICAL REPORT – CUP WELL LOCATIONS CUP-11A, CUP-23, CUP-36-1, CUP-44-1, AND CUP-M-1, SOUTH WESTSIDE BASIN GROUNDWATER STORAGE AND RECOVERY PROJECT, DECEMBER 2009

GEOTECHNICAL CONSULTANTS, INC.
Geotechnical Engineering • Geology • Hydrogeology

FINAL GEOTECHNICAL REPORT
CUP WELL LOCATIONS CUP-11A, CUP-23, CUP-36-1, CUP-44-1, AND CUP-M-1
SOUTH WESTSIDE BASIN GROUNDWATER STORAGE AND RECOVERY PROJECT
SAN MATEO COUNTY, CA

December 2009

Prepared for:

Kennedy/Jenks Consultants
303 Second Street, Suite 300 South
San Francisco, CA 94107

Owner:

San Francisco Public Utilities Commission

SF09020

500 Sansome Street, Suite 402 • San Francisco, CA 94111 • (415) 981-9950

TABLE OF CONTENTS

Page

TABLES

FIGURES

PLATES

APPENDIX A – SUPPORTING GEOTECHNICAL DATA

INTRODUCTION

This geotechnical report presents the findings, conclusions, and recommendations of our geotechnical study performed for proposed buildings at groundwater well stations, including chemical treatment and filtration facilities at five designated groundwater production and monitoring well sites located in the northern part of San Mateo County, California (Figure 1 – Site Location Map). Groundwater monitoring wells have recently been constructed as part of the South Westside Basin Groundwater Storage and Recovery Project (GSR), a project developed through the coordination of the San Francisco Public Utilities Commission (SFPUC) and three partner agencies (California Water Service Company [Cal Water], the City of Daly City, and the City of San Bruno). This geotechnical report is being prepared for Kennedy/Jenks Consultants as part of their design services contract with the SFPUC and represents Phase 2 of the GSR. GTC previously completed subsurface exploration, laboratory testing and analysis at five sites for Phase 1 (GTC, April 2009).

We anticipate that the proposed well station buildings will typically be constructed with concrete masonry units (CMU), although the material selection will depend on the surrounding structures. The preliminary building footprints are as shown in Plates 1 through 5, Boring Location Plans. Geotechnical recommendations for additional improvements such as new pipeline connections and upgrades, which may require additional geotechnical borings, were not part of our scope of work.

WORK PERFORMED

In accordance with our scope of work as documented in the Subcontract Agreement (Amendment No. 3) with Kennedy/Jenks Consultants, Incorporated (KJ) dated August 2009 and subsequent conversations with personnel from KJ, we have completed the scope of work described below:

1. **Exploratory Drilling.** Subsurface conditions were explored by means of drilling one hollow-stem auger boring at each of the five CUP sites designated as CUP-11A, CUP-23, CUP-36-1, CUP-44-1, and CUP-M-1. To maintain consistency with the site numbering, our borings have been accordingly labeled as GB-11A, -23, -36-1, -44-1 and –M-1 for the sites. Boring number, date of drilling, surface elevation and depth for each boring are summarized in Table 1 – Summary of Geotechnical Borings. The surface elevations of the borings were evaluated from topographic maps which were prepared by Chaudhary & Associates from their field surveys performed between March of 2008 and September of 2009. The surface elevations presented in this report are approximate. All elevations on Table 1, and referred to throughout this report (unless otherwise noted), are with respect to 1988 North American Vertical Datum (NAVD 88).

2.

TABLE 1 – SUMMARY OF GEOTECHNICAL BORINGS

Boring	Date Drilled	Approximate Surface Elevation (feet, NAVD 88)	Approximate Depth (feet)
GB-11A	9/28/2009	159.5	35
GB-23	9/25/2009	83.5	50
GB-36-1	9/25/2009	66.5	50
GB-44-1	10/19/2009	111.0	35
GB-M-1	9/28/2009	26.0	40

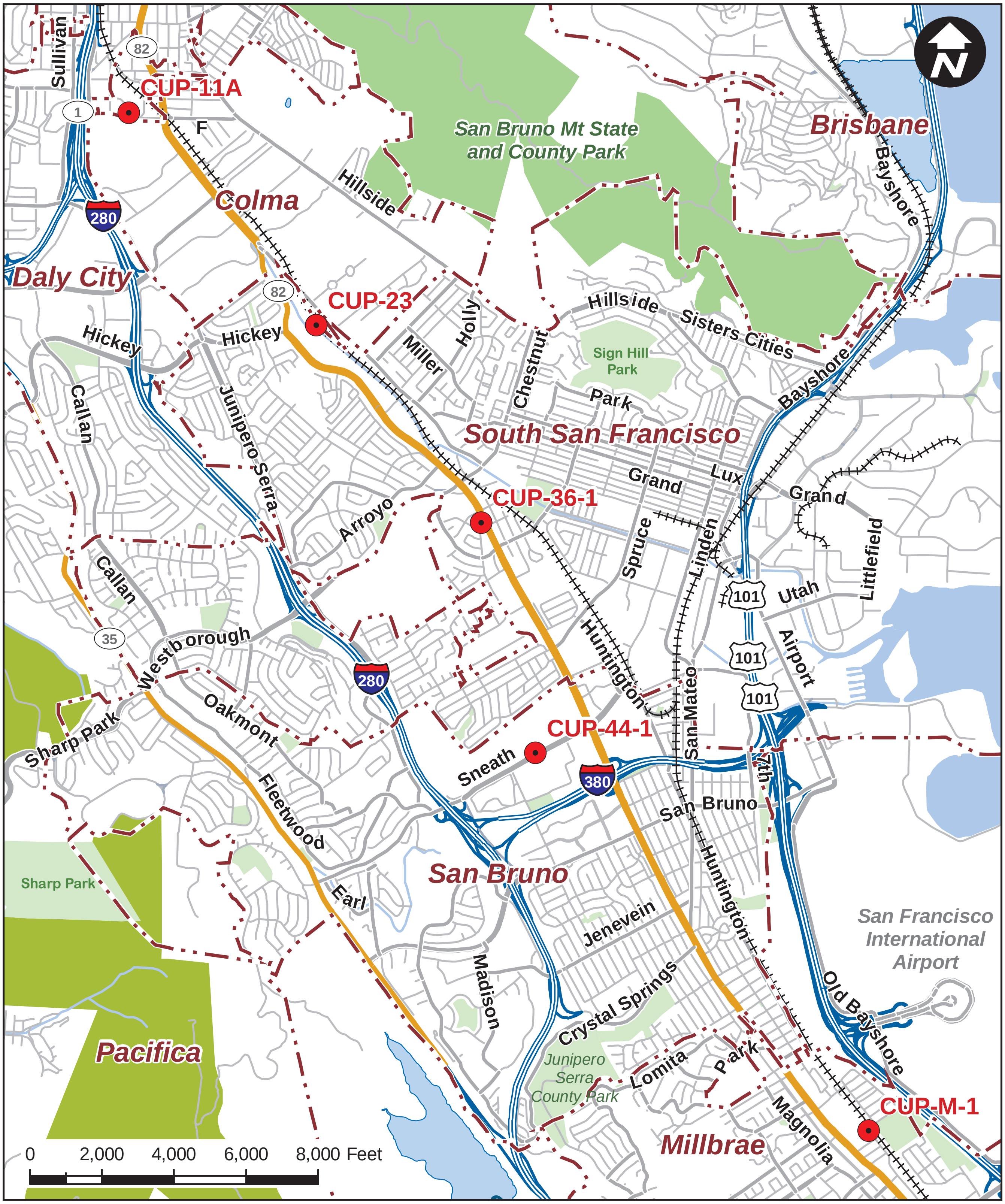
Soil samples were recovered using a split-spoon (Standard Penetration Test) sampler and relatively undisturbed 2 ½ inch diameter sleeve samples using a split-barrel sampler. We visually classified the soil during drilling. Selected samples were transferred to a laboratory for testing. The boring locations are shown on Plates 1 through 5 – Boring Location Plans. Boring logs are presented in Appendix A – Supporting Geotechnical Data as Plates A-1.1 through A-1.5. Upon completion of geotechnical exploration, the drill cuttings were collected in steel drums for analytical testing and appropriate disposal.

3. **Laboratory Testing.** Laboratory testing included moisture, density, grain size analysis, Atterberg limits and corrosion tests on selected soil samples to measure pertinent index and engineering properties. The laboratory test results are presented on the figures in Appendix A, and on the boring logs on Plates A-1.1 through -1.5.

4. **Engineering Analysis.** We analyzed subsurface conditions and laboratory test results, and reviewed regional and local geology and seismicity. Additionally, we analyzed the following geotechnical parameters:

- Seismic hazards evaluation including strong ground shaking, liquefaction, seismic and dynamic settlements, and seismically-induced landslides;
- Seismic design parameters in accordance with the 2006 International Building Code;
- Bearing capacity (allowable and ultimate) and modulus of subgrade reaction (vertical soil springs) for shallow footings and grade beams, and mat foundations; and
- Lateral earth pressures (active, passive, at-rest, and seismic increment) and base friction coefficients for restrained and unrestrained walls and/or buried footings.

FIGURE 1
SITE LOCATION MAP



5. **Report.** We prepared this report presenting our geotechnical findings, conclusions, and recommendations for the proposed improvements at the five sites for the GSR Phase 2.

Our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

FINDINGS

SITE CONDITIONS

The five sites are located from the north portion (CUP-11A) of the South Westside Groundwater Basin to near the southern boundary (CUP-M-1) in San Mateo County, California. The ground surface along a line which roughly transects the five sites, and parallels El Camino Real, generally descends in a northwest-to-southeast direction from elevations of approximately 160 feet to 20 feet above mean sea level for a distance of approximately 8 miles. Plates will be finalized in the Final Geotechnical Report. All boring locations were cleared of existing underground utilities prior to exploration.

The northernmost site CUP-11A is located southwest of F Street and the Colma BART station in the town of South San Francisco (Figure 1). As indicated on Plate 1 – Boring Location Plan for GB-11A, the site is located on a gentle to moderate east-facing slope. Southwest of the site are the BART parking lots and to the northeast, F Street.

GB-23 is located east of the intersection between Hickey Boulevard and El Camino Real in South San Francisco (Figure 1). As indicated on Plate 2 – Boring Location Plan for GB-23, the site is located on fairly level ground. The site is bounded by the Costco parking lot to the south, a mobile home park to the northwest and the drainage channel abutting the BART underground alignment to the northeast.

GB-36-1 is located to the south of the intersection between El Camino Real and Southwood Drive in the Town of South San Francisco (Figure 1). The general layout of the proposed improvements on Plate 3 – Boring Location Plan for GB-36-1 shows the boring on a gradual northeast-facing slope. The site is near recently re-graded pipeline construction access and is surrounded to the northwest by a parking lot for a funeral home, to the east by a descending slope with vegetation adjacent to El Camino Real and to the south by relatively flat, graded grounds with temporary structures and equipment serving as facilities for this project.

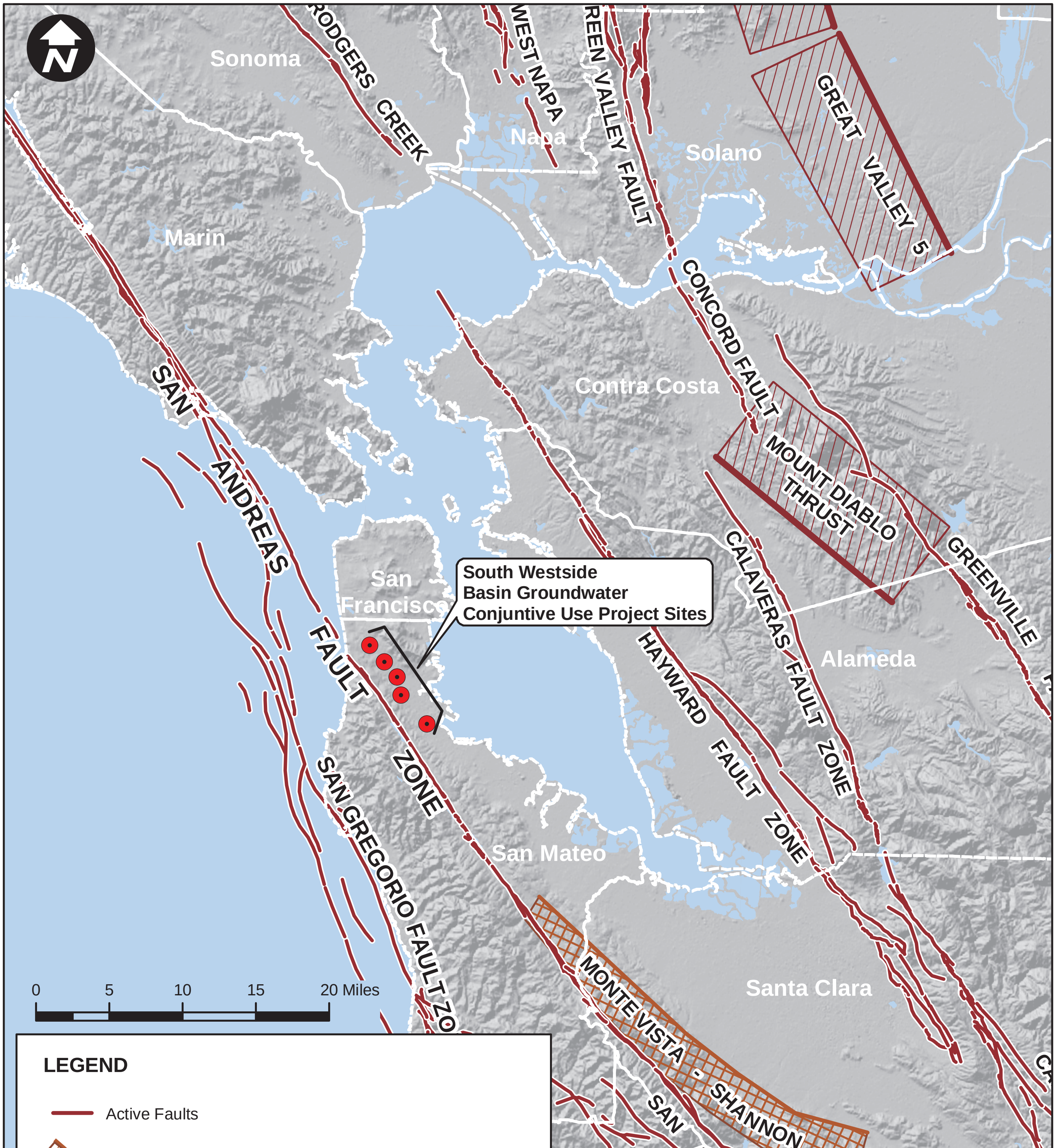
GB-44-1 is located to the south of the main building at the Golden Gate National Cemetery, just north of Sneath Lane in San Bruno (Figure 1). The general layout of the proposed improvements on Plate 4 – Boring Location Plan for GB-44-1 shows a generally level site with a slope some ways to the south, across Sneath Lane. The site is bounded to the south by a sidewalk abutting Sneath Lane and surrounded to the north, east and west by the Golden Gate Cemetery lawn and facilities.

The southernmost site of GB-M-1 is situated in the eastern corner of the parking lot at the Orchard Supply Hardware store at 900 El Camino Real in Millbrae (Figure 1). As shown on Plate 5 - Boring Location Plan for GB-M-1, this site is located in a flat asphalt-paved parking lot. The areas surrounding the site are also relatively flat. The site is surrounded to the northeast by the CalTrain tracks, to the southeast by a small lot containing a communications tower, to the northwest by the Orchard Supply Hardware storage yard, and to the southwest by the Orchard Supply Hardware loading dock and parking lot.

SEISMICITY

The San Francisco Bay Area contains several active faults that could cause strong ground shaking at the project site. Figure 2 – Regional Fault Map shows faults in the vicinity of the sites. The San Andreas Fault Zone – Peninsula Section is the nearest active fault and is located within 1.5 to 1.9 miles of the CUP-11A, CUP-23, CUP-36, CUP-44-1, and CUP-M-1 sites. The San Andreas Fault is a primary component in a complex system of right-lateral, strike-slip faults; including the San Andreas, San Gregorio-Seal Cove, Hayward, and Calaveras faults; collectively known as the San Andreas fault system. The San Andreas, Hayward, and Calaveras faults have produced historic earthquakes resulting in significant ground motion and movement. The San Andreas Fault is capable of producing an earthquake of an estimated maximum magnitude of 7.9M. This segment is estimated to have recurrence intervals on the order of 200 years. A summary of nearby faults is presented in Table 2 – Active and Potentially Active Faults.

FIGURE 2
REGIONAL ACTIVE FAULT MAP



LEGEND

Active Faults

Reverse Fault (rectangle represents projection of the fault plane to the surface)

Blind Thrust Faults (faults do not intersect the surface, mapped trace represents projection of upper edge of the fault to surface; rectangle represents projection of the fault plane to the surface)

Fault Data Sources: CGS 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0, Bryant, W. A. (compiler) and Cao, et. al., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, Appendix A - 2002 California Fault Parameters

TABLE 2 – ACTIVE AND POTENTIALLY ACTIVE FAULTS

Fault	Distance to Fault (miles)					Estimated Maximum Earthquake Magnitude [1]	Historic Earthquakes [2]	
	GB-11A	GB-23	GB-36-1	GB-44-1	GB-M-1		Year	Magnitude
San Andreas - 1906 rupture Section	1.6 [3]	1.8 [3]	1.9 [3]	1.5 [3]	1.7 [3]	7.9 [3]	1838 1898 1906 1989	6.8 6.2 8.1 7.1
San Andreas – Peninsula Section	1.6	1.8	1.9	1.5	1.7	7.1		
San Andreas – North Section	11.5	13.0	14.3	15.5	18.1	7.6		
San Gregorio-Seal Cove – North Section	5.6	6.2	6.6	6.5	7.5	7.3	N.A.	N.A.
Hayward- North Section	17.1	16.9	16.8	17.2	16.8	6.9	1868	6.8
Hayward – South Section	18.7	18.0	17.4	17.5	16.8	6.9		
Monte Vista-Shannon	20.7	19.2	17.9	16.7	14.1	6.8	N.A.	N.A.
Calaveras – North Section	26.7	26.2	25.8	26.0	25.4	6.8	1861 1955 1979 1984 2007	5.3 5.5 5.9 6.1 5.4
Calaveras – South Section	40.7	39.3	38.1	37.3	35.0	6.2		

(1) Maximum Moment Magnitude based on California Geological Survey (CGS) fault parameters as updated in 2002 (Cao, et al., 2003), or as suggested by the SFPUC's General Seismic Requirements (SFPUC, 2006).
(2) Historic earthquakes listed may have occurred on any one of the listed sections of the associated fault. N.A. – No significant historic earthquakes have occurred on this fault or fault section.
(3) The 1906 rupture event assumes rupture along the North Coast, Peninsula and Santa Cruz Mountains sections to San Juan Bautista. Maximum magnitude is based on the average 5 m displacement during the 1906 event (WGCEP, 2003; Petersen, et al., 1996).

GEOLOGY

The San Francisco Bay Area is located within the Coast Ranges Geomorphic Province of California. Past episodes of tectonism have folded and faulted the bedrock, creating the regional topography of northwest trending ridges and valleys that is characteristic of the Coast Ranges Geomorphic Province. The San Francisco Bay and vicinity occupy a structurally controlled basin within the province. Late Pleistocene and Holocene sediments (less than 1 million years old) were deposited in the basin as it subsided.

All five sites are located in areas mapped as Colma Formation (Brabb, et al., 1998; Bonilla, 1998). Other sedimentary deposits mapped in close proximity to these sites include stream channel deposits and Merced Formation. In addition, a layer of artificial fill was encountered at each site. The geology in the project vicinity is shown on Figure 3 – Regional Geologic Map. Based on a regional geologic study as compiled as a regional geologic cross section of the Westside Basin – Lake Merced (SFPUC, 2008), the Franciscan Complex bedrock is anticipated to be on the order of 600 to 700 feet below ground surface at the sites. Geologic maps (Brabb, et al., 1998) describe the geologic units at and near each boring as follows:

- **af:** Artificial fill – loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations and thicknesses which may exceed 30 m; some compacted and quite firm, but fill made before 1965 is nearly everywhere not compacted and consists of simply dumped materials.

- **Qhbm:** Bay mud (Holocene) – soft to stiff clay and silty clay underlying marshland and tidal flats (near Bayshore Freeway), contains few lenses of fine sand, silt, shells, and peat.

