PRELIMINARY GEOTECHNICAL INVESTIGATION
GEOTECHNICAL DUE DILIGENCE
BALBOA PARK BART UPPER YARD
SAN FRANCISCO, CALIFORNIA

Prepared for:
Ms. Karen Smith
Asian Neighborhood Design
1245 Howard Street
San Francisco, CA 94103
31 May 2013
13-050202

DC / RR JOINT VENTURE

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31 May 2013
Project No. 13-050202

Ms. Karen Smith
Asian Neighborhood Design
1245 Howard Street
San Francisco, CA 94103

Subject: Preliminary Geotechnical Investigation
Geotechnical Due Diligence
Balboa Park BART Upper Yard
San Francisco, California

Dear Ms. Smith:

This letter transmits our preliminary geotechnical investigation report for the Balboa Park BART Upper Yard site in San Francisco, California. The work described in this report was performed in accordance with our proposal dated 29 January 2013.

Our report contains preliminary recommendations including foundation design, retaining wall design, shoring, and site grading that should be reviewed in their entirety.

Our preliminary conclusions and recommendations are based on a limited subsurface exploration and laboratory testing program. Consequently, variations between expected and actual soil conditions may be found at localized areas during construction. Additional geotechnical studies should be performed to develop final geotechnical recommendations for the project.

We appreciate the opportunity to be involved with this project. If you have any questions, please call.

Sincerely yours,
DC/RG JV

Christian J. Divis
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PRELIMINARY GEOTECHNICAL INVESTIGATION
GEOTECHNICAL DUE DILIGENCE
BALBOA PARK BART STATION UPPER YARD
SAN FRANCISCO, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Divis Consulting/Rockridge Geotechnical Joint Venture (DC/RG JV), for the site located to the southwest of the intersection of Geneva Avenue and San Jose Avenue in San Francisco, California; as shown on the Site Location Map, Figure 1. We understand the purpose of this study is to provide geotechnical input for your due diligence activities and to address geotechnical issues regarding acquisition and development of the subject site for affordable housing. The subject site encompasses the entire Lot 039 of City of San Francisco Assessors Block 6973. The site is bound by Block 038 to the west, San Jose Avenue to the east and Geneva Avenue to the North. The lot is 30,750 square feet in area.

Schematic plans for the proposed affordable housing development (i.e. layout, number of stories, number of below-grade levels, and etc.) were not available when this study was performed.

The Balboa Park BART Station lies to the north of the site. The BART tunnel lies to the west of the site. Based on documents provided by BART, it appears that the area west and south of the site may have been excavated during construction for the BART station and tunnel. An easement map provided by BART is presented in Appendix D. The approximate location of the BART tunnel is shown on the Site Plan, Figure 2. As-built drawings for the Balboa Park Bart Station indicate the bottom of the BART tunnel is on the order of 30 feet below existing ground surface (bgs). A confidentiality agreement was required to obtain the as-built drawings; consequently, we have not included them in this report.

2.0 SCOPE OF WORK

Our preliminary geotechnical investigation was performed in general accordance with our proposal dated 29 January 2013. Our scope of work included performing two Cone Penetration Tests (CPTs) and performing laboratory tests on selected near surface soil samples. Based on the results of our field investigation, laboratory testing and engineering analysis, we developed preliminary geotechnical conclusions and recommendations regarding the following:

- most appropriate foundation type(s) for the proposed structures
- estimates of total and differential settlement of new foundations under design loads
- subgrade preparation for slab-on-grade floors and exterior flatwork
- flexible (asphalt concrete) and rigid (Portland-cement concrete) pavement design
- permeable pavers
- retaining wall design
- site grading and excavation, including criteria for fill quality and compaction
- utility trenching and backfill
- temporary slopes and shoring
- soil corrosivity testing results
- expansion potential of the near surface soil
- settlement of compressible soil, as appropriate
- geologic hazards and related ground deformation, including seismic hazards
- construction considerations.

3.0 PRELIMINARY FIELD INVESTIGATION

We explored subsurface conditions at the site by advancing two CPTs (CPT-1 and CPT-2) at the approximate locations shown on Figure 2. The CPTs were performed on 29 March 2013, by Gregg Drilling. CPT-1 and CPT-2 were advanced to practical refusal in cemented sands at depths of approximately 20 and 27 feet bgs, respectively.

Prior to performing the CPTs, the upper five feet were hand augered to check for utilities. The soil encountered in the upper five feet was logged by our field engineer and collected for laboratory testing. The CPT was performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone tip measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges within the cone continuously measure soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, are recorded by a computer. Accumulated data are processed to provide engineering information such as the types and approximate strength characteristics of the soil encountered.

The CPT logs, showing tip resistance and friction ratio by depth, as well as interpreted SPT N-values, soil shear strength parameters, and soil classifications, are presented in Appendix A on Figures A-1 and A-2. The soil was classified in accordance with the classification system presented on Figure A-3.

4.0 LABORATORY TESTING

Near surface samples were collected from the upper five feet at each CPT location. The soil samples were tested to evaluate soil corrosivity and resistance value (R-value) for pavement design. The results of the laboratory testing are presented in Appendix B.

5.0 SUBSURFACE CONDITIONS

The results of our preliminary subsurface investigation indicate the site is underlain by fill and sand with varying amounts of clay and silt. A description of each stratum is summarized below.

Fill – Where explored, the site is overlain by asphalt concrete (AC) of varying thickness. The AC is underlain by about five feet of fill. Fill thickness may be greater in areas with existing utility trenches. The fill generally consists of sand and gravel with varying amounts of debris (wood fragments).

Colma Formation – The fill is underlain by sands with varying amounts of silt and clay to the maximum depths explored. Geologic maps of the area indicate this sand is the Colma formation (Figure 3). The Colma formation generally consists of dense to very dense sand with interbedded stiff clay lenses. It is relatively incompressible and strong.

Groundwater was encountered at a depth of about 10 feet bgs in the CPTs. Seasonal groundwater elevations generally fluctuate on the order of several feet depending on rainfall and infiltration.

6.0 REGIONAL SEISMICITY

The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras faults. For active faults within 50 kilometers (km) of the site, the distance from the site and estimated maximum Moment magnitude[1] [Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

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1 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1
REGIONAL FAULTS AND SEISMICITY

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Maximum Magnitude
N. San Andreas - Peninsula	5	West	7.2
N. San Andreas (1906 event)	5	West	8.1
San Gregorio Connected	12	West	7.5
N. San Andreas - North Coast	13	West	7.5
Total Hayward	24	Northeast	7.0
Total Hayward-Rodgers Creek	24	Northeast	7.3
Monte Vista-Shannon	36	Southeast	6.5
Mount Diablo Thrust	40	East	6.7
Total Calaveras	40	East	7.0
Rodgers Creek	41	North	7.07
Point Reyes	42	Northwest	6.9
Green Valley Connected	45	East	6.8

Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (see Figure 4 for explanation of MM scale) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25.

In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989, in the Santa Cruz Mountains with an M_w of 6.9, approximately 91 km from the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably an M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably an M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

In 2006, the Working Group on California Earthquake Probabilities (WGCEP 2008) at the U.S. Geologic Survey (USGS) predicted a 62 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by the year 2031.

The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2008) has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring during the period 2007 to 2037 is 63 percent. The highest probabilities are assigned to the Northern segment of the San Andreas Fault and the northern Hayward/Rodgers Creek Fault. These probabilities are 21 and 31 percent, respectively.

7.0 GEOLOGIC HAZARDS

Because the project site is in a seismically active region, we preliminarily evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[2] lateral spreading,[3] and cyclic densification[4]. The results of our preliminary evaluation are presented in this section.

7.1 Ground Shaking

The seismicity of the site is governed by the activity of the San Andreas Fault, although ground shaking from future earthquakes on other faults, including the Calaveras and Hayward faults, would also be felt at the site. The intensity of earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake epicenter, and magnitude and duration of the earthquake. We judge that strong to violent ground shaking could occur at the site during a large earthquake on one of the nearby faults.

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[2] Liquefaction is a phenomenon where loose, saturated, cohesionless soil experiences temporary reduction in strength during cyclic loading such as that produced by earthquakes.

[3] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[4] Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

7.2 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we preliminarily conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

7.3 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

We evaluated the liquefaction potential of soil encountered below groundwater at the site using data collected in our CPTs. The results of our preliminary evaluation indicate the sands encountered below the groundwater table at the site were sufficiently dense and/or contained sufficient fines that we preliminarily conclude the potential for liquefaction and associated hazards are low.