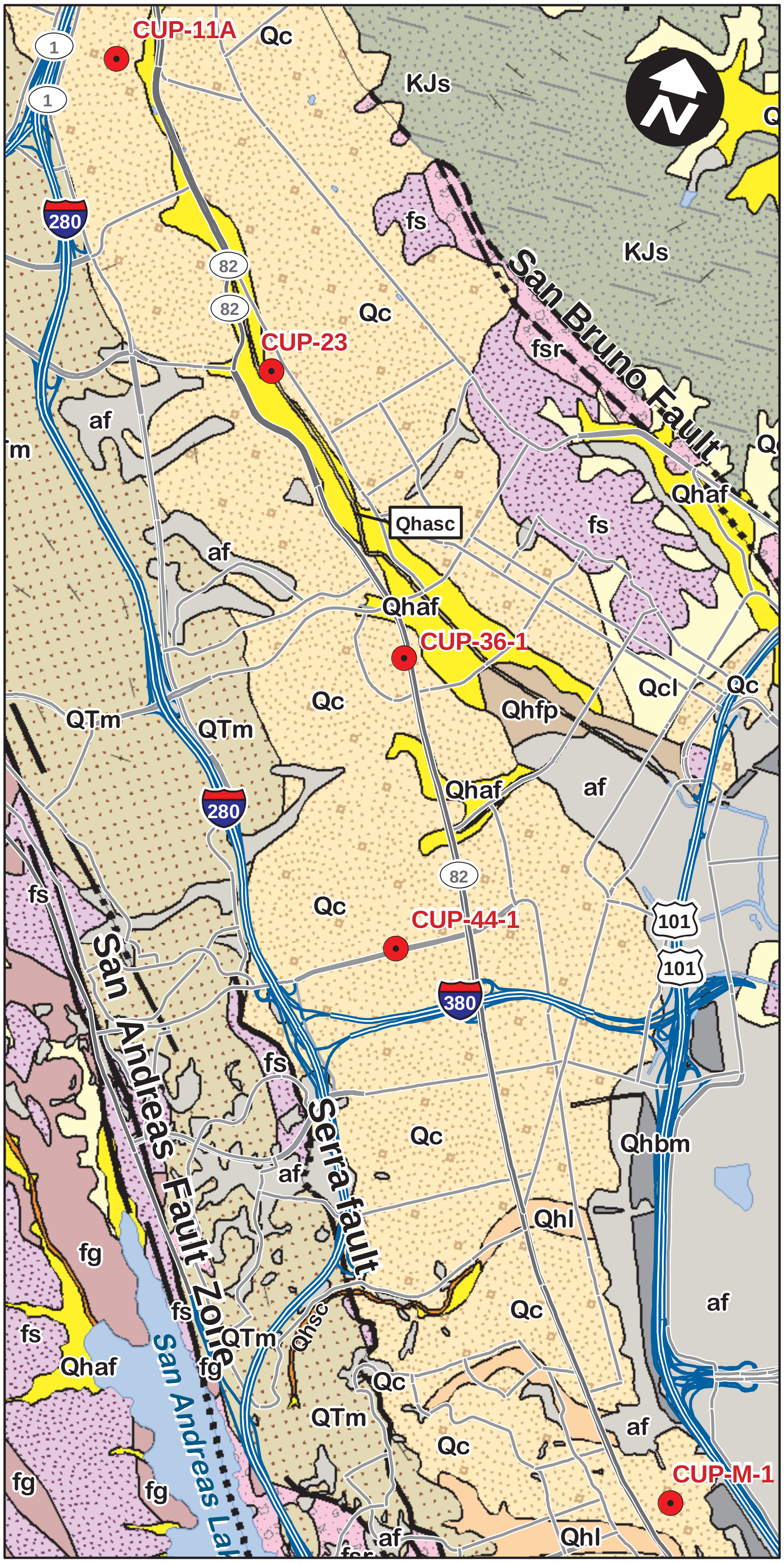
- **Qhl:** Natural levee deposits (Holocene) – loose, moderately to well-sorted sandy or clayey silt grading to sandy or silty clay deposits that border stream channels and slope away to flatter floodplains and basins.

- **Qhfp:** Floodplain deposits (Holocene) – dense sandy to silty clay, with local lenses of coarser material (silt, sand, and pebbles).

- **Qc:** Colma Formation (Pleistocene) – yellowish-gray, gray, yellowish-orange and red-brown, friable to loose, fine- to medium-grained arkosic sand with subordinate gravel, silt and clay; total thickness is typically unknown, but may up to 60 m.

- **QTm:** Merced Formation (lower Pleistocene and upper Pliocene) – medium gray to yellowish gray, yellowish orange, medium- to very fine-grained, poorly indurated to friable sandstone, siltstone, and claystone, with some conglomerate lenses and a few friable beds of white volcanic ash; sandstone is typically silty, clayey, or conglomeratic; fossiliferous conglomerate is well cemented.

FIGURE 3
REGIONAL GEOLOGIC MAP



Source: Brabb et. al., 1998, USGS OFR 98-137.

LEGEND

CUP-11A — Conjuntive Use Project (CUP) Sites

Geologic Units

Historic
- af — Artificial fill
- Qhasc — Artificial stream channels

Holocene
- Qhfp — Floodplain deposits
- Qhaf — Alluvial fan and fluvial deposits
- Qhsc — Stream channel deposits
- Qcl — Colluvium
- Qhbm — Bay mud
- Qhl — Natural levee deposits

Pleistocene
- Qc — Colma Formation

Pleistocene to Pliocene
- QTm — Merced Formation

Cretaceous to Jurassic
- KJs — Unnamed sandstone of San Bruno Mtn.
- fs — Franciscan sandstone
- fg — Franciscan greenstone
- fsr — Franciscan melange

Structual Features
- geologic contact
- fault, approx. located
- fault, certain
- fault, concealed

0 2,000 4,000 6,000 8,000 Feet

EARTH MATERIALS

The exploratory borings for this investigation (GB-11A, -23, -36-1, -44-1 and –M-1) encountered artificial fill which was underlain by poorly to moderately consolidated sandstone of the Colma Formation (Qc). The artificial fill represents disturbed soil and fill materials placed for site grading and pipeline trench backfill.

Artificial Fill. Artificial fill was encountered to depths of approximately 4 feet in borings GB-11A and GB-23 where the local topography is flat. Fill thickness measures 14.5 feet at GB-36-1 where trenching and construction of large diameter pipelines has disturbed the ground to greater depth. Fill at GB-44-1 was approximately 8.5 feet thick. Fill placed for leveling at GB-M-1 is 9 feet thick. The fill was mainly comprised of dry to damp, loose to medium dense, silty sand and sandy silt; A 5 foot thick gravel layer directly underlies the asphalt parking lot at GB-M-1. The origin of sand and silt fill at the sites was likely derived from grading and reuse of on-site, near surface materials of Colma Formation (Qc).

Colma Formation. Soils of the Colma Formation (Qc) were encountered at all five CUP sites below the artificial fill. The Colma Formation soils consisted predominantly of yellowish brown to yellowish gray, damp to moist, medium dense to very dense, silty sand and poorly graded sand with silt. Thin beds of clayey sand, sandy silt, silt, and clayey silt were encountered at the northerly sites (GB-11A, GB-23, GB-36-1 and GB-44-1). Layers of wet clay with sand and clayey gravel were encountered at the bottom of the two more southern borings, GB-44-1 and GB-M-1. Colma Formation soils at the five sites extended to the total depth of exploration (35 to 50 feet). Measured total unit weight for the Colma Formation soils at the five sites ranged from 101 to 115 pcf, with a moisture content ranging from 5 to 17 percent in the granular materials and 11 to 27 percent in the clay and silt layers.

GROUNDWATER

Groundwater was not encountered during drilling of our exploratory borings GB-11A, -23, -36-1 and -44-1 to total depths ranging from 35 to 50 feet. At GB-M-1, groundwater was encountered during drilling on September 28, 2009 at a depth of approximately 23 feet. A summary of our observed groundwater levels is presented in Table 3 – Observed Groundwater Levels. Seasonal variations are expected to cause fluctuations in groundwater levels.

TABLE 3 – OBSERVED GROUNDWATER LEVELS

Boring	Date of Observation	Depth to Groundwater (feet)
GB-11A	9/28/2009	Not Encountered
GB-23	9/25/2009	Not Encountered
GB-36-1	9/25/2009	Not Encountered
GB-44-1	10/19/2009	Not Encountered
GB-M-1	9/28/2009	23

CONCLUSIONS AND RECOMMENDATIONS

1.0 GENERAL

The following sections provide our conclusions and recommendations for evaluation and design of proposed station buildings at the five sites of CUP-11A, -23, -36-1, -44-1 and –M-1. According to preliminary site maps given us by Kennedy/Jenks Consultants, the station buildings at well sites CUP-23, -36-1, and –M-1 house chemical treatment facilities and the station building at well site CUP-44-1 houses filtration facilities. Based on our findings from our geotechnical field investigation, the GB-11A, -23, -36-1, -44-1 and -M-1 sites are underlain by artificial fill and Colma Formation.

We consider the proposed improvements to be geotechnically feasible, provided that our geotechnical recommendations are incorporated into design and construction documents.

2.0 SEISMIC DESIGN CONSIDERATIONS

2.1 **General.** The main seismic hazards at the site are expected to be strong ground shaking and dynamic settlement within isolated zones of loose fill. Our seismic design considerations, including fault rupture, ground shaking, liquefaction and dynamic settlement, inundation by tsunamis, seismically-induced landslides, and seismic design with respect to the 2006 International Building Code (which the 2007 California Building Code has adopted) are provided in the following sections.

2.2 **Fault Rupture.** No active or potentially active faults are known to cross the sites. Consequently, the hazard posed by ground rupture due to fault offset is considered to be negligible.

2.3 **Ground Shaking**. Strong ground shaking will occur at the site as a result of a moderate to large earthquake occurring on one of the active regional faults. The San Andreas Fault is closest to the sites (1.5 to 1.9 miles for all borings; GB-11A, -23, -36-1, -44-1 and -M-1) and therefore has the greatest capability of causing strong ground motions.

The California Geological Survey (CGS, formerly known as California Division of Mines and Geology) and United States Geological Survey (USGS) completed probabilistic seismic hazard maps in 1996 (Petersen et al., 1996), and subsequently updated fault parameters and revised the maps in 2002 (Cao, et al., 2003). USGS provides a web-based program to evaluate the USGS Probabilistic Uniform Hazard

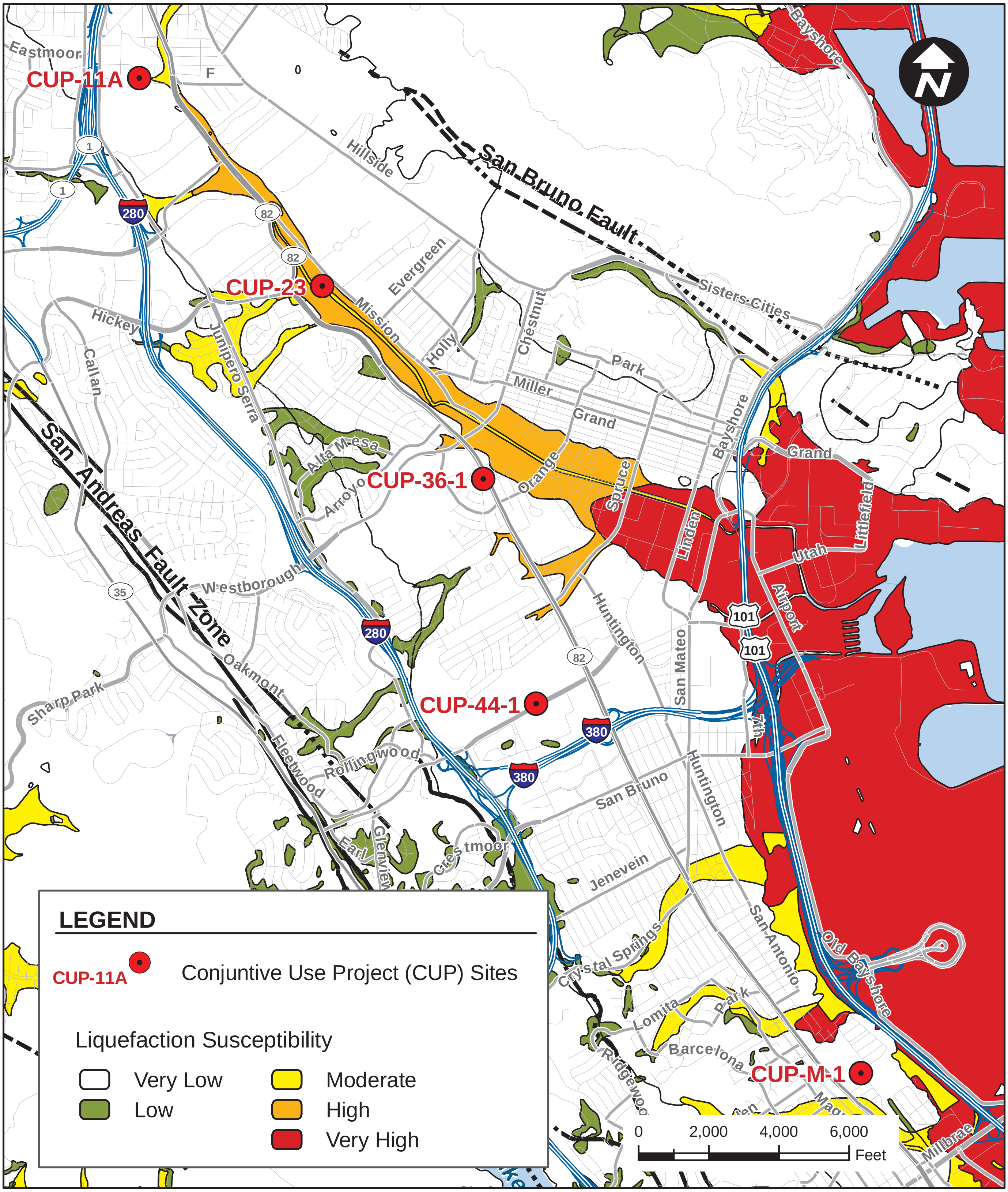
Response Spectra (http://earthquake.usgs.gov/research/hazmaps/design). Based on this data, the peak ground acceleration (PGA) at the site is estimated to be 0.71g for an earthquake having a 10 percent probability of exceedance in 50 years.

2.4 **Liquefaction and Dynamic Settlement.** Liquefaction is a phenomenon wherein a temporary, partial loss of shear strength occurs in a soil due to increases in pore pressure that result from cyclic loading during earthquakes. Saturated, loose to medium dense sands and silty sands are most susceptible to liquefaction. Consequences of liquefaction can include ground settlements, foundation failure, sand boils, and lateral spreading. Dynamic settlement is the densification of saturated and unsaturated soils during strong ground shaking. All soil types are prone to dynamic settlement, though loose, sand and silty sand are most susceptible.

The liquefaction susceptibility, as mapped by Witter et al. (2006), is illustrated on Figure 4 – Liquefaction Susceptibility Map. As can be seen from the figure, boring sites GB-11A, GB-36, GB-44-1, and GB-M-1 lie within a zone mapped as having very low liquefaction susceptibility. The mapped liquefaction susceptibility at site GB-23 is moderate. Because of the regional focus of the liquefaction susceptibility mapping, the data only generally correlates with areas of known liquefaction hazard. The site-specific data from the borings is considered to be more indicative of liquefaction and dynamic settlement hazard. The following discussion further describes this hazard based on our subsurface investigation and laboratory testing program.

Due to the absence of groundwater within the 35 to 50 feet of total exploration depth for each of the exploratory borings GB-11A, -23, -36-1 and -44-1, and the generally dense nature of the Colma Formation below this depth, liquefaction is not considered to be a significant consideration. Despite the observation of groundwater at a depth of 23 feet at the GB-M-1 site, liquefaction is also not considered to be a significant consideration because of the dense and clayey nature of the Colma Formation encountered at this site. Pore pressure generation and liquefaction may occur in isolated pockets of looser material within the Colma Formation, however, the amount of surface settlement resulting from liquefaction is considered to be negligible at the five sites.

FIGURE 4
LIQUEFACTION SUSCEPTIBILITY MAP



Source: Witter, R.C., et. al.., 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California: U.S. Geological Survey Open-File Report 06-1037

The amount of dynamic settlement for each site has been evaluated based on an anticipated earthquake event having a 10 percent probability of exceedance in 50 years. Dynamic settlement resulting from strong ground shaking at GB-11A and -23 is estimated at less than ¼ inches due to the dense nature of the near-surface Colma Formation beneath a relatively thin stratum of artificial fill. Dynamic settlement of the artificial fill at GB-36-1 is considered relatively significant with an estimate of up to 2 inches, provided proper mitigations are made in accordance with Section 6.1. As a result of medium dense silty sand within the upper 15 feet, dynamic settlement is estimated at 1 inch for GB-44-1. Dynamic settlement resulting at GB-M-1 is estimated at less than 1 ½ inches, as a result of medium dense silty sand in the Colma Formation above the groundwater level. The hazard posed by dynamic settlement is therefore considered to be low at GB-11A and,-23 and moderately high at GB-36-1, -44-1 and –M-1. Flexible pipe connections are recommended to accommodate dynamic settlements due to seismic loading.

2.5 **Inundation by Tsunamis.** Tsunamis are long period waves usually caused by underwater seismic disturbances, volcanic eruptions, or submerged landslides. The disturbance can occur thousands of miles from the San Francisco area, and generate a tsunami wave that affects the site. As tsunami waves approach the coast, they may increase in height to tens of feet.

Flooding due to tsunami is unlikely to occur at GB-11A, -23, -36-1 and -44-1 due to their relatively high ground elevations and distance from the open Northern California coastline. Although GB-M-1 is located on relatively low lying terrain at elevation 26 feet above Mean Sea Level (MSL), the potential of flooding during a tsunami is unlikely because of the distance to San Francisco Bay.

2.6 **Seismically-Induced Landslides.** Based on the flat topography surrounding the sites of GB-23, -44-1 and –M-1, seismically-induced landslide hazards do not exist at these sites. At GB-11A which is located on mildly sloping terrain (on the order of 5:1 horizontal to vertical side slope ratio), seismically-induced landslide hazards are considered not likely because of the dense nature of the subsurface soils and absence of shallow groundwater. Boring GB-36-1 is situated with very mild slopes (on the order of 10:1 horizontal to vertical side slope ratio) to the north and northeast towards the funeral home and El Camino Real. Seismically-induced landslide hazards are considered not likely due to the presence of generally dense granular materials and absence of shallow groundwater.

2.7 **Seismic Design Parameters.** The proposed improvements may be designed in accordance with the International Building Code Static Force Procedure (ICC, 2006) using the seismic parameters as presented in Table 4 – 2006 International Building Code (IBC) Seismic Design Parameters in developing the site seismic response:

TABLE 4 – 2006 INTERNATIONAL BUILDING CODE SEISMIC DESIGN PARAMETERS

	Site GB-11A	Site GB-23	Site GB-36-1	Site GB-44-1	Site GB-M-1
Site Class	C	C	D	D	D
S_s (1) at 0.2-second	2.162	2.129	2.105	2.160	2.105
S_1 (1) at 1-second	1.213	1.180	1.157	1.210	1.158
Site Coefficient F_a	1.0	1.0	1.0	1.0	1.0
Site Coefficient F_v	1.3	1.3	1.5	1.5	1.5

(1) Maximum Considered Earthquake (MCE) Spectral Response Acceleration (in units of g).

3.0 GROUNDWATER

With the exception of exploratory boring GB-M-1, groundwater was not encountered in the remaining exploratory borings. At GB-M-1, groundwater was encountered during drilling at a depth of 23 feet below ground surface. The observation of groundwater at GB-M-1 is consistent with the low lying topography (ground elevations of 25 to 30 feet above mean sea level). It should be noted that groundwater levels are influenced by seasonal variations in precipitation, local irrigation, groundwater pumping and other factors, and are therefore, subject to variation. As the proposed footing foundations are expected to be within the top 5 feet, groundwater is not anticipated within the depth of foundation excavation.

4.0 EARTHWORK

4.1 **General.** Given the earth materials on the project sites encountered during our exploration, the contractor should be able to carry out planned excavations using conventional heavy equipment.

Evaluation of the presence, or absence, and treatment of hazardous materials was not part of this study. If hazardous materials are encountered during excavation, proper handling and treatment during construction will depend on the contaminant type, concentration, and volatility of the contaminated materials.

General geotechnical considerations for site preparation, excavations, temporary shoring and bracing, engineered fill material, engineered fill placement and compaction, pipe bedding, and utility trench backfill are presented in the following sections.

4.2 **Site Preparation.** Site preparation will consist of demolition, excavation and removal of on-site materials such as pavement, concrete, abandoned utilities, and miscellaneous debris in preparation for the foundation excavations. Any creation of holes from the removal of such materials should be backfilled with engineered fill.

Recommendations for engineered fill are provided in Sections 4.5 and 4.6. Also as part of site preparation, the location of active underground utilities should be determined and, if affected by construction activities, should be relocated or protected.

4.3 **Excavations.** We anticipate that excavations for the planned building improvements to extend only a few feet below existing ground elevation. Since GB-11A is located near the foot of mildly sloping terrain, greater excavation may be necessary at this site.

Shallow excavations for the buildings will allow for unshored excavations with adequately sloped sidewalls. Vertically shored walls or braced excavations are anticipated where space constraints may not allow for open, sloped excavations. At a minimum, excavations should be constructed in accordance with the current California Occupational Safety and Health Administration (OSHA) regulations (Title 8, California Code of Regulations) pertaining to excavations. Temporary cut slopes are expected to be stable for configurations described in Title 8 for Type C soils and when unsupported, should be cut back no steeper than 1 ½ horizontal to 1 vertical. All excavations should be closely monitored during construction to detect any evidence of instability.

Care should be taken when excavating near existing utilities and pipelines. Excavations can undermine support of adjacent existing pipelines and other subsurface structures. We recommend that some form of vertical shoring system be considered for excavated sidewalls that are adjacent to existing pipelines or other known buried adjacent structures.

Some of the near surface loose soils at the five sites will likely be removed during excavation for the proposed improvements. If any footings are founded above loose soils, over-excavation of loose soils and replacement with engineered fill may be required. Remediation of loose materials at intermediate depths can be performed using densification improvement methods, as discussed in Section 6.1.

4.4 **Temporary Shoring and Bracing.** The type and design of the shoring will depend on the depth of excavation and excavation-bracing sequence. The shoring and bracing design and installation should be the responsibility of the construction contractor. As a general guideline, construction procedures, excavations, and design and construction of any temporary shoring should comply with the current OSHA Title 8 regulations pertaining to excavations. The shoring and bracing should accommodate surcharge loads that may be imposed by adjacent structures, traffic, or construction activities.

Possible shoring schemes include soldier pile and lagging and steel sheeting, both of which may include internal bracing struts to limit lateral deflections. Such braced and shored excavations will be subjected to lateral earth pressures. Recommended active, at-rest, and passive lateral earth pressures are provided in Section 5.

Horizontal and vertical movements of the ground are possible in the vicinity of the excavations. These movements can generally be reduced to acceptable levels by use of a properly designed and constructed shoring system. Measures should be taken to prevent the loss of sand through the gaps in the shoring or lagging.

4.5 **Engineered Fill Material.** Material for engineered fill should be inorganic, well graded, free of rocks or clods greater than 4 inches in greatest dimension or any other deleterious materials, and have a low potential for expansion. The material should have a liquid limit less than 35, a plasticity index less than 15 and no more than 25 percent passing the No. 200 sieve. Existing on-site soil may be re-used as engineered fill provided it meets the above criteria.

4.6 **Engineered Fill Placement and Compaction.** Engineered fill should be placed in layers no greater than 8 inches in uncompacted thickness, conditioned with water or allowed to dry to achieve a moisture content near optimum, then mechanically compacted to at least 90 percent relative compaction based on ASTM D1557. All engineered fill placed to support footings and the upper 6 inches of engineered fill supporting slabs-on-grade should be mechanically compacted to at least 95 percent relative compaction as determined by ASTM D1557. Specific engineered fill placement requirements exist for GB-36-1 as outlined in Section 6.1. All compaction should be performed using mechanical compaction means; flooding or jetting should not be used as a means to achieve compaction. The ASTM D1557 laboratory compaction tests should be performed at the time of construction to provide a proper basis for compaction control.

4.7 **Pipe Bedding for Small Diameter Pipes.** Pipe bedding should consist of well-graded sand or a sand-gravel mixture. Maximum gravel size should be ½ inch and the bedding material should have less than 12 percent passing the No. 200 sieve. Uniformly graded material such as pea gravel should not be used as pipe bedding material. Pipe bedding should have a minimum thickness of 6 inches beneath the pipe and 6 inches above the pipe. If soft or otherwise unsuitable soils are exposed in the bottom of the trench excavation, the necessity of over-excavation should be evaluated by the project geotechnical engineer. All pipe bedding should be placed to achieve uniform contact with the pipe and mechanically compacted to a minimum relative compaction of 90 percent per ASTM D1557. Flexible pipe connections are recommended to accommodate dynamic settlements due to seismic loading. Estimates of dynamic settlement at each site are provided in Section 2.4 – Liquefaction and Dynamic Settlement.

4.8 **Utility Trench / Pipe Backfill.** Utility and pipe trenches may be backfilled above the pipe zone with excavated on-site soils, provided they meet the gradation requirements of engineered fill. The backfill material should be placed in layers no greater than 8 inches in uncompacted thickness, moisture conditioned or allowed to dry to achieve a moisture content near optimum, then mechanically compacted to at least 90 percent relative compaction based on ASTM D1557. The upper 2 feet should be compacted to at least 95 percent relative compaction in areas where structural or traffic loads are anticipated.

5.0 LATERAL EARTH PRESSURES

5.1 Active Earth Pressure. Active earth pressures are imposed by the soil on walls that are unrestrained so that the top of the wall is free to translate or rotate at least 0.004H, where H is the height of the wall. The active earth pressure may be calculated using a design equivalent fluid pressure (EFP) for each of the sites as indicated in Table 5.1 – Active Earth Pressures.

TABLE 5.1 – ACTIVE EARTH PRESSURES

Site Location	GB-11A	GB-23	GB-36-1	GB-44-1	GB-M-1
Active EFP (1) (pcf)	30	30	30	35	35

1. EFP assumes that excavations do not extend below the groundwater table.

5.2 At-Rest Earth Pressure. At-rest pressures should be used for design of walls that are restrained such that the deflections required to develop active earth pressures cannot occur or are undesirable. The at-rest earth pressures may be calculated using a design EFP for each of the sites as indicated in Table 5.2 – At-Rest Earth Pressures.

TABLE 5.2 – AT-REST EARTH PRESSURES

Site Location	GB-11A	GB-23	GB-36-1	GB-44-1	GB-M-1
At-Rest EFP (1) (pcf)	50	50	50	55	55

1. EFP assumes that excavations do not extend below the groundwater table.

5.3 Seismic Earth Pressure. In addition to the active and at-rest pressures, retaining walls should be designed to consider additional earth pressures due to earthquake loading. The increment in earth pressure due to seismic loading, for both restrained and unrestrained below-grade walls, may be calculated using an inverted triangular distribution with the pressure at the top of the wall equal to a design earth pressure (EP) of 50H, wherein H is the height of the wall in feet, and diminishes to zero at the base of the wall, as indicated in Table 5.3 – Seismic Earth Pressures.

TABLE 5.3 – SEISMIC EARTH PRESSURES

Site Location	GB-11A	GB-23	GB-36-1	GB-44-1	GB-M-1
Seismic EP (1) at Top of Wall (psf)	50 H (2)	50 H (2)	50 H (2)	55 H (2)	55 H (2)

1. EFP assumes that excavations do not extend below the groundwater table.
2. H is the height of the wall in feet, and diminishes to zero at the base of the wall.

5.4 **Passive Earth Pressure.** Lateral loads on structures can be resisted by passive pressures that develop against the sides of below-grade structures such as walls or footings. The passive pressure depends on the lateral displacement of the wall or footing. In accordance with FEMA 356 (FEMA, 2000), the ultimate passive pressure is mobilized at a displacement of approximately 6 percent of the wall height. The ultimate passive earth pressure may be calculated using a design EFP that corresponds to the ultimate EFP as long as the structure can be mobilized to such level of displacement and still does not exceed the allowable displacement of the structure. Oftentimes, the displacement to achieve ultimate passive earth pressures exceeds the allowable displacement of the structure. Consequently, a design EFP needs to be reduced when the allowable displacement of the structure is less than 6 percent of the wall height. For displacements of approximately 0.8 and 3 percent of the wall height, the design EFP may be reduced to 50 and 85 percent of the ultimate EFP. Passive pressures computed using these design EFPs may be combined with the base friction mobilized at the concrete-soil interface to resist lateral loading (see Section 5.5). The passive earth pressures may be computed using the following design EFPs as indicated in Table 5.4 – Passive Earth Pressures:

TABLE 5.4 – PASSIVE EARTH PRESSURES

Site Location	GB-11A	GB-23	GB-36-1	GB-44-1	GB-M-1
Passive Ultimate EFP (1) at 6% Wall Height Displacement (pcf)	300	280	300	320	320
Passive EFP (1) at 3% Wall Height Displacement (pcf)	250	240	250	270	270
Passive EFP (1) at 0.8% Wall Height Displacement (pcf)	150	140	150	160	160

1. EFP assumes that excavations do not extend below the groundwater table.

5.5 **Base Friction.** A coefficient of friction of 0.4 may be used for estimating the resistance due to base friction. The coefficient should be multiplied by the dead load only. The passive earth pressure and base friction mobilized at the concrete-subgrade interface may be combined to resist lateral loading.

6.0 FOUNDATIONS

6.1 **Subgrade Preparation.** Subgrades to new shallow foundations for the proposed structures should be prepared to provide a flat, relatively dry, and firm working surface. If any unsuitable materials, such as, soft clays or silts, soils containing organic material, debris or other deleterious materials are encountered at subgrade, they should be over-excavated and restored to grade with engineered fill in accordance with Sections 4.5 and 4.6. The fill soils encountered in our exploratory borings were suitable for support of the proposed improvements provided the upper 12 inches are scarified, moisture conditioned, and recompacted. We recommend that the upper 12 inches of subgrade be scarified, moisture conditioned to near optimum moisture content, and compacted in accordance with Sections 4.5 and 4.6. The subgrade should be free of loose debris and ponded water prior to placing reinforcing steel and concrete.

Dynamic settlements of loose to medium dense granular soils at GB-36-1, -44-1, and -M-1 are anticipated during an earthquake event if these sites are not mitigated. Estimates of dynamic settlement at each site are provided in Section 2.4 – Liquefaction and Dynamic Settlement. Special mitigation measures against settlement at CUP-36-1 require additional over-excavation of artificial fill materials below any foundations. This over-excavation must extend three feet below proposed footing elevation, or, if competent Colma Formation materials are encountered within those three feet, six inches into Colma Formation materials. Engineered fill shall then be placed, moisture treated to near optimum water content and mechanically compacted to 95 percent relative compaction as determined by ASTM D1557.

6.2 **Shallow Foundation Alternatives.** A shallow foundation system is suitable for support of the proposed improvements at the sites. Alternatives for shallow foundation systems include grade beams / shallow footings, mat foundations, and post-tensioned foundations.

Grade Beams / Shallow Footings: Based on the findings from our subsurface evaluation and laboratory testing, the ultimate bearing capacity of soils below new footings within the footprint of proposed buildings varies according the geotechnical characteristics of soils encountered at each site. We recommend an allowable bearing capacity of 3,000 pounds per square foot (psf) for soils below new footings at the GB-11A, -23, -36-1, -44-1 and -M-1 sites. This bearing capacity includes a factor of safety of at least three against bearing failure, and is applicable to newly constructed footings with widths of at least 18 inches and footing embedment of at least 24 inches below lowest adjacent grade.

A static modulus of subgrade reaction of 60 pounds per cubic inch (pci) may be used in order to develop soil springs below the foundation elements. For the lateral resistance of grade beams and footings, the geotechnical design parameters provided in the Lateral Earth Pressures section may be used.

As discussed in Section 2.4, dynamic settlements of up to approximately ¼ inch may affect the GB-11A and -23 sites during an earthquake event. The remaining three sites are more susceptible to significant dynamic settlements during an earthquake event. Larger dynamic settlements, on the order of 1 to 2 inches at GB-36-1, -44-1 and -M-1 are anticipated during an earthquake event if these sites are not mitigated. These dynamic settlements are in addition to the settlements estimated for the building loads described above. Long-term consolidation settlements are not likely due to the granular nature of much of the subsurface soils, and the stiffness and overconsolidation of clayey soils.

<u>Mat Foundations</u>**:** Effects from differential dynamic settlements at the GB-36-1, 44-1 and M-1 sites may be limited by supporting the structures at these sites on structurally rigid mat foundations. A mat foundation is a large concrete slab, designed by a structural engineer for specific use, to interface one or more columns or pieces of equipment with the foundation soil. It may encompass the entire foundation footprint or only a portion. The mat contact stresses are generally lower than other shallow foundation types due to distribution of stress over a larger area and stress compensation from excavated soil. Thickness and reinforcement of the mat foundation should be in accordance with the recommendations of a structural engineer. The appropriate allowable contact pressure(s) beneath the mat foundations will vary with their size, shape, and other factors. To limit foundation static settlements to less than ½ inch for dead and live loads and less than 1 inch for total loads including wind and seismic, the contact pressure beneath the mats should not exceed the allowable bearing capacities as recommended above for grade beams / shallow foundations. Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the mat. A design coefficient of subgrade reaction, $\mathbf{K_{v1}}$, of 260 kips per cubic foot (kcf) in compacted fill soils may be used for evaluating such deflections at the sites. This value is based on a square foot area and should be adjusted for the planned mat size. The coefficient of subgrade reaction, $\mathbf{K_B}$, for a mat of a specific dimension may be evaluated using the following equation:

$$\mathbf{K_{B}, = K_{v1}\ [(B+1)/2B]^2\ [(1+0.5(B/L)/1.5]}$$

where **B** is the width and **L** is the length of the foundation measured in feet.

Mat foundations bearing on fill may be designed using a coefficient of friction of 0.4 (total frictional resistance equals coefficient of friction times the dead load). The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed two-thirds of the total allowable resistance.

***Post-Tensioned Foundations*:** Effects from differential dynamic settlements at the GB-36-1, -44-1 and -M-1 sites may be limited through the application of post-tensioning in reinforcing, and hence, increasing the structural rigidity of grade beams / shallow footings. Thickness and reinforcement of a post-tensioned foundation should be in accordance with the recommendations of a structural engineer.

6.3 **Floor Slabs.** Slabs-on-grade should be supported on a 12-inch thick mat of compacted, engineered fill. Material for engineered fill and compaction requirements are presented in Sections 4.5 and 4.6. For moisture-sensitive flooring, floor slabs resting on soil should be underlain, at a minimum, by a capillary break system. We recommend 6 inches of clean coarse sand or pea gravel. When floor dampness is a concern, possibly in a low-lying area such as GB-M-1, floor slabs should be underlain by a vapor barrier and capillary break system. We recommend a system consisting of a 10-mil polyethylene (or equivalent) membrane placed over 6 inches of clean coarse sand or pea gravel. The exposed subgrade should be moistened just prior to the placement of the capillary break system. A sand layer above the moisture barrier to aid in concrete curing should be evaluated by the structural engineer. The slab underlayment including the capillary break can be taken as part of the 12-inch thick pad of compacted, engineered fill described above. Flooring and waterproofing consultants should be consulted for additional slab waterproofing recommendations.

7.0 CORROSION

Schiff Associates performed corrosivity laboratory tests on one soil sample for each of the five completed sites. Their laboratory results are included in Appendix A – Supporting Geotechnical Data. They performed the following tests:

- Resistivity (As-Received and Saturated),
- pH,
- Electrical Conductivity,
- Chemical Analyses of Cations (Calcium, Magnesium, Sodium, Potassium)
- Chemical Analyses of Anions (Carbonate, Bicarbonate, Fluoride, Chloride, Sulfate, Phosphate)
- Chemical Analyses of Ammonium
- Chemical Analyses of Nitrate

Electrical resistivities indicate soils range from moderately corrosive to highly corrosive to ferrous metals in GB-11A, -M-1 and -44-1.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 **Geotechnical Observation of Construction Activities.** We should be retained during construction to provide site observation and consultation concerning the condition of the bottom of excavations pertaining to foundation construction and pipeline trench excavation. Foundation grades should be observed and, where necessary, tested under the direction of a qualified geotechnical engineer to verify compliance with final design recommendations. All site preparation work and excavations should also be observed to compare the generalized site conditions assumed in the final design report with those found on site at the time of construction.

8.2 **Existing Underground Utilities.** A number of underground water main pipelines pass beneath and in the vicinity of the proposed sites. Other existing subsurface lines include the SFPUC transmission lines, sanitary sewer and storm sewer lines. Some of these utilities were located and marked prior to our subsurface investigation so that we would not damage them during drilling.

The City may consider remarking these utilities prior to construction of the improvements so they remain visible during earthwork and construction of the improvements. Any excavations made adjacent to existing utilities should be backfilled with on-site or imported soil to at least 90 percent relative compaction as evaluated by ASTM D 1557.

8.3 **Surface Drainage.** Proper surface drainage is essential for satisfactory site performance. Positive drainage should be provided and maintained to direct surface water away from building foundations and other site improvements. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet or greater away from the foundations, flatwork, and tops of slopes. Runoff should then be directed by the use of swales or pipes into a collective drainage system. Surface water should not be allowed to pond adjacent to footings. We further recommend that the proposed structure be equipped with appropriate roof gutters and downspouts. Downspouts should discharge to a system of closed pipes that transport the collected water to a suitable discharge facility. We recommend that drought tolerant vegetation be used for site landscaping. Irrigation should be kept at levels just sufficient to maintain plant vigor.

9.0 CLOSURE

The conclusions and recommendations presented herein are professional opinions based on geotechnical and geologic data and the project as described. A review by this office of any foundation, excavation, grading plans and specifications, or other work product that relies on the content of this report, together with the opportunity to make supplemental recommendations is considered an integral part of this study. Should unanticipated conditions come to light during project development or should the project change from that described, we should be given the opportunity to review our recommendations.

The findings and professional opinions presented in this report are presented within the limits prescribed by the client, in accordance with generally accepted professional engineering and geologic practices. There is no other warranty, either express or implied, regarding the conclusions, recommendations, and opinions presented in this report.

Submitted by:
GEOTECHNICAL CONSULTANTS, INC.

Dustin Agnew
Project Engineer

G. Neelakantan, P.E., G.E.
Geotechnical Engineer 2391

REGISTERED PROFESSIONAL ENGINEER
G. NEELAKANTAN
No. 2391
Exp. 9/30/2011
GEOTECHNICAL
STATE OF CALIFORNIA

REFERENCES

Blake, T.F., 2000, “EQFAULT, A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration Using Three-Dimensional California Faults as Earthquake Sources.”

Bonilla, M.G., 1998, Preliminary Geologic Map of the San Francisco South 7.5’ Quadrangle and Part of the hunters Point 7.5’ Quadrangle, San Francisco Bay Area, California, U.S. Geological Survey, Open-File Report 98-354.

Brabb, E.E., Graymer, R.W., and Jones, D.L., 1998, Geology of the onshore part of San Mateo County, California: A Digital Database, U.S. Geological Survey Open-File Report 98-137.

California Geological Survey (CGS), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0, Bryant, W. A. (compiler): http://www.consrv.ca.gov/CGS/information/publications/QuaternaryFaults_ver2.htm.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, “The Revised 2002 California Probabilistic Seismic Hazard Maps,” June.

Chaudhary & Associates, Incorporated, 2008a, Topographic Maps for Sites 10A, 18, 19 and 22A, South Westside Groundwater Conjunctive Use Project, September.

Chaudhary & Associates, Incorporated, 2008b, Topographic Maps for Site 41-4, South Westside Groundwater Conjunctive Use Project, March.

Coduto, D.P., 1998, Geotechincal Engineering: Principles and Practice, Prentice Hall.

Department of Conservation, Division of Mines and Geology, 1997, “Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California,” March.

Farrell, T., Taylor, A., 2004, “Rammed Aggregate Pier Design and Construction in California – Performance, Constructability, and Economics,” SEAOC 2004 Convention Proceedings.

Farrell, T., FitzPatrick, B., Kenney, W., 2008, “Uplift Testing of Rammed Aggregate Pier Systems,” ASCE Geo Institute Geotechnical Earthquake Engineering and Soil Dynamics IV Proceedings, May.

Federal Emergency Management Agency (FEMA), 2000, “Prestandard and Commentary for the Seismic Rehabilitation of Buildings”, FEMA 356, November.

Frankel, A., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1996, National Seismic Hazard Maps: Documentation, June 1996, USGS Open File Report 96-532, July 19.

Geotechnical Consutlants, Inc. (GTC), April 2009, Geotechnical Report South Westside Groundwater Basin Conjunctive Use Project, San Mateo County, California, for Kennedy/Jenks Consultants.

International Code Council (ICC), 2006, 2006 International Building Code.

Kristiansen, H., and Davise, M., 2004, "Ground Improvement Using Rapid Impact Compaction," 13th World Conference on Earthquake Engineering, Vancouver, British Columbia, Canada, Paper No. 496, August.

Majchrzak, M., Lew, Marshall, Sorensen, K., and Farrell, T., 2004, "Settlement of Shallow Foundations Constructed Over Reinforced Soil: Design Estimates vs. Measurements, "Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering, New York, NY, April.

Montgomery Watson Harza, Incorported (MWH), 2007, Draft Conceptual Engineering Report, CS-826 South Westside Groundwater Basin Conjunctive Use Program, Prepared for the San Francisco Public Utilities Commission, Illustrated Figures, November.

Montgomery Watson Harza, Incorported (MWH), 2008, Final Conceptual Engineering Report, CS-826 South Westside Groundwater Basin Conjunctive Use Project, Prepared for the San Francisco Public Utilities Commission, November.

Naval Facilities Engineering Command (NAVFAC), 1982a, "Soil Mechanics," Design Manual 7.1, May (Revalidated September, 1986).

Naval Facilities Engineering Command (NAVFAC), 1982b, "Foundations and Earth Structures," Design Manual 7.2, May (Revalidated September, 1986).

Petersen, M.D., Bryant, W.A., Cramer, C.H., Cao, T., Reichle, M., Frankel, A.D., Lienkaemper, J.J., McCrory, P.A., and Schwartz, D.P., 1996, Probabilistic Seismic Hazard Assessment for the State of California, CDMG Open-File Report 96-08. http://www.consrv.ca.gov/cgs/rghm/psha/ofr9608/Pages/Index.aspx

Pradel, D., 1998, "Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 124, No. 4, pp. 364 – 368, and Volume 124, No. 10, p. 1048.

San Francisco Public Utilities Commission (SFPUC) – Engineering Management Bureau, 2006, "General Seismic Requirements for Design of New Facilities and Upgrade of Existing Facilities," Engineering Standard General Seismic Design Requirements, EMB Document WSIP/CSP-001, August 15 (Updated with Addenda 1 and 2, August 21, 2007).