7.4 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. Loose to medium dense sand and gravel was encountered within the upper 5 feet at the site. Where this sand does not contain sufficient fines, it may be susceptible to cyclic densification. We preliminarily concluded settlements associated with cyclic densification would be on the order of 1/2 inch after a major seismic event on a nearby fault.

8.0 PRELIMINARY DISCUSSIONS AND CONCLUSIONS

Our preliminary conclusions and recommendations are presented in the remainder of this report. The primary geotechnical issues to be addressed for proposed development at the site are:

- A portion of the site is located within or adjacent to the BART Zone of Influence; design and construction of the proposed project within the BART Zone of Influence has to follow BART guidelines.
- The presence of about five feet of undocumented fill underlying the site that is susceptible to cyclic densification and has low strength and high compressibility.
- Providing adequate vertical and lateral support of proposed improvements.

8.1 BART Considerations

Tunnels and below grade station improvements that contain rail lines for the BART subway underlie the area to the west of the site (between the site and Highway 280) and north of the site. BART has published general guidelines for construction near its subway structures. General guidelines for design and construction over or adjacent to BART subway structures are attached as Appendix C and a copy of documents relating to the BART easement are attached as Appendix D. On the basis of the available subsurface and proposed project information, we anticipate the following BART guidelines will impact design and construction at the site:

- The Zone of Influence is defined by BART as the area above a Line of Influence which is a line from the critical point of substructure at a slope of 1-1/2 horizontal to 1 vertical (line sloping toward ground level).
- A minimum clearance between the proposed structure and BART of 7.5 feet must be maintained. Temporary shoring elements may exceed this clearance and may require BART approval.
- Soil redistribution caused by temporary shoring or permanent foundation systems shall be analyzed.
- Shoring shall be required to maintain an at-rest soil condition; shoring structure shall be monitored for movement.
- Minimum pre-drilled depth for piles shall be approximately 10 feet below the Line of Influence.
- Tunnels, where affected, shall be monitored for movement and deformation caused by adjacent construction activities to ensure structural and operational safety.
- Dewatering shall be monitored for changes in groundwater level; a recharge program will be required if existing groundwater level is expected to drop more than two feet.
- Where basements are excavated, the amount of loading (on subway) can be increased to the extent it is balanced by the weight of the removed material (120 pounds per cubic foot [pcf] for dry soil, and 70 pcf for submerged soil; however, the effect of soil rebound in such cases shall be fully analyzed.
- All structures shall be designed as not to impose any temporary or permanent adverse effects including unbalanced loading and seismic loading, on the adjacent BART subways.

The guidelines and discussions within this report regarding BART should <u>not</u> be relied upon as complete. The project architect should obtain further details regarding design and construction requirements directly from BART. As-built information of the adjacent BART subway structure should be made available to the Geotechnical Engineer for developing final geotechnical recommendations.

8.2 Foundation Support

8.2.1 Foundations beyond BART Zone of Influence

Low to mid-rise buildings with moderate loads may be supported on shallow foundations such as spread footings or mats bearing on medium dense to very dense native sand, or on engineered fill. As previously discussed, native medium dense to very dense sand of the Colma formation was encountered about 5 feet bgs at the CPT locations. For preliminary design, we recommend the shallow foundations be designed for allowable bearing capacities of 3,000 pounds per square foot (psf) for dead plus live loads and 4,000 psf for total loads (including wind and seismic loads) if bearing on engineered fill or medium dense sand within the upper 10 feet bgs; and 5,000 psf for dead plus live loads and 6,500 psf for total loads if bearing on dense to very dense sand below a depth of 10 feet bgs.

We estimate total settlement of shallow foundations designed using the allowable bearing pressures presented this report will be on the order of one inch and differential settlement will be on the order of 1/2 inch over a 30-foot horizontal distance.

To evaluate the pressure distribution beneath the shallow foundations, we recommend a modulus of vertical subgrade reaction (k_{v1}) of 100 and 150 pounds per cubic inch (pci) be used above and below a depth of 10 feet bgs, respectively. This modulus value (k_s) should be scaled to account for foundation width (B) using the following equation:

$$k_s=[k_{v1}][(B+1)/(2*B)]^2$$

Where: B = width of loaded area
k_{v1} = modulus of vertical subgrade reaction for one-foot-square plate

Lateral loads can be resisted by a combination of passive resistance acting against the vertical face of the shallow foundations and friction along the bases of the foundations. Passive resistance may be calculated using preliminary lateral pressures corresponding to an equivalent fluid weight of 250 and 150 pcf above and below ground water table, respectively. The upper foot of soil should be ignored for passive resistance unless confined by a concrete slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.35. The passive resistance and base friction values include a factor of safety of about 1.5 and may be used in combination without reduction.