San Francisco Public Utilities Commission (SFPUC), 2008, 2007 Annual Groundwater Monitoring Report, Westside Basin, San Francisco and San Mateo Counties, California, April 25.

Seed, H.B. and Idriss, I.M., 1971, "Simplified Procedure for Evaluating Soil Liquefaction during Earthquakes," Journal of Geotechnical Engineering Division, ASCE, Volume 97, No. 9, pp. 1249-1273.

Seed, H.B., Tokimatsu, K., Harder, L.F., and Chung, R.M., 1985, "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Journal of Geotechnical Engineering, ASCE, Volume 111, No. 12, pp. 1425 – 1445.

Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, J.D., Kayen, R.E., and Faris, A., 2003, "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework," 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Keynote Presentation, H.M.S. Queen Mary, Long Beach, California, 71 pp., April 30.

TerraSystems, Incorporated, Undated, "Rapid Impact Compaction, Another Form of Dynamic Compaction?," Terra Notes, A Ground Improvement Update from TerraSystems, Lovettsville, Virginia.

Terzaghi, K., Peck, R.B., and Mesri, G., 1996, Soil Mechanics in Engineering Practice, Third Edition, Wiley-Interscience.

Tokimatsu, K. and Seed, H.B., 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," American Society of Civil Engineers, Journal of Geotechnical Engineering, Vol. 118.

United States Geological Survey (USGS), 1993, San Francisco South Quadrangle, California, 7.5-Minute Series (Topographic), Scale 1:24,000.

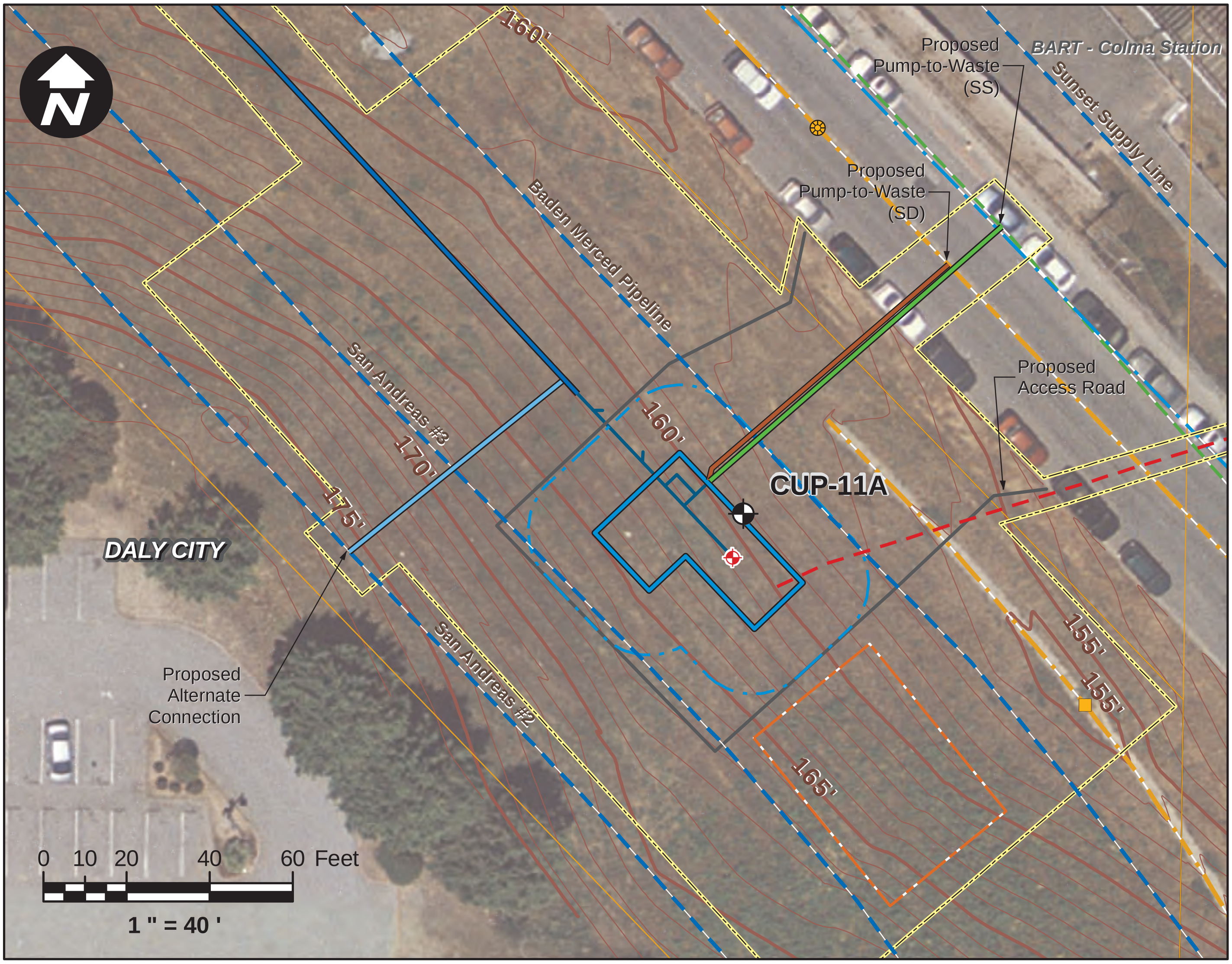
United States Geological Survey (USGS), 2008, Seismic Hazard Curves, Response Parameters and Design Parameters: Earthquake Ground Motion Parameters, A Java-based Application, Version 5.0.9., http://earthquake.usgs.gov/research/hazmaps/design, October 6.

Witter, R.C., Knudsen, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., Randolph, C.E., Brooks, S.K. and Gans, K.D., 2006, "Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California," U.S. Geological Survey OFR 2006-1037, Scale 1:200,000. http://pubs.usgs.gov/of/2006/1037/.

Working Group on California Earthquake Probabilities (WGCEP), 2003, "Earthquake Probabilities in the San Francisco Bay Region: 2002–2031," USGS Open-File Report 03-214. http://pubs.usgs.gov/of/2003/of03-214/.

Working Group on California Earthquake Probabilities (WGCEP), 2008, "The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF2)," USGS Open-File Report 2007-1437, http://pubs.usgs.gov/of/2007/1437/of2007-1437/.

Youd, T.L. and Idriss, I.M., eds., 2001, "Summary Report of Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Technical Report NCEER 97-0022, October.



Legend

- Geotecnical Boring by GTC in September 2009.
- Proposed Test Well - Phase 1
- Proposed Construction Area for Test Well and Connections
- Proposed Staging Area Boundary
- Proposed Construction Area-16ft Building Buffer
- Proposed Building with Chemical Treatment
- Existing Parcels - San Mateo County
- Proposed Access Road
- Proposed Connection Main
- Proposed Alternate Connection
- Proposed Pump-to-Waste (SS)
- Proposed Pump-to-Waste (SD)
- Existing 5-Foot Contour Lines
- Existing 1-Foot Contour LInes
- Existing PG&E Pole
- Proposed Underground Electrical
- Existing Transmission Line - SFPUC
- Existing Water - CalWater
- Existing Sanitary Sewer - DalyCity
- Existing Stormdrain Catch Basin - Colma
- Existing Stormdrain Manhole - Colma
- Existing Stormdrain - Colma

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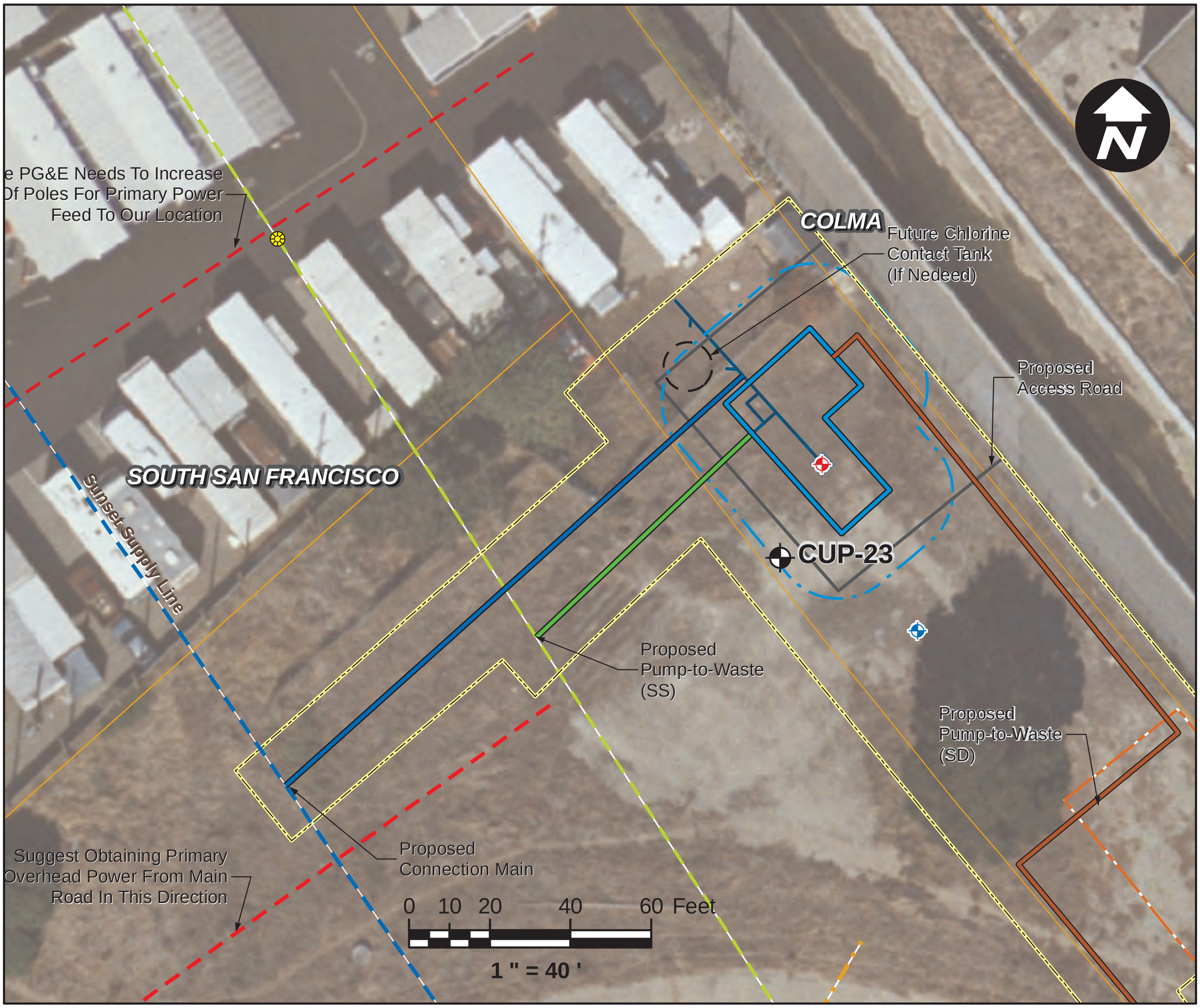
500 Sansome St., Suite 402
San Francisco, CA 94111

BORING LOCATION PLAN FOR CUP-11A	PLATE 1
SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
DECEMBER 2009	SF09020



Legend

- Geotecnical Boring by GTC in September 2009.
- Proposed Test Well - Phase 2
- Proposed Monitoring Well - Phase 2
- Proposed Construction Area for Test Well & Connections
- Proposed Staging Area Boundary for Well Building
- Proposed Construction Area-16ft Building Buffer
- Proposed Building and Chemical Treatment
- Proposed Access Road
- Existing Parcels - San Mateo County
- Proposed Connection Main
- Proposed Pump-to-Waste (SS)
- Proposed Pump-to-Waste (SD)
- Existing Transmission Line - SFPUC
- Proposed Underground Electrical
- Existing Sanitary Sewer Manhole - DalyCity
- Existing Sanitary Sewer - DalyCity
- Existing Stormdrain - SSF

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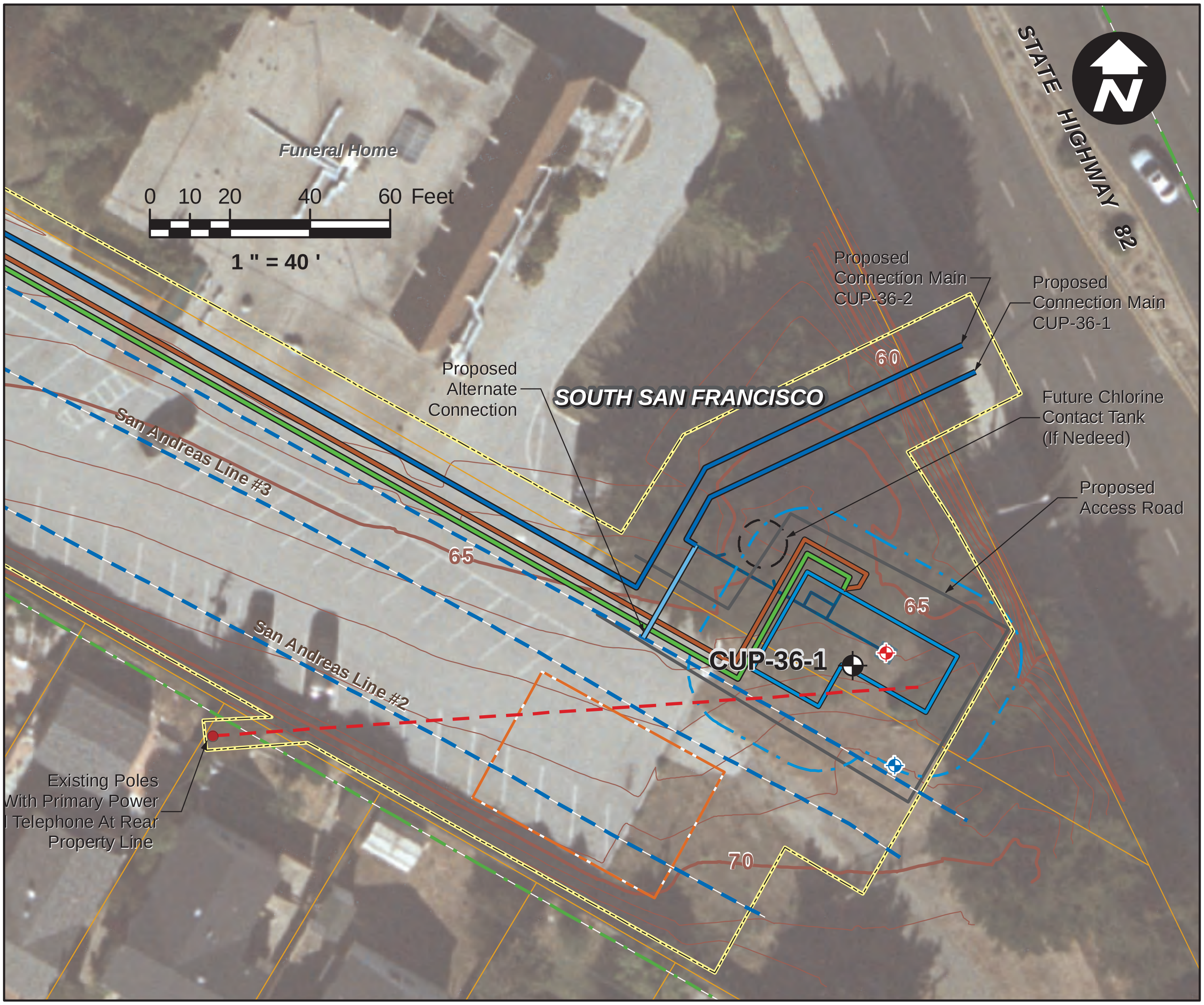
BORING LOCATION PLAN FOR CUP-23	PLATE 2
SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
DECEMBER 2009	SF09020



Legend

- Geotecnical Boring by GTC in September 2009.
- Proposed Test Well - Phase 2
- Monitoring Well Installed in Phase 1
- Proposed Construction Area for Test Well and Connections
- Proposed Staging Area Boundary for Well Building
- Proposed Construction Area-16ft Building Buffer
- Proposed Building with Chemical Treatment
- Proposed AccessRoad
- Proposed Connection Main
- Proposed Alternate Connection
- Proposed Pump-to-Waste (SS)
- Proposed Pump-to-Waste (SD)
- 5-Foot Contour Lines
- 1-Foot Contour Lines
- Existing Parcels - San Mateo County
- Existing Transmission Line - SFPUC
- Existing PG&E Pole
- Proposed Underground Electrical
- Existing Sanitary Sewer Manhole - SSF
- Existing Sanitary Sewer - SSF

GTC

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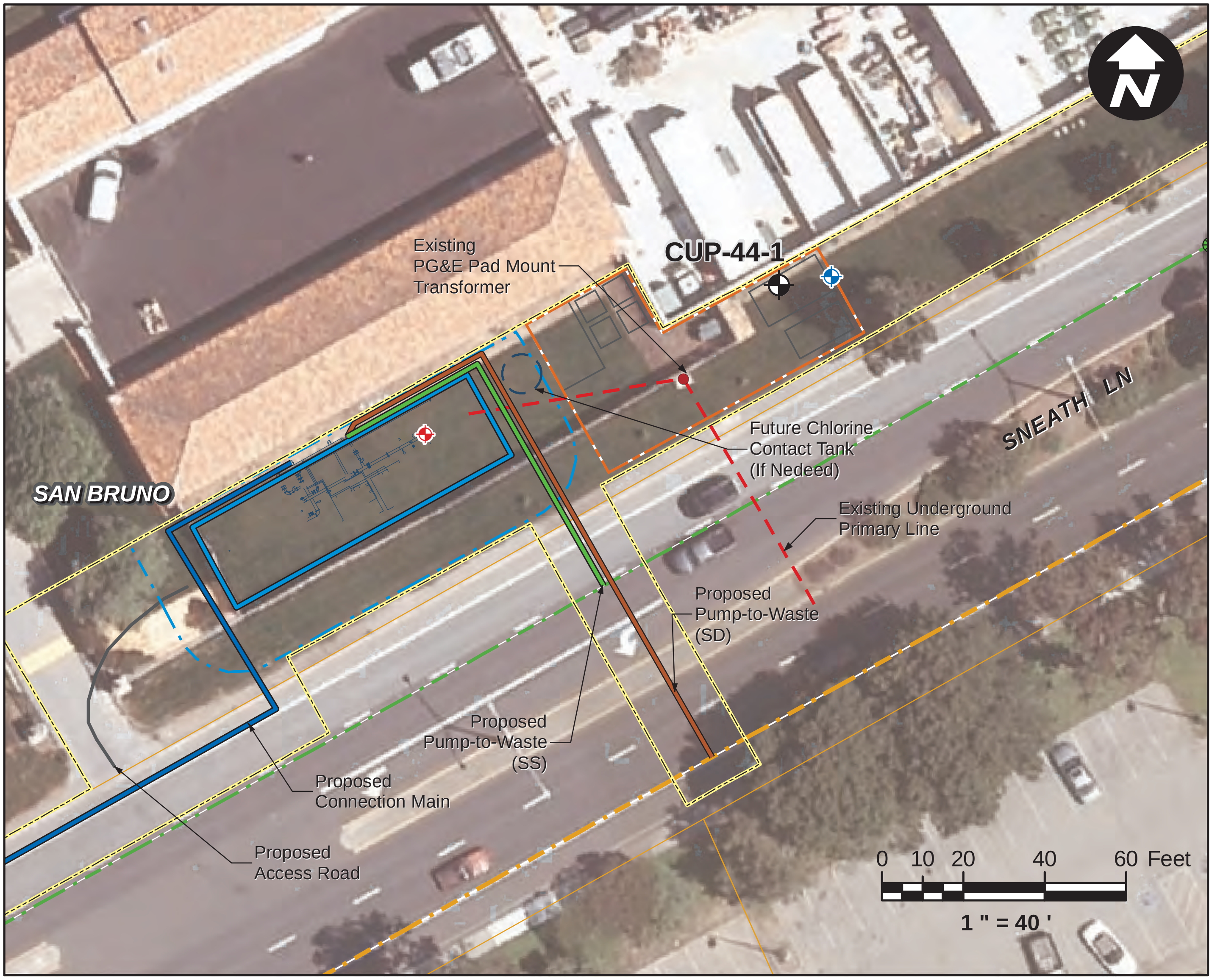
500 Sansome St., Suite 402
San Francisco, CA 94111

BORING LOCATION PLAN FOR CUP-36-1	PLATE 3
SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
DECEMBER 2009	SF09020



Legend

- Geotecnical Boring by GTC in September 2009.
- Proposed Test Well - Phase 2
- Proposed Monitoring Well - Phase 2
- Proposed Construction Area for Test Well and Connections
- Proposed Staging Area Boundary for Well Building
- Proposed Construction Area-16ft Building Buffer
- Proposed Building with Filtration
- Proposed Access Road
- Proposed Connection Main
- Proposed Alternate Connection
- Proposed Pump-to-Waste (SS)
- Proposed Pump-to-Waste (SD)
- Topography
- Existing Parcels - San Mateo County
- Existing Transmission Line - SFPUC_Surveyed
- Existing PG&E Transformer
- Proposed Underground Electrical
- Existing Water - CalWater
- Existing Catch Basin - San Bruno
- Existing Manhole - San Bruno
- Existing Storm Drain - San Bruno
- Existing Sanitary Sewer Manhole - San Bruno
- Existing Sanitary Sewer - San Bruno

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome St., Suite 402 San Francisco, CA 94111	BORING LOCATION PLAN FOR CUP-44-1	PLATE 4
	SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
	DECEMBER 2009	SF09020

Legend

- Geotecnical Boring by GTC in September 2009.
- Proposed Test Well - Phase 2
- Proposed Monitoring Well - Phase 2
- Proposed Construction Area for Test Well and Connections
- Proposed Staging Area Boundary for Well Building
- Proposed Construction Area-16ft Building Buffer
- Proposed Building with Chemical Building
- Proposed Access Road
- Proposed Connection Main
- Proposed Well Connnection Pipe
- Proposed Alternate Connection
- Proposed Pump-to-Waste (SS)
- Proposed Pump-to-Waste (SD)
- Existing Parcels - San Mateo County
- Transmission Line - SFPUC_Surveyed
- Existing PG&E Pole
- Existing Over Head Electrical
- Proposed Underground Electrical
- Existing Water - DalyCity
- Existing Water - CalWater

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome St., Suite 402 San Francisco, CA 94111	BORING LOCATION PLAN FOR CUP-M-1	PLATE 5
	SOUTH WESTSIDE GROUNDWATER BASIN CUP PROJECT	
	DECEMBER 2009	SF09020

GEOTECHNICAL REPORT – CUP-3A AND CUP-7 SITES, REGIONAL GROUNDWATER STORAGE AND RECOVERY PROJECT, NOVEMBER 2011 (REVISED JANUARY 2012)

GEOTECHNICAL CONSULTANTS, INC.
Geotechnical Engineering • Geology • Hydrogeology

GEOTECHNICAL REPORT
CUP-3A AND CUP-7 SITES
REGIONAL GROUNDWATER STORAGE AND RECOVERY PROJECT
SAN MATEO COUNTY, CA

November 2011
(Revised January 2012)

Prepared for:

San Francisco Public Utilities Commission
1155 Market Street
San Francisco, California 94103

Owner:

San Francisco Public Utilities Commission

GTC Project No. SF11004

500 Sansome Street, Suite 402 • San Francisco, CA 94111 • (415) 981-9950

GEOTECHNICAL CONSULTANTS, INC.
Geotechnical Engineering • Geology • Hydrogeology

Mr. Thomas Hull, S.E.
San Francisco Public Utilities Commission
1155 Market Street
San Francisco, California 94103

November 28, 2011
(Revised January 16, 2012)
GTC Project No. SF11004

Subject: Geotechnical Report
Regional Groundwater Storage & Recovery Project
CUP-3A and CUP-7 Sites
San Mateo County, California

Dear Mr. Hull:

The San Francisco Public Utilities Commission (SFPUC) is planning for the design and construction of proposed improvements to facilitate groundwater well stations, and chemical treatment and filtration facilities at two designated CUP-3A and CUP-7 sites located in northern San Mateo County, California. The proposed wells are part of the Regional Groundwater Storage and Recovery Project. We have previously submitted geotechnical reports for ten other GSR sites located in northern San Mateo County. We prepared this report (revised from the previously submitted report dated November 28, 2011) presenting our geotechnical findings, conclusions, and recommendations for the proposed improvements at the CUP-3A and CUP-7 sites. This report was developed in accordance with Task Order No. 6 of the design services Contract No. CS-998B.

Sincerely,
Geotechnical Consultants, Inc.

Nick S. Ng, G.E.
Senior Geotechnical Engineer

TABLE OF CONTENTS

Page

TABLES

FIGURES

PLATE

APPENDIX A – SUPPORTING GEOTECHNICAL DATA

INTRODUCTION

This geotechnical report presents the findings, conclusions, and recommendations of our geotechnical study performed for proposed buildings to facilitate groundwater well stations, and chemical treatment and filtration facilities at two designated sites, CUP-3A and CUP-7, located in the northern part of San Mateo County, California (**Figure 1 – Site Location Map**). The proposed wells are part of the Regional Groundwater Storage and Recovery Project (GSR), a project being developed through the coordination of the San Francisco Public Utilities Commission (SFPUC) and local partner agencies (i.e., City of Daly City, City of San Bruno, and Cal Water). We have previously performed geotechnical investigations and submitted geotechnical design reports (GTC, 2009a and 2009b) at ten other sites in northern San Mateo County for the project. This geotechnical report is being prepared for the SFPUC as part of Task Order No. 6 of the design services Contract No. CS-998.B.

Although the CUP-44-2 site was initially proposed along with the CUP-7 site for our geotechnical evaluation, we were subsequently instructed by the SFPUC not to pursue our study of the CUP-44-2 site for this task due to issues pertaining to restrictions on accessibility and building layout. Instead, we have been authorized to evaluate the CUP-3A and CUP-7 sites.

We anticipate that the proposed lightly loaded station buildings will typically be constructed with concrete, although the material selection will depend on the surrounding structures. According to the site location and floor plans developed at the 65 percent design progress in June, 2011 (SFPUC, 2011), a new well station building which houses a production well and related chemical treatment facilities are anticipated at the CUP-3A site. The footprint size of proposed well station building is approximately 1,523 square feet (35 feet by 43½ feet). At the CUP-7 site, the well station fenced enclosure is approximately 576 square feet (18 feet by 32 feet). Other improvements located adjacent to each well station exterior include concrete paving, and a transformer pad. The preliminary layout of the proposed well station buildings and related facilities is shown on **Plates 1 and 2 – Exploration Location Plan**. Geotechnical recommendations for additional improvements such as new pipeline connections and upgrades, which may require additional geotechnical borings, were not part of our scope of work.

Our understanding of the project is based on a site visit on July 26, 2011, discussions with the SFPUC Design Team, preliminary 65 percent progress drawings of the project sites, a review of geotechnical information as referenced in this report, and results from our field exploration and laboratory testing programs. The objectives of our geotechnical study are to: (1) review available geotechnical/geologic information in the site vicinity to understand site conditions; (2) perform a subsurface exploration program to classify subsurface soil types, conduct in-situ soil tests, and collect soil samples for geotechnical laboratory testing; and (3) perform geotechnical engineering analyses to assess potential geo-hazards and develop recommendations for the design and construction of the proposed well station facilities.

FIGURE 1 – SITE LOCATION MAP

WORK PERFORMED

In accordance with our proposal dated January 24, 2011, and subsequent discussions with the SFPUC Design Team, we completed the scope of work described below:

1. **Review of Background Information.** We reviewed available plans, and geotechnical and geologic data for the project sites. Based on our review of existing data, we developed a field exploration program as discussed below.

2. **Field Exploration Program.** We explored subsurface conditions by means of drilling one hollow-stem auger boring at each of the CUP-3A and CUP-7 sites. The exploratory locations for the CUP-3A and CUP-7 sites are shown on **Plates 1 and 2 – Exploration Location Plans**, respectively. Details of our exploration program including the site location and exploration number, method of exploration, date of drilling, existing surface elevation, and bottom depth and elevation are presented for each boring in **Table 1 – Summary of Geotechnical Exploration**. The elevations presented on **Table 1**, and referred to throughout this report, are estimated from the topographic contours on the preliminary 65 percent site plans (SFPUC, 2011) and referenced with respect to 1988 North American Vertical Datum (NAVD88).

   TABLE 1 – SUMMARY OF GEOTECHNICAL EXPLORATION

Site Location and Exploration No.	Method	Exploration Date	Surface Elevation (feet)[1]	Bottom Depth (feet)	Bottom Elevation (feet)[1]
CUP-3A	Stem Auger	8/8/2011	+190	51.4	+139
CUP-7	Stem Auger	8/8/2011	+132	36.3	+96

   1. Surface elevation relative to NAVD88 datum is estimated from the topographic contours on the preliminary 65 percent progress site location plans dated June, 2011 from SFPUC (2011).

   We visually classified the soil during drilling. We recovered split-spoon (Standard Penetration Test) samples and relatively undisturbed 2 ½ inch diameter sleeve samples using a split-barrel sampler. Selected samples were transferred to a laboratory for testing. Boring logs are presented on **Plates A-1.1 and A-1.2** in **Appendix A – Supporting Geotechnical Data**.

3. **Laboratory Testing.** We performed moisture, density, grain size analysis, Atterberg limits, direct shear and corrosion tests on selected soil samples to measure pertinent index and engineering properties. The laboratory test results are presented on the figures in **Appendix A**, and on the boring logs on **Plates A-1.1 and A-1.2**.

4. **Engineering Analysis.** We analyzed subsurface conditions and laboratory test results, and reviewed regional and local geology and seismicity. Based on our evaluation, we provided the following geotechnical recommendations for design:

    - <u>Geologic and seismic hazards</u>: Assessment of hazards associated with fault rupture, strong ground shaking, liquefaction, seismically-induced landslide, lateral spread and tsunami, seismic settlement and differential compaction, and recommendations on mitigation measures, where appropriate; and allowable design parameters for short-term seismic loading.
    - <u>Site response spectra</u>: Evaluated seismic design parameters in accordance with the International Building Code Static Force Procedure (ICC, 2009) as adopted in the 2010 California Building Code (ICC, 2010), and ASCE7-05.
    - <u>Allowable and ultimate bearing capacity</u>: Evaluation of allowable and ultimate soil bearing pressures and modulus of subgrade reaction (vertical soil springs) for the anticipated shallow foundation systems (shallow footings with grade beams, and mat foundations).
    - <u>Anticipated settlements</u>: Assessment of total and differential settlements for shallow foundation systems that are anticipated for the proposed well stations. Development of options for mitigating excessive dynamic settlements.
    - <u>Earthwork recommendations</u>: Development of recommendations for site preparation and grading, excavations, engineered fill (including placement and compaction), structural fill, and pipe trenching, bedding and backfilling; and assessment of the suitability of site-excavated material for re-use as fill or backfill material.
    - <u>Lateral earth pressures</u>: Recommendations of design lateral earth (including active, passive, at-rest, and seismic increment) pressures and coefficient(s) of base sliding friction for unrestrained and restrained retaining walls and/or buried footings.
    - <u>Corrosion recommendations</u>: Discussion of the corrosion test results, identification of on-site soils which may cause corrosion or other deleterious effects to concrete or steel.
    - <u>Construction considerations</u>: Discussion pertaining to geotechnical conditions at the project sites including mitigation of excessive dynamic settlements.
    - <u>Groundwater considerations</u>: Discussion of anticipated groundwater conditions during construction.

5. **Report.** We prepared this report presenting our geotechnical findings, conclusions, and recommendations for the proposed improvements at the GSR project sites.

FINDINGS

SITE CONDITIONS

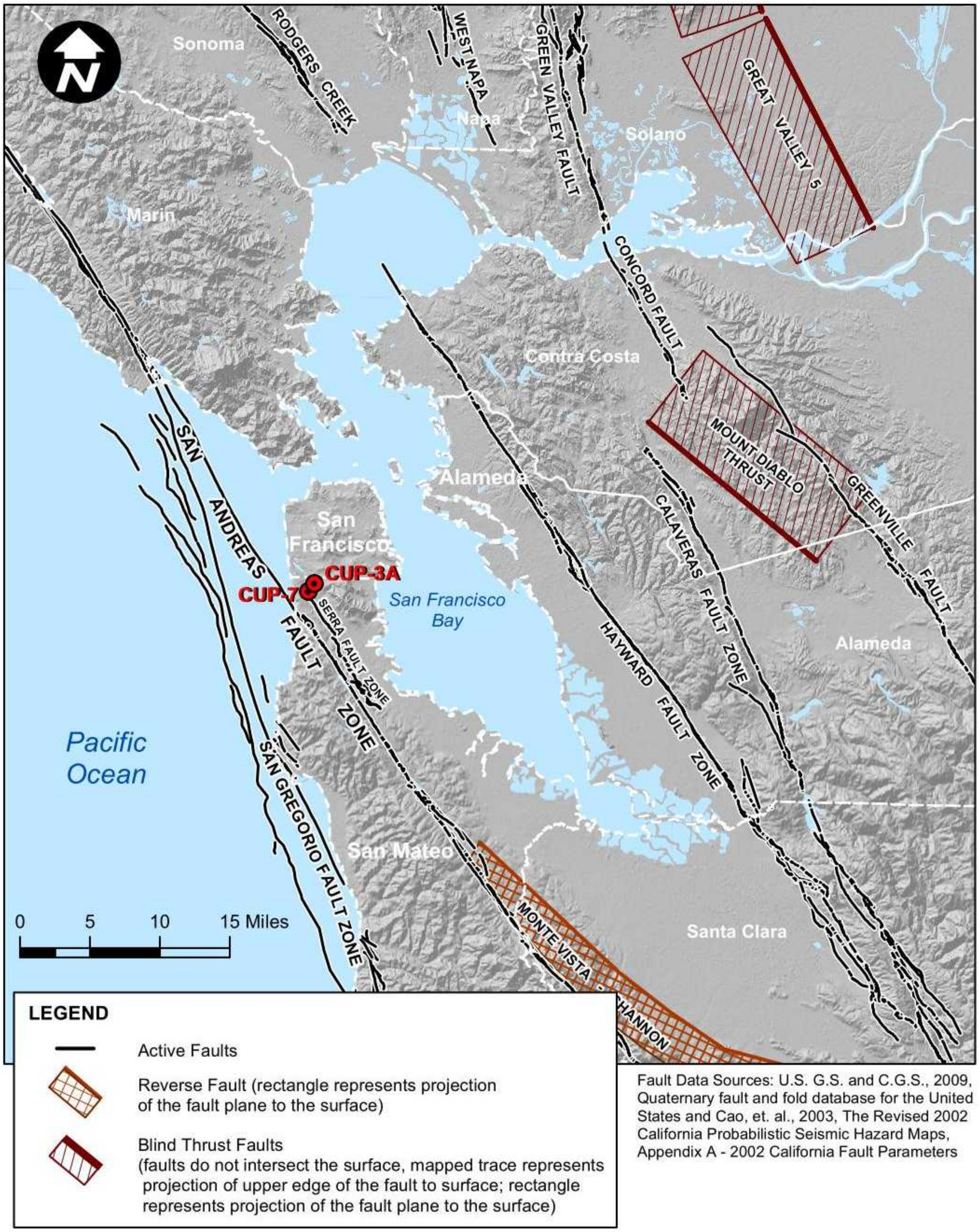
The two GSR project sites are located at northern San Mateo County, California. The CUP-3A site is located within the northeast portion of the Lake Merced Golf Club in Daly City, California, and is surrounded at about 30 feet to the east by Interstate 280 (I-280), and about 100 feet to the north by parking lot of the 45 Poncetta Drive apartment complex. As indicated on **Plate 1**, the CUP-3A site is situated on a relatively flat, unpaved pad that is currently occupied by an existing public restroom and some buried utility lines (including a PG&E gas transmission pipeline and some water main pipelines). About 20 feet to the west from the nearest edge of the proposed well station building at the site, the relatively flat terrain descends about 8 feet on a 3:1 (horizontal:vertical) slope to a paved driveway that separates the project site from a putting green (lawn). The slope appears to be sparsely planted with trees.

The CUP-7 site is located about 160 feet northeast of the intersection between 87th Street and Park Plaza Drive in Broadmoor, California. The project site which is situated on an undeveloped, grassed area is surrounded with Park Plaza Drive to the west, a 10-foot wide paved walkway and residential units to the south, and a sloping terrain to the north and east. As indicated on **Plate 2**, the CUP-7 site is situated on a relatively flat to mildly sloping terrain that descends north-to-northeast along the Park Plaza Drive orientation. From the northeast corner of the proposed well station fenced enclosure at the CUP-7 site, the terrain descends about 20 feet on an approximately 3:1 (horizontal:vertical) slope in a northeast direction toward the track and field of the Garden Village Elementary School. The slope appears to be densely vegetated with low to moderately tall trees and shrubs. The nearest residential unit is located about 50 feet south of the site.

SEISMICITY

The San Francisco Bay Area contains several active faults that could cause strong ground shaking at the project sites. **Figure 2 – Regional Active Fault Map** shows faults in the vicinity of the project sites. The San Andreas Fault Zone (including the 1906 Rupture Event and Peninsula Segment) is the nearest active fault and is located about 0.8 and 1.4 miles from the CUP-7 and CUP-3A sites, respectively. The San Andreas Fault is a primary component in a complex system of right-lateral, strike-slip faults; including the San Andreas, San Gregorio-Seal Cove, Hayward, and Calaveras faults; collectively known as the San Andreas Fault system. The San Andreas, San Gregorio-Seal Cove, Hayward, and Calaveras Faults have produced measurable historic ground motion and movement. The San Andreas Fault is capable of producing an earthquake of an estimated maximum magnitude of M7.9. This segment is estimated to have recurrence intervals on the order of 200 years. A summary of nearby faults is presented in **Table 2 – Active and Potentially Active Faults**.

FIGURE 2 – REGIONAL ACTIVE FAULT MAP



Fault Data Sources: U.S. G.S. and C.G.S., 2009, Quaternary fault and fold database for the United States and Cao, et. al., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, Appendix A - 2002 California Fault Parameters

TABLE 2 – ACTIVE AND POTENTIALLY ACTIVE FAULTS

Fault	Distance to Fault (miles)		Estimated Maximum Earthquake Magnitude (1)	Historic Earthquakes (2)	
	CUP-3A	CUP-7		Year	Magnitude
San Andreas - 1906 Rupture Section	1.4 (3)	0.8 (3)	7.9 (3)	1838 1898 1906 1989	6.8 6.2 8.1 7.1
San Andreas – Peninsula Section	1.4	0.8	7.1		
San Andreas – North Section	8.0	8.2	7.6		
San Gregorio-Seal Cove – North Section	5.8	5.2	7.3	N.A.	N.A.
Hayward- North Section	16	16	6.9	1868	6.8
Hayward – South Section	18	18	6.9		
Monte Vista-Shannon	20	20	6.8	n.a.	n.a.
Calaveras – North Section	26	26	6.8	1861 1955 1979 1984 2007	5.3 5.5 5.9 6.1 5.4
Calaveras – South Section	40	40	6.2		

(1) Maximum Moment Magnitude based on California Geological Survey (CGS) fault parameters as updated in 2002 (Cao, et al., 2003), or as suggested by the SFPUC's General Seismic Requirements (SFPUC, 2006).
(2) Historic earthquakes listed may have occurred on any one of the listed sections of the associated fault; n.a. (not applicable) indicates that no significant historic earthquakes have occurred on this fault or fault section.
(3) The 1906 rupture event assumes rupture along the North Coast, Peninsula and Santa Cruz Mountains sections to San Juan Bautista. Maximum magnitude is based on the average 5 m displacement during the 1906 event (WGCEP, 2003; Petersen, et al., 1996). Site-to-fault distances are based on the USGS 2008 updated National Seismic Hazard Mapping Program (Petersen et al., 2008) and interactive de-aggregation at URL https://geohazards.usgs.gov/deaggint/2008/.

GEOLOGY

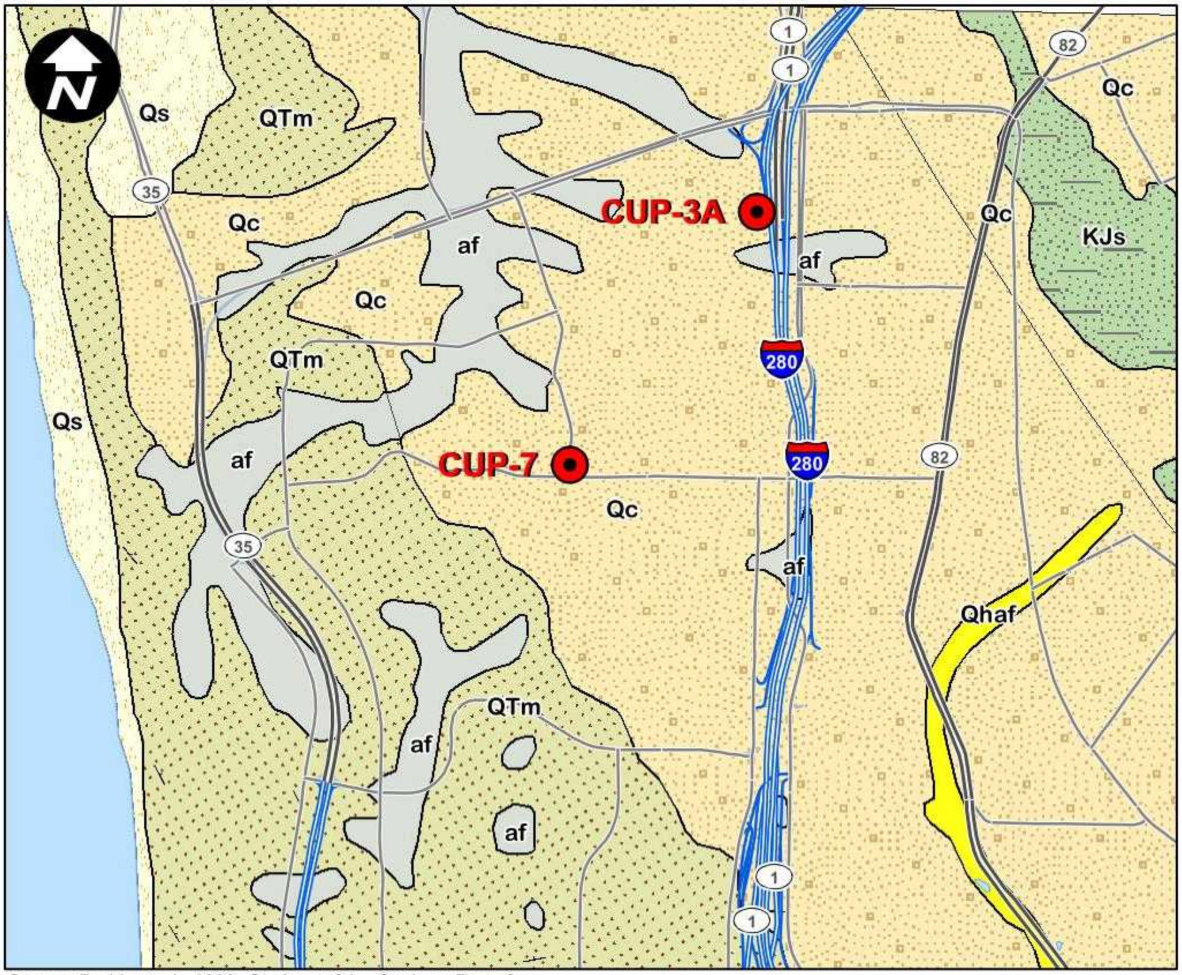
The San Francisco Bay Area is located within the Coast Ranges Geomorphic Province. Past episodes of tectonism have folded and faulted the bedrock, creating the regional topography of the northwest trending ridges and valleys characteristic of the Coast Ranges Geomorphic Province. The San Francisco Bay and vicinity occupy a structurally controlled basin within the province. Late Pleistocene and Holocene sediments (less than 1 million years old) were deposited in the basin as it subsided.