8.2.2 Foundations within BART Zone of Influence

According to BART guidelines, all structures located within the BART Zone or Influence should be designed as not to impose any temporary or permanent adverse effects, including unbalanced loading and seismic loading, on the adjacent BART subways. Where basements are excavated, the amount of loading (on subway) can be increased to the extent it is balanced by the weight of the removed material.

If proposed structures within the BART Zone of Influence are designed such that there are no net loading on the subway (i.e. the load of new structures is balanced by the weight of excavated soil for partial or full basements); then the proposed structures may be supported on shallow foundations as presented in Section 8.2.1.

If new loads will be greater than the weight of excavated soil; then the structures should be supported on deep foundations that derive load bearing capacity in the soil below the Line of Influence. Appropriate deep foundation systems include drilled piers and proprietary deep foundation systems such as torqued-down piles, auger cast piles, and rammed aggregate piers. Special low frictional resistance coating or casing will need to be applied to the upper portion of the deep foundations (portion above the Line of Influence) to minimize load transfer between the pier/pile and the soil above the Line of Influence.

8.3 Site Preparation and Grading

Grading operations should commence after demolition and removal of the existing pavements, foundations, slabs, and underground utilities within the development area. Following demolition, all areas to receive improvements should be stripped of vegetation and organic topsoil. The pavement material, including asphalt, may be segregated from organic topsoil and used as engineered fill, provided it meets the fill requirements presented in a subsequent paragraph of this section and is acceptable from an environmental standpoint. The stripped organic soil can be stockpiled for later use in landscaped areas, if approved by the architect; organic topsoil should not be used as engineered fill.

Approximately five feet of undocumented will consisting of sand and gravel was encountered while hand auguring at CPT-1 and CPT-2; consequently overexcavation and recompaction of up to five feet of fill may be required where encountered below proposed improvements.

In areas to receive improvements or fill, the soil subgrade exposed by stripping or overexcavation, if any, should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least percent 90 relative compaction[5]. The soil subgrade should be kept moist until it is covered by concrete, capillary break material, or aggregate base. The exposed subgrade should be checked by the geotechnical engineer prior to placing fill or improvements.

Fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. Fill placed below foundations and any material consisting of clean sand or gravel

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[5] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-09 laboratory compaction procedure.

(defined as soil with less than 10 percent fines by weight) should be compacted to at least 95 percent relative compaction. Fill greater than five feet in thickness or fill placed within the upper foot of pavement soil subgrade should also be compacted to at least 95 percent relative compaction, and be non-yielding.

Material excavated at the site will primarily consist of sand with varying amounts of fines. If on-site material is to be used as fill, it should contain no rocks larger than four inches in greatest dimension, with no more than 20 percent of the rock fragments larger than two inches in greatest dimension.

Imported fill should be free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the Geotechnical Engineer. Samples of proposed select fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed imported material.

8.3.1 Exterior Flatwork and Paver Subgrade Preparation

The performance of exterior flatwork and pavers will depend on the care taken to prepare the subgrade below them. We recommend four inches of Class 2 aggregate base (AB) be placed beneath exterior concrete flatwork and permeable pavers on the site. The AB should be moisture-conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction beneath flatwork that will receive only pedestrian traffic. Permeable pavers may be underlain by a drainage layer as recommended by the manufacturer.

8.3.2 Pavement Subgrade Preparation

In vehicular pavement areas where native or imported soil is exposed at subgrade level, the upper eight inches of subgrade should be moisture-conditioned to above optimum moisture content and compacted to at least 95 percent relative compaction to achieve a firm, non-yielding subgrade. The soil subgrade should be kept moist until it is covered by AB. The AB placed beneath pavements receiving vehicular traffic should be compacted to at least 95 percent relative compaction.

8.3.3 Utility Trench Backfill

All trenches should conform to the current CAL-OSHA requirements. Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted as according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

8.3.4 Retaining/Basement Wall Backfill

Wall backfill should consist of on-site soil or select fill and should be placed using light (hand-operated) compaction equipment per the requirements presented in Section 8.3. If heavy equipment is used within five feet of the wall, the wall may require design for the additional surcharge pressure exerted by the equipment.