The two project sites are located in areas mapped as Colma Formation (Brabb, et al., 1988). Other sedimentary deposits mapped in close proximity to the sites include Merced

Formation, Sand Dune and Beach Deposits, and Unnamed Sandstone. A layer of artificial fill was encountered at each site. The geology in the project vicinity is shown on **Figure 3 – Regional Geologic Map**. Based on a regional geologic study as compiled as a regional geologic cross section of the Westside Basin – Lake Merced (SFPUC, 2008), the Franciscan Complex bedrock is anticipated to be on the order of 600 to 700 feet below ground surface at the sites. Geologic maps (Brabb, et al., 1998) describe the geologic units at and near each boring as follows:

- **af:** Artificial fill (Historic) – loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations and thicknesses which may exceed 30 m; some compacted and quite firm, but fill made before 1965 is nearly everywhere not compacted and consists of simply dumped materials.
- **Qs:** Sand Dune and Beach Deposits (Holocene) – predominantly loose, medium- to coarse-grained, well-sorted sand but also includes pebbles, cobbles, and silt; thickness is typically less than 6 m in most places, but in other places may exceed 30 m.
- **Qc:** Colma Formation (Pleistocene) – yellowish-gray, gray, yellowish-orange and red-brown, friable to loose, fine- to medium-grained arkosic sand with subordinate gravel, silt and clay; total thickness is typically unknown, but may up to 60 m.
- **QTm:** Merced Formation (lower Pleistocene and upper Pliocene) – medium gray to yellowish gray, yellowish orange, medium- to very fine-grained, poorly indurated to friable sandstone, siltstone, and claystone, with some conglomerate lenses and a few friable beds of white volcanic ash; sandstone is typically silty, clayey, or conglomeratic; fossiliferous conglomerate is well cemented.
- **KJs:** Unnamed Sandstone (Cretaceous or Jurrasic) – dark gray to yellowish brown greywacke interbedded with shale in approximately equal amounts; unit resembles some Franciscan greywacke (fs) but bedding is better developed herein; the unit is exposed in San Bruno Mountain, where it is about 1,000 m thick.

FIGURE 3 – REGIONAL GEOLOGIC MAP



Source: Brabb et. al., 1998, Geology of the Onshore Part of San Mateo County, Califonria, USGS Digital Database OFR 98-137.

0 1,000 2,000 3,000 4,000 Feet

LEGEND

Geologic Units

Historic

- af — Artificial fill

Holocene

- Qhaf — Alluvial fan and fluvial deposits
- Qcl — Sand dune and beach deposits

Pleistocene

- Qc — Colma Formation

Pleistocene to Pliocene

- QTm — Merced Formation

Cretaceous to Jurassic

- KJs — Unnamed sandstone

Structual Features

- geologic contact

EARTH MATERIALS

The exploration for this investigation encountered artificial fill (af) which was underlain by Colma Formation (Qc). The artificial fill represents disturbed soil and fill materials previously placed during site grading at the project sites. The exploratory locations are shown on **Plates 1 and 2**.

Artificial Fill (af). Artificial fill consisting of medium dense, poorly grade fine grained sand with silt was encountered to a depth of about 8 feet in boring CUP-7. The grade at the Garden Village Elementary School track and field is located about 20 feet below the CUP-7 site. The origin of fill at the site was likely derived from grading and reuse of on-site, near surface materials of Colma Formation (Qc).

At boring CUP-3A, artificial fill consisted of an upper 20 feet of loose to dense, poorly graded fine sand with silt, and a remainder 11 feet of dense, silty fine sand. Judging from distinctly lower density and less fines content, the upper 20 feet of looser materials may likely have been derived from more recent activities such as, grading and reuse of on-site, near surface artificial fill around the Lake Merced Golf Course, and construction of an elevated pad for the existing public restroom building. In comparison to the upper fill, the lower stratum of fill with higher density and higher fines content are closer in resemblance to the engineering properties of the underlying Colma Formation.

At the project sites, measured total unit weights ranged from 101 to 113 pounds per cubic foot (pcf) and moisture contents ranged from 4 to 12 percent.

Colma Formation. Soils of the Colma Formation (Qc) were encountered below the artificial fill at the two project sites. The Colma Formation soils consisted predominantly of yellowish, reddish and grayish brown, dense to very dense, silty fine grained sand with oxide staining. An isolated layer of medium dense, silty fine sand was observed within the upper portion of the Colma Formation at CUP-3A. Colma Formation soils at the two sites extended to the total depth of exploration (36.3 to 51.4 feet). A moisture content ranging from 9 to 18 percent was measured in the Colma Formation soils at the two sites.

GROUNDWATER

Groundwater was not encountered during auger drilling of the two exploratory borings CUP-3A and CUP-7. Groundwater levels are likely to be influenced by seasonal variations in precipitation, percolations from storm water runoff and local irrigation, groundwater pumping and other factors, and are therefore expected to fluctuate considerably from the observed groundwater levels.

CONCLUSIONS AND RECOMMENDATIONS

1.0 GENERAL

The following sections provide our conclusions and recommendations for evaluation and design of the proposed well station buildings at two sites of CUP-3A and CUP-7. According to preliminary 65 percent drawings (SFPUC, 2011), proposed improvements at CUP-3A consist of a well station building that houses facilities such as, a production well and chemical treatment equipment, concrete paving, and transformer pad. Proposed improvements at CUP-7 consist of a fenced pad with a production well and electrical equipment. Based on findings from our geotechnical field investigation, the project sites are underlain by artificial fill (af) and Colma Formation (Qc).

We consider the proposed improvements to be geotechnically feasible, provided that our geotechnical recommendations are incorporated into design and construction documents.

2.0 SEISMIC DESIGN CONSIDERATIONS

2.1 **General.** The main seismic hazards at the site are expected to be strong ground shaking and seismic settlement and differential compaction within the loose to medium dense portion of fill and upper Colma Formation. Our seismic design considerations, including fault rupture, ground shaking, liquefaction, seismic settlement and dynamic (differential compaction) settlement, inundation by tsunamis, seismically-induced lateral spreading, and seismic design with respect to the 2009 International Building Code (which the 2010 California Building Code has adopted) and ASCE7-05 are provided in the following sections.

2.2 **Fault Rupture.** No active or potentially active faults are known to cross the subject sites. Consequently, the hazard posed by ground rupture due to fault offset is considered to be negligible.

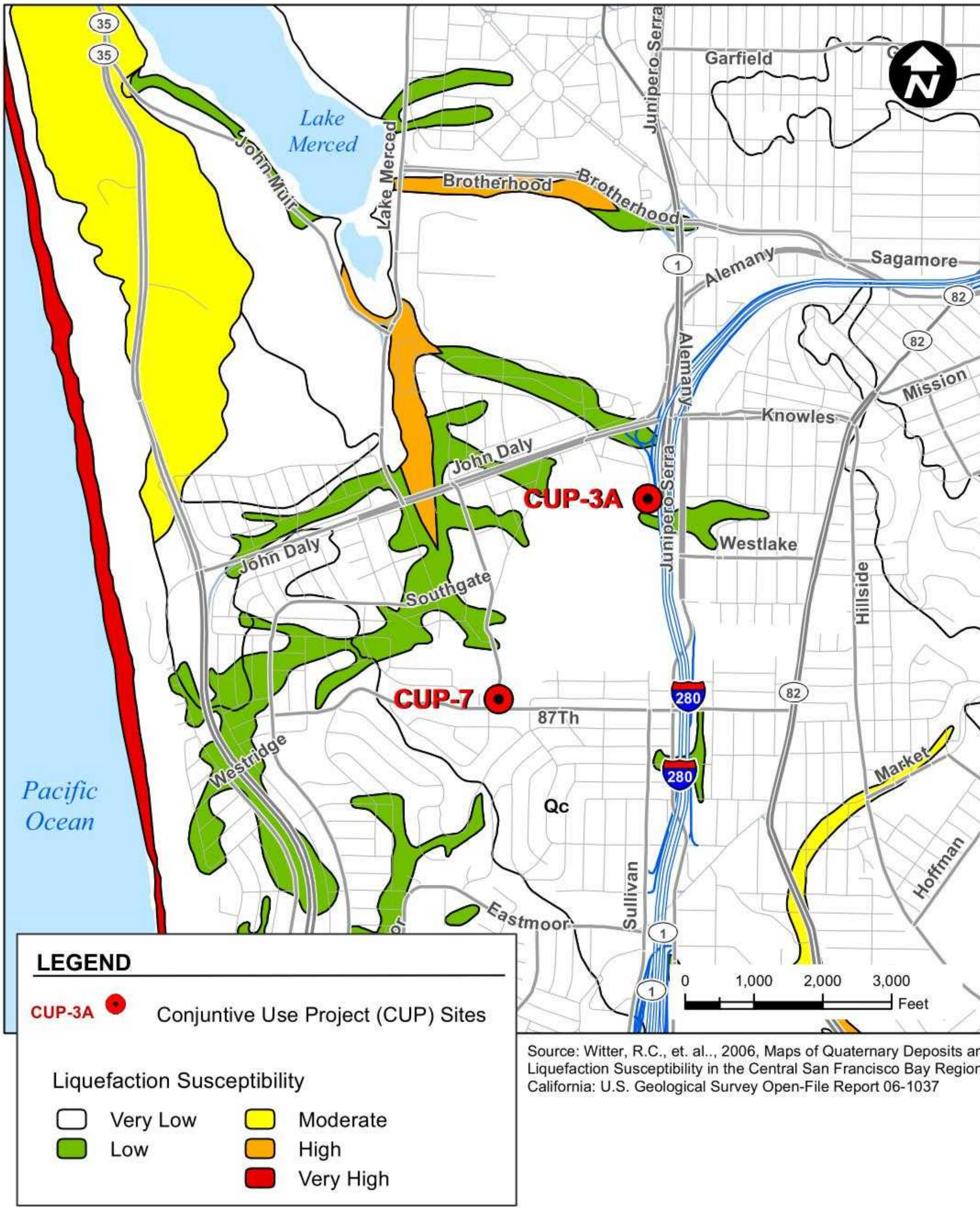
2.3 **Ground Shaking**. Strong ground shaking will occur at the site as a result of a moderate to large earthquake occurring on one of the active regional faults. The San Andreas Fault is closest to the sites at about 0.8 and 1.4 miles to the southwest from CUP-7 and CUP-3A sites, respectively. Based on de-aggregation of seismic sources from the probabilistic seismic hazard analysis (USGS, 2008), the Northern San Andreas Fault and San Gregorio-Seal Cove Fault segments of the San Andreas Fault system are the only individual fault segments that each contributes more than 2 percent to the overall mean hazard at various spectral periods from 0 to 5 seconds. Therefore, the San Andreas Fault system has the greatest capability of causing strong ground motions. Of the two segments of the San Andreas Fault system, the Northern San Andreas Fault segment with an event magnitude M7.9 and shorter source-to-side distances of 0.8 to 1.4 miles is the dominant event relative to the smaller event magnitude M7.3 at longer source-to-site distances of 5.2 to 5.8 miles for the San Gregorio-Seal Cove Fault segment.

The California Geological Survey (CGS, formerly known as California Division of Mines and Geology) and United States Geological Survey (USGS) completed probabilistic seismic hazard maps in 1996 (Petersen et al., 1996), and subsequently updated fault parameters and revised the maps in 2002 (Cao, et al., 2003, and WGCEP, 2003) and 2008 (Petersen, et al, 2008, and WGCEP, 2008). USGS provides a web-based program to evaluate the USGS Probabilistic Uniform Hazard Response Spectra (http://earthquake.usgs.gov/research/hazmaps/design). Based on the 2008 USGS update, the peak ground acceleration (PGA) at a 975-year return period (an earthquake event having a 5 percent probability of exceedance in 50 years) is estimated to be 0.82g and 0.87g for the CUP-3A and CUP-7 sites, respectively. PGA at the Maximum Credible Earthquake (MCE) level for the two sites are controlled by the dominant event of the Northern San Andreas Fault segment with a magnitude M7.9 and R0.8 to R1.4 miles, as discussed above and based on seismic de-aggregation of the PSHA (USGS, 2008). To evaluate PGA at the MCE level, the 2008 Next Generation Attenuation (NGA08) method (EERI, 2008) provides estimated PGA of 0.80g and 0.84g which correspond to the upper limits at the 84th percentile deterministic level (median plus one standard deviation) for the dominant earthquake event. For this study, PGA corresponding to 0.80g and 0.84g are used for geotechnical earthquake engineering evaluation at the CUP-3A and CUP-7 sites, respectively.

2.4 Liquefaction and Dynamic Settlement. Liquefaction is a phenomenon wherein a temporary, partial loss of shear strength occurs in a soil due to increases in pore pressure that result from cyclic loading during earthquakes. Saturated, loose to medium dense sands and silty sands are most susceptible to liquefaction. Consequences of liquefaction can include ground settlements, foundation failure, sand boils, and lateral spreading. Dynamic settlement is the densification of saturated and unsaturated soils during strong ground shaking. All soil types are prone to dynamic settlement, though loose, sand and silty sand are most susceptible.

Liquefaction: The liquefaction susceptibility, as mapped by Witter et al. (2006), is illustrated on **Figure 4 – Liquefaction Susceptibility Map**. As can be seen from the figure, the CUP-3A site lies within a zone mapped as having very low to low liquefaction susceptibility. A zone of very low liquefaction susceptibility is mapped for the CUP-7 site. Because of the regional focus of the liquefaction susceptibility mapping, the data only generally correlates with areas of known liquefaction hazard. The site-specific data from the borings is considered to be more indicative of liquefaction and dynamic settlement hazard. The following paragraphs further describe this hazard based on our subsurface investigation and laboratory testing program.

FIGURE 4 – LIQUEFACTION SUSCEPTIBILITY MAP



Source: Witter, R.C., et. al.., 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California: U.S. Geological Survey Open-File Report 06-1037

Due to the absence of groundwater within the total exploration depths of about 36 to 51 feet at the two project sites and material density that generally increases with depth, liquefaction is not considered to be a significant consideration for the Colma Formation below these depths. As discussed earlier in this report, groundwater levels are likely to be influenced by rainfall and storm water runoff, and are expected to fluctuate considerably from the observed groundwater levels. Hence, liquefaction susceptibility has to be considered for higher groundwater conditions as recommended in Section 3. In evaluating liquefaction susceptibility of the materials explored from the borings at the project sites, we have conservatively assumed groundwater levels of 20 feet at CUP-3A, and 10 feet at CUP-7. Below an assumed groundwater level of 10 feet, the dense to very dense silty sand of the Colma Formation encountered in boring CUP-7 is not susceptible to liquefaction. The dense silty sand of the artificial fill encountered below an assumed groundwater level of 20 feet in boring CUP-3A is also not susceptible to liquefaction. An isolated layer/pocket of medium dense silty sand within the upper portion the Colma Formation at a depth of about 35 feet is not considered to pose significant risk of seismic induced reconsolidation settlement to the site. Volumetric reconsolidation settlement is not considered to be significant for the soil below a groundwater depth of 10 feet in boring CUP-7. Results from our liquefaction analysis are presented on **Table 3 – Summary of Dynamic Settlements**.

Our liquefaction analysis has been conducted using the Simplified Cyclic Stress Ratio module within the SHAKE2000 computer program for one-dimensional analysis of geotechnical earthquake engineering problems (Geomotions, 2011). Detailed information regarding the analysis methods can be found in the following references: Cetin and Seed (2000 and 2004), Cetin et al. (2004), Moss et al. (2006), Seed et al. (1985 and 2003), Seed and Idriss (1971), and Youd et al. (2001 and 2003).

Dynamic Settlement of Dry Sand: Seismically induced dynamic settlements at CUP-3A are estimated at 4 inches, due to the presence of up to 20 feet of unsaturated, loose to medium dense fill sand near the surface. At CUP-7, such dynamic settlements are estimated at ¾ inch. Differential settlements (over a distance of 80 feet) are estimated to be 1 inch at CUP-3A and ¼ inch at CUP-7. Differential settlements can be linearly interpolated from these estimated values when the dimensions (distances) of the proposed improvement footprint are less than 80 feet. Results of our dynamic settlements of dry sands are presented on **Table 3 – Summary of Dynamic Settlements**.

Our evaluation of dynamic differential compaction settlement of unsaturated sand has been conducted in conjunction with liquefaction analysis using the Simplified Cyclic Stress Ratio module within the SHAKE2000 computer program for one-dimensional analysis of geotechnical earthquake engineering problems (Geomotions, 2011). For unsaturated sand layers, the volumetric strains for a site-specific dominant earthquake magnitude other than the reference magnitude M7.5 are calculated by multiplying the site-specific volumetric strains with correction factors as recommended by Tokimatsu and Seed (1987). These adjusted volumetric strains are doubled to account for the effects from multi-directional shaking. Detailed information regarding the calculation method can be found in the above references.

<u>*Total Seismic Settlement*</u>: Total seismic settlement is the cumulative of volumetric reconsolidation settlement of saturated sand due to liquefaction and dynamic settlement of dry sand. Since volumetric reconsolidation settlement due to liquefaction is not considered as likely to occur at the two project sites, the total seismic settlement is equivalent to the dynamic settlement of dry sand. The results indicate the propensity for dynamic (compaction) settlement of dry sand is similar for the two groundwater conditions. Results of total and differential dynamic settlements are presented on **Table 3 – Summary of Dynamic Settlements**.

In addition to the estimated seismic settlements presented above, pockets of loose unsaturated granular soil which may be encountered during subgrade preparation should be removed to reduce potential for uneven seismic densification. Based on our evaluation, the hazard posed by differential settlement due to dynamic settlement resulting from liquefaction of saturated sand and dynamic settlement of unsaturated sand is considered to be moderate for CUP-3A and low for CUP-7. Measures for mitigating excessive seismically induced settlements for the project sites are addressed in **Section 6**.

TABLE 3 – SUMMARY OF DYNAMIC SETTLEMENTS

	CUP-3A		CUP-7	
	Groundwater Depth		Groundwater Depth	
	20 feet	50 feet	10 feet	50 feet
Volumetric Reconsolidation (inches)	0	--(1)	0	--(1)
Dynamic Dry Sand Settlement (inches)	4	4	½	¾
Total Dynamic Settlement (inches)	4	4	½	¾
Differential Dynamic Settlement (inches) (2)	1	1	¼	¼

1. Liquefaction does not occur in unsaturated soil above the lower groundwater depth of 50 feet.
2. Differential dynamic settlements can be linearly interpolated from these estimated values when the dimensions (distances) of the proposed improvement footprint are less than 80 feet.

2.5 Inundation by Tsunamis. While tsunamis can be triggered by various sources such as an earthquake, a landslide, a volcanic eruption, or even a large meteor crashing into the ocean, the most common trigger is related to a large, submarine earthquake that creates a significant upward movement of the sea floor to result in a rise of water at the ocean surface (CGS, 2009). As the mound of water, which can travel up to 500 miles per hour in the open ocean, approaches the shoreline, it slows down to about 30 miles per hour and builds up significantly in amplitude (height). Hence, a tsunami hazard mitigation program which includes emergency preparedness and evacuation is essential to areas that have been identified as potentially susceptible to inundation from tsunami.

The project sites are not mapped within areas that are potentially susceptible to tsunami inundation (CalEMA, 2009). Given that the project site elevations are well above the Mean Sea Level (MSL) and they are located at distances in excess of one mile from the nearest Pacific Ocean coastal area to the west, the project sites are not considered to be potentially susceptible to inundation from tsunami.

2.6 **Seismically-Induced Landsliding and Lateral Spreading.** Although an embankment (about 8-foot high, descending on an about 3:1 slope) is located about 20 feet to the west from the nearest edge of the proposed well station building at the CUP-3A site, the potential susceptibility of the site to lateral spreading toward the embankment free face is considered low because the isolated layer of potentially liquefiable medium dense within the Colma Formation at a depth of 35 feet is located well below the toe of the 8-foot tall embankment.

At the CUP-7 site, the terrain can be characterized as mildly sloping (descending about 13:1) along the Park Plaza Drive, and an embankment (about 20-foot high) that descends on an about 3:1 slope from the northeast corner of the proposed building footprint to the Jefferson Elementary School track and field. The potential susceptibility of the CUP-7 site to lateral spreading is considered to be low because Colma Formation soil at this site is not susceptible to liquefaction.

An evaluation of static stability of the slopes at the CUP-3A and CUP-7 sites using the method of stability charts by Janbu (USACE, 2003) indicates stable slopes with factors of safety (FOS) in excess of 2. Roots from vegetation/shrubs and low to moderately tall trees along the slopes at the two project sites provide additional strengthening of the near surface soil mass and may reduce the potential for surficial sloughing. A confluence of the above factors suggests that the potential for seismically-induced instability of the slope (including landsliding and lateral spreading) is considered to be low at the two project sites.

2.7 **Seismic Design Parameters.** The proposed improvements may be designed in accordance with the International Building Code Static Force Procedure (ICC, 2009) as adopted in the 2010 California Building Code (ICC, 2010) using the seismic parameters presented in **Table 4 – Seismic Design Parameters**. Based on our exploration, a Site Class D has been designated for the CUP-3A site, and a Site Class C for CUP-7. The seismic design parameters have been developed for the ASCE7-05 Maximum Considered Earthquake using the Earthquake Ground Motion Parameters Application (Version 5.1.0) as developed by the USGS (2011).

TABLE 4 – SEISMIC DESIGN PARAMETERS

	CUP-3A	**CUP-7**
Mapped Spectral Acceleration		
S_s at 0.2-second	2.096	0.875
S_1 at 1-second	1.149	2.186
Site Adjustment Factor		
Site Class	D	C
Site Coefficient F_a	1.0	1.0
Site Coefficient F_v	1.5	1.3
Site Adjusted Spectral Acceleration		
SMs = F_a x S_s	2.096	2.186
SM1 = F_v x S_1	1.724	1.607
Design Spectral Acceleration		
SDs = 2/3 x SMs	1.397	1.457
SD1 = 2/3 x SM1	1.149	1.071

3.0 GROUNDWATER

Groundwater was not encountered during drilling at the two CUP-3A and CUP-7 borings. Groundwater levels are influenced by seasonal variations in precipitation, percolations from storm water runoff and local irrigation, groundwater pumping and other factors, and are therefore, subject to variation. To account for seasonal variations, we recommend conservative design groundwater levels for structural design purposes as presented in **Table 5 – Recommended Design Groundwater Levels**.

Groundwater related design issues such as hydrostatic pressures on shoring elements (if implemented), excavation dewatering, and hydrostatic uplift pressures on the proposed buildings are not anticipated for excavations less than 5 feet below the ground surface. For excavations exceeding the design groundwater depths, the contractor should anticipate groundwater inflow that may require dewatering. For intermediate excavations between 5 feet and the design groundwater depths, the contractor should anticipate the possibility of inflow of groundwater seepage.

TABLE 5 – RECOMMENDED DESIGN GROUNDWATER LEVELS

Proposed Site Location	**Design Groundwater Depth (feet)**
CUP-3A	20
CUP-7	10

4.0 EARTHWORK

4.1 **General.** Given the earth materials on the project site encountered during our exploration, the contractor should be able to carry out planned excavations using conventional heavy equipment.

Evaluation of the presence, or absence, and treatment of hazardous materials was not part of this study. If hazardous materials are encountered during excavation, proper handling and treatment during construction will depend on the contaminant type, concentration, and volatility of the contaminated materials.

General geotechnical considerations for site preparation, excavations, temporary shoring and bracing, engineered fill material, engineered fill placement and compaction, pipe bedding, and utility trench backfill are presented in the following sections.

4.2 **Site Preparation.** Site preparation will consist of demolition, excavation and removal of on-site materials such as pavement, concrete, abandoned utilities, and miscellaneous debris in preparation for the foundation excavations. Any creation of holes from the removal of such materials should be backfilled with engineered fill. Recommendations for engineered fill are provided in Sections 4.5 and 4.6. Also as part of site preparation, the location of active underground utilities should be determined and, if affected by construction activities, should be relocated or protected.

4.3 **Excavations.** We anticipate that excavations for the planned building improvements to extend up to no more than a few feet below existing ground elevation. Shallow excavations for the proposed facilities will allow for unshored excavations with adequately sloped sidewalls. Vertically shored walls or braced excavations are anticipated where space constraints may not allow for open, sloped excavations. At a minimum, excavations should be constructed in accordance with the current California Occupational Safety and Health Administration (OSHA) regulations (Title 8, California Code of Regulations) pertaining to excavations. Temporary cut slopes are expected to be stable for configurations described in Title 8 for Type C soils and where unsupported should be cut back no steeper than 1 ½ horizontal to 1 vertical. All excavations should be closely monitored during construction to detect any evidence of instability.

Care should be taken when excavating near existing utilities and pipelines. Excavations can undermine support of adjacent existing pipelines and other subsurface structures. We recommend that some form of vertical shoring system be considered for excavated sidewalls that are adjacent to existing pipelines or other known buried adjacent structures.

Some of the near surface loose soils at the project sites will likely be removed during excavation for the proposed improvements. If any footings are founded above loose or soft soils, overexcavation of loose or soft soils and replacement with engineered fill may be required.

4.4 **Temporary Shoring and Bracing.** The type and design of the shoring will depend on the depth of excavation and excavation-bracing sequence. The shoring and bracing design and installation should be the responsibility of the construction contractor. As a general guideline, construction procedures, excavations, and design and construction of any temporary shoring should comply with the current OSHA Title 8 regulations pertaining to excavations. The shoring and bracing should accommodate surcharge loads that may be imposed by adjacent structures, traffic, or construction activities.

Possible shoring schemes include soldier pile and lagging and steel sheeting, both of which may include internal bracing struts to limit lateral deflections. Such braced and shored excavations will be subjected to lateral earth pressures. Recommended active, at-rest, and passive lateral earth pressures are provided in Section 5.

Horizontal and vertical movements of the ground are possible in the vicinity of the excavations. These movements can generally be reduced to acceptable levels by use of a properly designed and constructed shoring system. Measures should be taken to prevent the loss of sand through the gaps in the shoring or lagging.

4.5 **Engineered Fill Material.** Material for engineered fill should be inorganic, well graded, free of rocks or clods greater than 4 inches in greatest dimension or any other deleterious materials, and have a low potential for expansion. The material should have a liquid limit less than 35, a plasticity index less than 15 and no more than 25 percent passing the No. 200 sieve. Existing on-site soil may be re-used as engineered fill provided it meets the above criteria.

4.6 **Engineered Fill Placement and Compaction.** Engineered fill consisting of existing on-site fill which meets the requirements above should be placed in layers no greater than 8 inches in un-compacted thickness, conditioned with water or allowed to dry to achieve moisture content near optimum, then mechanically compacted to at least 90 percent relative compaction based on ASTM D1557. All engineered fill placed to support footings and the upper 6 inches of engineered fill supporting slabs-on-grade should be mechanically compacted to at least 95 percent relative compaction as determined by ASTM D1557. All compaction should be performed using mechanical compaction means; flooding or jetting should not be used as a means to achieve compaction. The ASTM D1557 laboratory compaction tests should be performed at the time of construction to provide a proper basis for compaction control.

4.7 **Structural Backfill.** Structures extending below grade should be backfilled with structural fill to a minimum width of two feet beyond the foundation footprint. Structural backfill should meet the following gradation:

Sieve Size	Percent Passing
3 inches	100
1½ inches	80 to 100
#4	50 to 100
#16	40 to 90
#50	10 to 60
#200	0 to 10

Backfill should be moisture conditioned to within two percent above optimum, placed in layers not exceeding 8 inches in uncompacted uniform thickness, and mechanically compacted to 90 percent relative compaction per ASTM D1557.

4.8 **Pipe Bedding for Small Diameter Pipes.** Pipe bedding should consist of well-graded sand or a sand-gravel mixture. Maximum gravel size should be ½ inch and the bedding material should have less than 12 percent passing the No. 200 sieve. Uniformly graded material such as pea gravel should not be used as pipe bedding material. Pipe bedding should have a minimum thickness of 6 inches beneath the pipe and 6 inches above the pipe. If soft or otherwise unsuitable soils are exposed in the bottom of the trench excavation, the necessity of over-excavation should be evaluated by the project geotechnical engineer. All pipe bedding should be placed to achieve uniform contact with the pipe and a minimum relative compaction of 90 percent per ASTM D1557.

4.9 **Utility Trench / Pipe Backfill.** Utility and pipe trenches may be backfilled above the pipe zone with excavated on-site soils, provided they meet the gradation requirements of engineered fill. The backfill material should be placed in layers no greater than 8 inches in uncompacted thickness, moisture conditioned or allowed to dry to achieve a moisture content near optimum, then mechanically compacted to at least 90 percent relative compaction based on ASTM D1557. The upper 2 feet should be compacted to at least 95 percent relative compaction in areas where structural or traffic loads are anticipated.

5.0 LATERAL EARTH PRESSURES

General. Structural components that extend below ground surface, such as concrete vaults, below-grade walls, and the sides of shallow foundations, will experience lateral earth pressure from the soil and hydrostatic pressure from any existing groundwater. Recommendations for the active, at-rest, passive, and seismic earth pressures, and coefficient of base friction to resist active and at-rest loads are summarized on **Table 6 – Lateral Earth Pressures**, and discussed in the following sections. Because the anticipated excavations will be limited to a depth not exceeding about 5 feet, and the design groundwater level is expected to be below 5 feet, hydrostatic pressures have not been considered.

Active Earth Pressure. Active earth pressures are imposed by the soil on below-grade structures that are unrestrained so that the top of the wall is free to translate or rotate at least 0.004H, where H is the height of the wall. The active earth pressure may be calculated using a design equivalent fluid pressure (EFP) of 40 pcf at the project sites.

At-Rest Earth Pressure. At-rest pressures should be used for design of below-grade structures that are restrained such that the greater deflections that are mobilized to develop the lesser active earth pressures cannot occur (or are undesirable). The at-rest earth pressures may be calculated using a design EFP of 60 pcf at the project sites.

Seismic Earth Pressure. In addition to the active and at-rest pressures, below-grade structures should be designed to consider additional earth pressures due to earthquake loading. The increment in earth pressure due to seismic loading, for both restrained and unrestrained below-grade structures, may be calculated using an inverted triangular distribution with the pressure at the top of the below-grade structures equal to a design earth pressure (EP) of 35H at the project sites, wherein H is the height of the buried structure in feet, and diminishes linearly with depth to zero at the base of the buried structure.

Passive Earth Pressure. Lateral loads can be resisted by passive pressures that develop against the sides of below-grade structures. The passive pressure depends on the lateral displacement of the wall or footing. In accordance with FEMA 356 (FEMA, 2000), the ultimate passive pressure is mobilized at a displacement of approximately 6 percent of the wall height. The ultimate passive earth pressure may be calculated using a design EFP that corresponds to the ultimate EFP as long as the structure can be mobilized to such level of displacement and still does not exceed the allowable displacement of the structure. Oftentimes, the displacement to achieve ultimate passive earth pressures exceeds the allowable displacement of the structure. Consequently, a design EFP needs to be reduced when the allowable displacement of the structure is less than 6 percent of the wall height. For displacements of approximately 0.8 and 3 percent of the wall height, the design EFP may be reduced to 50 and 85 percent of the ultimate EFP. Passive pressures computed using these design EFPs may be combined with the base friction mobilized at the concrete-soil interface to resist lateral loading. Passive earth pressures at the project sites may be computed using the design EFP of 400, 340 and 200 pcf for allowable wall displacements of about 6, 3 and 0.8 percent of wall height, respectively.

Base Friction. A coefficient of friction of 0.4 may be used for estimating the resistance due to base friction at the project sites. The coefficient should be multiplied by the dead load only. The passive earth pressure and base friction mobilized at the concrete-subgrade interface may be combined to resist lateral loading.

TABLE 6 – LATERAL EARTH PRESSURES

Lateral Pressures and Base Friction	CUP-3A	CUP-7
Active Equivalent Earth Pressure (pcf)	40	40
At-Rest Equivalent Earth Pressure (pcf)	60	60 pcf
Seismic Active Earth Pressure[2] (pcf)	35H [2,3]	35H [2,3]
Passive Equivalent Earth Pressure:		
Allowable Displacement 0.06 H[3] (psf)	400	400
Allowable Displacement 0.03 H[3] (psf)	340	340
Allowable Displacement 0.008 H[3] (psf)	200	200
Base Friction Factor	0.4	0.4

1. No hydrostatic effect assuming structural embedment remains above a depth of 5 feet.
2. The seismically induced active earth pressure increment should be applied to the wall as an inverted triangular distribution that decreases linearly from the top to zero at the bottom.
3. H is buried structure height relative to the finished exterior grade adjacent to the buried structure.

6.0 FOUNDATIONS

6.1 Subgrade Preparation. Subgrades to new shallow and deep foundations for the proposed structures should be prepared to provide a flat, relatively dry, and firm working surface. If any unsuitable materials, such as, soft clays or silts, soils containing organic material, debris or other deleterious materials are encountered at subgrade, they should be over-excavated and restored to grade with engineered fill in accordance with Sections 4.5 and 4.6. The fill soils encountered in our exploratory borings were suitable for support of the proposed improvements provided the upper 12 inches are scarified, moisture conditioned, and recompacted. We recommend that the upper 12 inches of subgrade be scarified, moisture conditioned to near optimum moisture content, and compacted in accordance with Sections 4.5 and 4.6. The subgrade should be free of loose debris and ponded water prior to placing reinforcing steel and concrete.

Although long term consolidation settlement is considered minor due to the granular nature of the fill materials, dynamic settlements of loose to medium dense granular soils at CUP-3A and CUP-7 are anticipated during an earthquake event if these sites are not mitigated. Estimates of dynamic settlement at each site are provided in Section 2.4 and Table 3. Special mitigation measures against dynamic settlement at two project sites require additional over-excavation of artificial fill materials below any foundations. This over-excavation must extend at least three feet below proposed footing elevation. Engineered fill shall then be placed, moisture treated to near optimum water content and mechanically compacted to 95 percent relative compaction as determined by ASTM D1557.

6.2 Shallow Foundation Alternatives. A shallow foundation system is suitable for support of the proposed improvements at the CUP-7 site as long as recommendations in Section 6.1 are incorporated into design. Alternatives for shallow foundation systems include grade beams / shallow footings, mat foundations, and post-tensioned foundations. Since a significant dynamic settlement on the order of 4 inches anticipated at the CUP-3A site is due to the loose sandy fill in the upper 20 feet, ground improvement may be needed at this site for a shallow foundation system. Ground improvement strategies such as, in situ densification methods of Geopiers and Rapid Impact Compaction, may not be very feasible because: 1) they may be cost prohibitive due to a significant treatment depth of at about 20 feet; and 2) they may generate vibration related impacts to adjacent structures during construction. Earthwork grading to excavate and recompact the upper 5 feet of loose fill beneath the proposed building footprint at CUP-3A is more appropriate from a cost standpoint in reducing the differential settlement from 1 inch to ¼ inch (and total settlement from 4 inches to 1 inch). Other alternatives to overexcavation and recompaction of the upper 5 feet of loose fill may include a more costly deep foundation system which will be discussed in Section 6.4.

Grade Beams / Shallow Footings: Based on the findings from our subsurface evaluation and laboratory testing, we recommend an allowable bearing capacity of 2,500 pounds per square foot (psf) for soils below new footings at the CUP-3A and CUP-7 sites as long as the recommendations for subgrade preparation in Section 6.1 are incorporated into the design. This bearing capacity includes a factor of safety of at least three against bearing failure, and is applicable to newly constructed footings with widths of at least 18 inches and footing embedment of at least 24 inches below lowest adjacent grade.

A static modulus of subgrade reaction of 60 pounds per cubic inch (pci) may be used in order to develop soil springs below the foundation elements. For the lateral resistance of grade beams and footings, the geotechnical design parameters provided in the Lateral Earth Pressures section may be used.

As discussed in Section 2.4, differential dynamic settlement is relatively minor on the order of ¼ inch at the CUP-7 site during an earthquake event. The remaining CUP-

3A site is more susceptible to a differential dynamic settlement on the order of 1 inch during an earthquake event if the site is not mitigated. To reduce this to a minor amount on the order of ¼ inch, the site should be mitigated by overexcavating and recompacting the upper 5 feet of soil below grade to develop a mass of densified soil beneath the proposed building at CUP-3A. Long-term consolidation settlements are not likely due to the granular nature of much of the subsurface soils. Therefore, total dynamic settlements are approximately equivalent to the estimated dynamic settlements at the two project sites. After site mitigation via overexcavating and recompacting the upper 5 feet of soil at CUP-3A, the total dynamic settlement is expected to reduce from 4 inches to 1 inch, and the differential settlement from 1 inch to ¼ inch. Total settlements due to dead loads and normal duration live loads are expected to be less than ¼ inch, and are likely to occur during or immediately after construction.

***Mat Foundations*:** Effects from differential dynamic settlements at the two project sites may be limited by supporting the structures at these sites on structurally rigid mat foundations. A mat foundation is a large concrete slab, designed by a structural engineer for specific use, to interface one or more columns or pieces of equipment with the foundation soil. It may encompass the entire foundation footprint or only a portion. The mat contact stresses are generally lower than other shallow foundation types due to distribution of stress over a larger area and stress compensation from excavated soil. Thickness and reinforcement of the mat foundation should be in accordance with the recommendations of a structural engineer. The appropriate allowable contact pressure(s) beneath the mat foundations will vary with their size, shape, and other factors. Without mitigating the upper 5 feet at loose fill at CUP-3A, a mat foundation system may limit foundation differential settlements to less than 3/4 inch for dead and live loads and less than 1 inch for total loads including wind and seismic, as long as the contact pressure beneath the mats should not exceed the allowable bearing capacities as recommended above for grade beams / shallow foundations. Mat foundations are not anticipated at CUP-7. Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the mat. A design coefficient of subgrade reaction, $\mathbf{K_{v1}}$, of 260 kips per cubic foot (kcf) in compacted fill soils may be used for evaluating such deflections at the sites. This value is based on a square foot area and should be adjusted for the planned mat size. The coefficient of subgrade reaction, $\mathbf{K_B}$, for a mat of a specific dimension may be evaluated using the following equation:

$$\mathbf{K_B, = K_{v1}\ [(B+1)/2B]^2\ [(1+0.5(B/L)/1.5]}$$

where **B** is the width and **L** is the length of the foundation measured in feet.

Mat foundations bearing on fill may be designed using a coefficient of friction of 0.4 (total frictional resistance equals coefficient of friction times the dead load). The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed two-thirds of the total allowable resistance.

***Post-Tensioned Foundations*:** Effects from differential dynamic settlements at the two project sites may be limited through the application of post-tensioning in reinforcing, and hence, increasing the structural rigidity of grade beams / shallow footings. Thickness and reinforcement of a post-tensioned foundation should be in accordance with the recommendations of a structural engineer.

6.3 **Floor Slabs.** Slabs-on-grade should be supported on a 12-inch thick mat of compacted, engineered fill. Material for engineered fill and compaction requirements are presented in Sections 4.5 and 4.6. For moisture-sensitive flooring, floor slabs resting on soil should be underlain, at a minimum, by a capillary break system. We recommend 6 inches of clean coarse sand or pea gravel. When floor dampness is a concern, floor slabs should be underlain by a vapor barrier and capillary break system. We recommend a system consisting of a 10-mil polyethylene (or equivalent) membrane placed over 6 inches of clean coarse sand or pea gravel. The exposed subgrade should be moistened just prior to the placement of the capillary break system. A sand layer above the moisture barrier to aid in concrete curing should be evaluated by the structural engineer. The slab underlayment including the capillary break can be taken as part of the 12-inch thick pad of compacted, engineered fill described above. Flooring and waterproofing consultants should be consulted for additional slab waterproofing recommendations.

6.4 **Deep Foundations.** To mitigate significant dynamic settlement at the CUP-3A site, a deep foundation system that may include feasible alternatives such as, driven precast concrete piles (DPCP) and closed-end pipe piles, may be used to transfer building loads to a competent material of the Colma Formation for end bearing support at a depth of at least 40 feet. Should deep foundation be considered for design at the CUP-3A site, we would like to be given an opportunity in providing design consultation services/support to the structural engineer in providing geotechnical design parameters for evaluating the pile foundation system, as appropriate.

7.0 CORROSION

Schiff Associates performed corrosivity laboratory tests on two soil samples. Their laboratory results are included in **Appendix A – Supporting Geotechnical Data**. They performed the following tests:

- Resistivity (As-Received and Saturated)
- pH
- Electrical Conductivity
- Chemical Analyses of Cations (e.g. Calcium, Magnesium, Sodium)
- Chemical Analyses of Anions (e.g. Carbonate, Bicarbonate, Chloride, Sulfate)
- Chemical Analyses of Ammonium

- Chemical Analyses of Nitrate
- Chemical Analyses of Sulfide
- Oxidation-reduction (Redox) Potential

Electrical resistivities indicate soils are moderately corrosive to ferrous metals at the CUP-3A site and mildly corrosive at the CUP-7 site. The soil pH values indicate moderately alkaline soils at the CUP-3A site and slightly acidic soils at the CUP-7 site. Based on the pH values, the sites are classified as non-corrosive. The soluble salt contents of the samples are low indicating a low corrosion potential, and on-site near-surface soils present a negligible sulfate exposure to concrete structures. Based on the criteria in the Caltrans Corrosion Guidelines (Caltrans, 2003), the two project sites would not be classified as a corrosive site based on testing of near-surface soil samples.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Geotechnical Observation of Construction Activities. We should be retained during construction to provide site observation and consultation concerning the condition of the bottom of excavations pertaining to foundation construction and pipeline trench excavation. Foundation grades should be observed and, where necessary, tested under the direction of a qualified geotechnical engineer to verify compliance with final design recommendations. All site preparation work and excavations should also be observed to compare the generalized site conditions assumed in the final design report with those found on site at the time of construction.

8.2 Existing Underground Utilities. A number of underground water main pipelines pass beneath and in the vicinity of the proposed sites. Other existing subsurface lines include the SFPUC transmission lines, and sanitary and storm sewer lines. A PG&E gas transmission pipeline is located near the CUP-3A site. Some of these utilities were located and marked prior to our exploration to avoid damaging them during drilling.

The City may consider remarking these utilities prior to construction of the improvements so they remain visible during earthwork and construction of the improvements. Any excavations made adjacent to existing utilities should be backfilled with on-site or imported soil to at least 90 percent relative compaction (ASTM D 1557).

8.3 Surface Drainage. Proper surface drainage is essential for satisfactory site performance. Positive drainage should be provided and maintained to direct surface water away from building foundations and other site improvements. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet or greater away from the foundations, flatwork, and tops of slopes. Runoff should then be directed by the use of swales or pipes into a collective drainage system. Surface water should not be allowed to pond adjacent to footings. We further recommend that the proposed structure be equipped with appropriate roof gutters and downspouts. Downspouts should discharge to a system of closed pipes that transport the collected water to a suitable discharge facility. We recommend that drought tolerant vegetation be used for site landscaping. Irrigation should be kept at levels just sufficient to maintain plant vigor.

9.0 CLOSURE

The conclusions and recommendations presented herein are professional opinions based on geotechnical and geologic data and the project as described. A review by this office of any foundation, excavation, grading plans and specifications, or other work product that relies on the content of this report, together with the opportunity to make supplemental recommendations is considered an integral part of this study. Should unanticipated conditions come to light during project development or should the project change from that described, we should be given the opportunity to review our recommendations.

The findings and professional opinions presented in this report are presented within the limits prescribed by the client, in accordance with generally accepted professional engineering and geologic practices. There is no other warranty, either express or implied.

REGISTERED PROFESSIONAL ENGINEER
G. NEELAKANTAN
No.
Exp. 9/30/2013
GEOTECHNICAL
STATE OF CALIFORNIA

Submitted by:
GEOTECHNICAL CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
NICK S. NG
GE 2831
EXP 9/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

Reviewed by:
G. Neelakantan, P.E., G.E
Geotechnical Engineer, GE 2391

Nick S. Ng, P.E., G.E.
Geotechnical Engineer, GE 2831

REFERENCES

American Society of Civil Engineers (ASCE), 2005, Minimum Design Loads for Building and Other Structures (7-05).

Blake, T.F., 2000, "EQFAULT, A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration Using Three-Dimensional California Faults as Earthquake Sources."

Bonilla, M.G., 1998, Preliminary Geologic Map of the San Francisco South 7.5' Quadrangle and Part of the hunters Point 7.5' Quadrangle, San Francisco Bay Area, California, U.S. Geological Survey, Open-File Report 98-354.

Brabb, E.E., Graymer, R.W., and Jones, D.L., 1998, Geology of the onshore part of San Mateo County, California: A Digital Database, United States Geological Survey (USGS) Open-File Report (OFR) 98-137.

Bray, J.D. and Sancio, R.B., 2006, "Assessment of the Liquefaction Susceptibility of Fine-grained Soils," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 132, No. 9, pp. 1165 – 1177.

California Geological Survey (CGS), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0, Bryant, W. A. (compiler): http://www.consrv.ca.gov/CGS/information/publications/QuaternaryFaults_ver2.htm.

California Emergency Management Agency (CalEMA) – California Geological Survey (CGS) – University of Southern California (USC), 2009, Tsunami Inundation Map for Emergency Planning, San Francisco Quadrangle, Scale 1:36,000, June 15.

California Geological Survey (CGS) Department of Conservation, 2009, Tsunamis, Note 55.

California Department of Conservation, Division of Mines and Geology (CDMG), 1997, "Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California," March.

California Department of Transportation (Caltrans), 2003, Corrosion Guidelines, Version 1.0, Division of Engineering Services, Materials Engineering and Testing Services, September.

California Department of Transportation (Caltrans), 2008, Highway Design Manual, July 1.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, "The Revised 2002 California Probabilistic Seismic Hazard Maps," June.

Cetin, K.O. and Seed, R.B., 2000, Earthquake-Inducted Nonlinear Shear Mass Participation Factor (rd), Geotechnical Engineering Research Report No. UCB/GT-2000/08, Department of Civil Engineering, University of California at Berkeley, June.

Cetin, K.O. and Seed, R.B., 2004, Nonlinear Shear Mass Participation Factor (rd) for Cyclic Shear Stress Ratio Evaluation, Soil Dynamics and Earthquake Engineering, Volume No. 24, pp. 103 – 113.

Cetin, K.O., Seed, R.B., Der Kiureghian, A., Tokimatsu, K., Harder, L.F., Kayen, R.E., and Moss, R., 2004, Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 130, No. 12, pp. 1314 – 1340.

Cetin, K.O., Seed, R.B., Der Kiureghian, A., Tokimatsu, K., Harder, L.F., Kayen, R.E., and Moss, R., 2006, Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential – Discussion, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 132, No. 5, pp. 667 – 669.

Coduto, D.P., 1998, Geotechincal Engineering: Principles and Practice, Prentice Hall.

Earthquake Engineering Research Institute (EERI), 2008, "The Next Generation Attenuation Special Issue," Earthquake Spectra 24:1, February.

Federal Emergency Management Agency (FEMA), 2000, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings", FEMA 356, November.

GeoMotions, 2011, SHAKE2000, A Computer Program for 1D Analysis of Geotechnical Earthquake Engineering Problems, Version 8.5.0, April.

Frankel, A., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1996, National Seismic Hazard Maps: Documentation, June 1996, USGS Open File Report 96-532, July 19.

Geotechnical Consultants, Inc. (GTC), 2009a, Geotechnical Report, South Westside Groundwater Basin Conjunctive Use Project, San Mateo County, California, (study performed for CUP Well Locations CUP-10A, CUP-18, CUP-19, CUP-22A and CUP-41-4), Prepared for Kennedy/Jenks Consultants, April.

Geotechnical Consultants, Inc. (GTC), 2009b, Final Geotechnical Report, CUP Well Locations CUP-11A, CUP-23, CUP-36-1, CUP-44-1, and CUP-M-1, South Westside Basin Groundwater Storage and Recovery Project, San Mateo County, California, Prepared for Kennedy/Jenks Consultants, December.

International Code Council (ICC), 2009, 2009 International Building Code, February.

International Code Council (ICC), 2010, 2010 California Building Code, Title 24, Part 2, Volume 2, June.

Naval Facilities Engineering Command (NAVFAC), 1982a, "Soil Mechanics," Design Manual 7.1, May (Revalidated September, 1986).

Naval Facilities Engineering Command (NAVFAC), 1982b, "Foundations and Earth Structures," Design Manual 7.2, May (Revalidated September, 1986).

Petersen, M.D., Bryant, W.A., Cramer, C.H., Cao, T., Reichle, M., Frankel, A.D., Lienkaemper, J.J., McCrory, P.A., and Schwartz, D.P., 1996, Probabilistic Seismic Hazard Assessment for the State of California, CDMG Open-File Report 96-08.

Petersen, M.D., Frankel, A.D., Harmsen, S.C., Mueller, C.S., et al., 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps, United States Geological Survey (USGS) Open-File Report 2008-1128, 61p.

San Francisco Public Utilities Commission (SFPUC), 2008, 2007 Annual Groundwater Monitoring Report, Westside Basin, San Francisco and San Mateo Counties, California, April 25.

San Francisco Public Utilities Commission (SFPUC) – Engineering Management Bureau, 2009, "General Seismic Requirements for Design of New Facilities and Upgrade of Existing Facilities," Engineering Standard General Seismic Design Requirements, EMB Document WSIP/CSP-001-R2, Revision No. 2, October 6.

San Francisco Public Utilities Commission (SFPUC), 2011, Site and Floor Plans for Locations CUP-3A and CUP-7, 65% Progress Print Not For Construction, Regional Groundwater Storage and Recovery Project, Contract No. , Plan Nos. A1-0, A1-1, and A4-0, June.

Seed, H.B., Idriss, I.M., 1971, Simplified Procedure for Evaluating Soil Liquefaction during Earthquakes, Journal of Geotechnical Engineering, ASCE, Volume No. 97, No. 9, pp. 1249 – 1273.

Seed, H.B., Tokimatsu, K., Harder, L.F., and Chung, R.M., 1985, Influence of SPT Procedudres in Soil Liquefaction Resistance Evaluations, Journal of Geotechnical Engineering, ASCE, Volume No. 112, No. 12, pp. 1425 – 1445.

Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M., and Riemer, M.F., 2001, Recent Advances in Soil Liquefaction Engineering and Seismic Site Response Evaluation, Paper No. SPL-2, http://nisee.berkeley.edu.

Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, J.D., Kayen, R.E., and Faris, A., 2003, "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework," Earthquake Engineering Research Center, Report No. EERC 2003-06, College of Engineering, University of California, Berkeley.

Terzaghi, K., Peck, R.B., and Mesri, G., 1996, Soil Mechanics in Engineering Practice, Third Edition, Wiley-Interscience.

Tokimatsu, K. and Seed, H.B., 1987, Evaluation of Settlement in Sands due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Volume No. 113, No. 8, pp. 861 – 878.

United States Geological Survey (USGS), 1993, San Francisco South Quadrangle, California, 7.5-Minute Series (Topographic), Scale 1:24,000.

United States Army Corps of Engineers (USACE), 2003, Slope Stability, Engineering and Design, Engineering Manual EM 1110-2-1902, CECW-EW, October 31.

United States Geological Survey (USGS), 2008, 2008 National Seismic Hazard Mapping Program (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation Web Site, http://geohazards.usgs.gov/deaggint/2008/index.php.

United States Geological Survey (USGS), 2011, Seismic Hazard Curves, Response Parameters and Design Parameters: Earthquake Ground Motion Parameters, A Java-based Application, Version 5.1.0., http://earthquake.usgs.gov/research/hazmaps/design, February 10.

Witter, R.C., Knudsen, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., Randolph, C.E., Brooks, S.K. and Gans, K.D., 2006, "Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California," United States Geological Survey OFR 2006-1037, Scale 1:200,000. http://pubs.usgs.gov/of/2006/1037/.

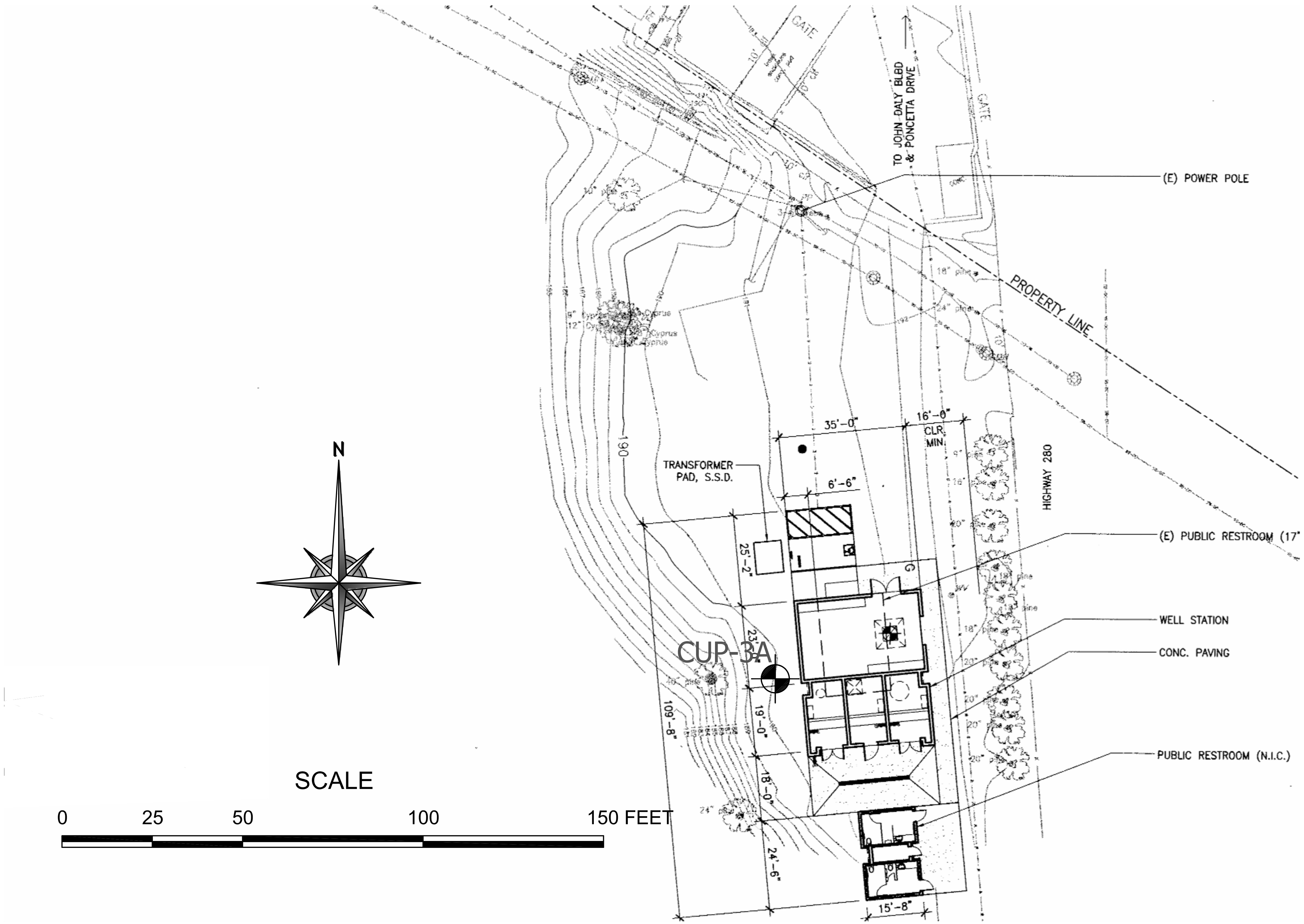
Working Group on California Earthquake Probabilities (WGCEP), 2003, "Earthquake Probabilities in the San Francisco Bay Region: 2002–2031," USGS Open-File Report 03-214. http://pubs.usgs.gov/of/2003/of03-214/.

Working Group on California Earthquake Probabilities (WGCEP), 2008, "The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF2)," USGS Open-File Report 2007-1437, http://pubs.usgs.gov/of/2007/1437/of2007-1437/.

Youd, T.L., et al., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 127, No. 10, pp. 817 – 833.

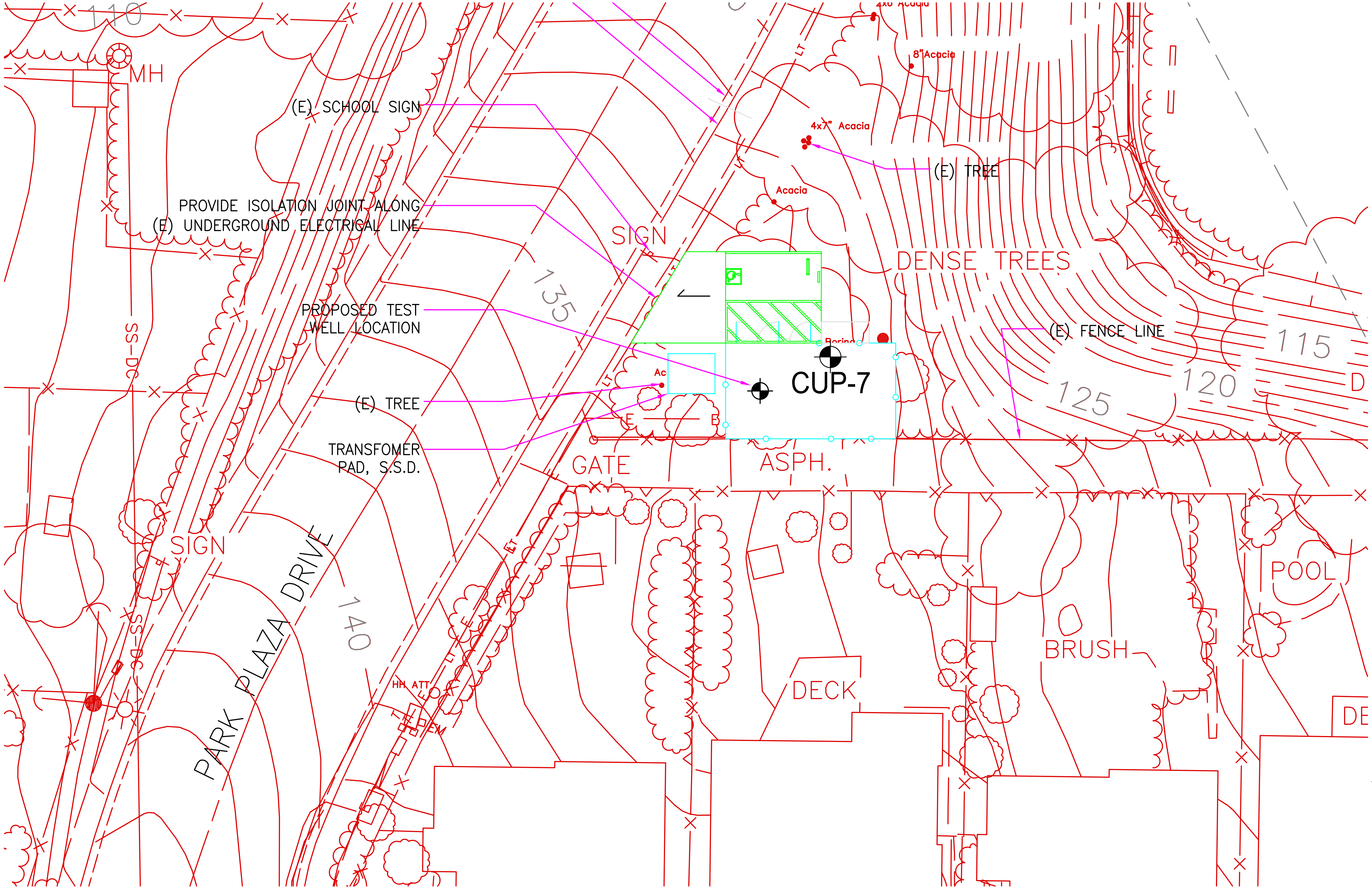
LEGEND

CUP-3A Geotechnical boring performed by Geotechnical Consultants, Inc. on August 8, 2011.



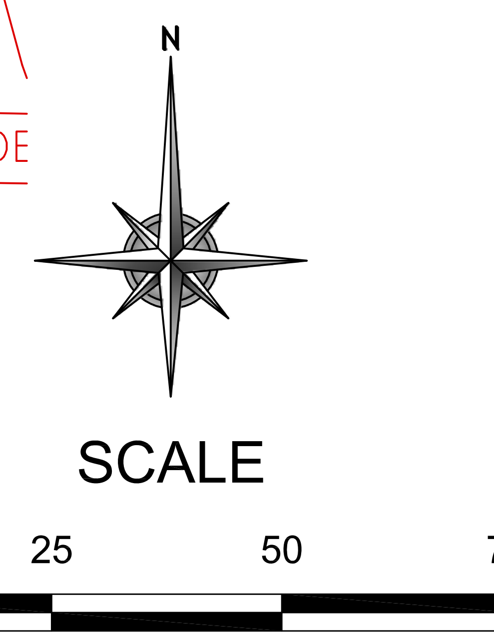
Source: San Francisco Public Utilities Commission, 2011, 65% Progress Print, Site Plan, CUP-3A, Lake Merced Golf Club, Daly City, Regional Groundwater Storage and Recovery Project, Contract No. WD-2600, Sheet No. A-1.0, June.

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, CA 94111	EXPLORATION LOCATION PLAN	PLATE 1
	REGIONAL GROUNDWATER STORAGE & RECOVERY PROJECT, CUP-3A SITE	
	JANUARY 2012	SF11014



LEGEND

CUP-7 Geotechnical boring performed by Geotechnical Consultants, Inc. on August 8, 2011.



Source: San Francisco Public Utilities Commission, 2011, 65% Progress Print, Site Plan, CUP-7, Park Plaza, Regional Groundwater Storage and Recovery Project, Contract No. WD-2600, Sheet No. A-4.0, June.

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, CA 94111	EXPLORATION LOCATION PLAN	PLATE 2
	REGIONAL GROUNDWATER STORAGE & RECOVERY PROJECT, CUP-7 SITE	
	JANUARY 2012	SF11014