8.3.5 Surface Drainage

Positive surface drainage should be provided around the residence to direct surface water away from the foundations. To reduce the potential for water ponding adjacent to the building, we recommend the ground surface within a horizontal distance of five feet from the building slope down away from the building with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations.

8.4 Temporary and Permanent Slopes

Excavations that will be deeper than five feet and will have to be entered by workers should be shored or sloped in accordance with the latest version of the Occupational Safety and Health Administration (OSHA) standards. The contractor should be responsible for all temporary slopes and shoring systems used at the site, and should designate one of their on-site employees as a “competent person” who is responsible for trench and excavation safety. The competent person should be responsible for determination of the actual OSHA soil type and should direct the excavation crews to adjust slopes inclinations if appropriate. Based on the results of our preliminary subsurface investigation, we judge that temporary cuts in native soil less than 10 feet in height and above the groundwater inclined no steeper than 1.5:1 (horizontal to vertical) will be stable provided that they are not surcharged by equipment, building material or adjacent foundations. Permanent slopes should not be steeper than 2:1 (horizontal to vertical) and be located above groundwater table.

8.5 Temporary Shoring

The depth and extent of excavations for the proposed development are unknown at this time. Where space permits, temporary cuts may be excavated as discussed in Section 8.4. Where space does not allow for temporary slopes, adjacent improvements may be retained by temporary shoring.

We judge a cantilevered soldier-pile-and-lagging system would be the most suitable and economical shoring system for excavations of less than 10 feet. If the excavation extends below 10 feet, dewatering will likely be required. Due to the BART dewatering restrictions, as discussed in Section 8.1, a temporary shoring system which minimizes the flow of groundwater into the excavation may be required. Temporary shoring systems that minimize the flow of groundwater into the excavation include mixed-in-place secant soil-cement column walls and interlocking sheet pile walls. Where the depth of excavation exceeds about 10 to 12 feet, a cantilevered temporary shoring system may become uneconomical and tiebacks or internal bracing may be required. We can provide recommendations for alternative shoring systems once the extent of the excavations, if any, is known. The shoring should be designed to limit ground deformations to less than an inch. Cantilevered temporary shoring systems should be designed for active-rest lateral earth pressures, except where the shoring is located within BART Zone of Influence. Temporary shoring located within the BART Zone of Influence should be designed for at-rest lateral earth pressures. For preliminary design, we recommend equivalent fluid weights of 35 and 55 pcf be used to evaluate active and at-rest lateral earth pressures, respectively. The temporary shoring should also be designed for surcharges from traffic, construction equipment and stockpiles, groundwater pressure, and adjacent structures, as applicable.

For tieback or internally-braced temporary shoring systems, the design lateral earth pressures will be dependent on the type of shoring system selected and should be addressed during final geotechnical investigation for the project. Passive resistance for the temporary shoring systems will depend on the type of shoring and the depth of excavation and should be addressed during the final geotechnical investigation.

During excavation, the shoring system may deform laterally, which could cause the ground surface adjacent to the shoring walls to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. We believe that the movements of a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch. A monitoring program should be established to evaluate the effects of the construction on the adjacent buildings and surrounding ground and possibly the adjacent BART structures.

8.6 Basement/Retaining Walls

Retaining walls that are free to rotate at the top may be designed using an active earth pressure. For these walls, we recommend using a preliminary design equivalent fluid weight of 35 pcf for level backfill. For restrained walls (no movement at the top of the wall), we recommend designing for at-rest earth pressures using a preliminary design equivalent fluid weight of 55 pcf for level backfill.

Because the site is in a seismically active area, retaining walls should be designed to resist pressures associated with earthquake forces. We recommend retaining walls be designed to resist the greater of the earth pressures given in the preceding paragraph, or the unrestrained pressure (active earth pressure) plus a seismic increment. For level backfill behind the wall, the seismic increment should be taken as equivalent fluid weight of 12 pcf.

The walls should be designed for traffic surcharge if located within 10 feet from streets; and for hydrostatic pressures if extending below the groundwater table. Where new or existing foundations are located behind retaining walls and an imaginary plane taken from the bottom of the footing projected at 1.5:1 (horizontal to vertical) downward intersects the retaining wall, additional surcharge pressures should be included to account for vertical and lateral foundation loading on the retaining wall. If present, we should evaluate these pressures on a case by case basis. The foundations of the retaining walls should be designed using the same design values as presented in Section 8.2.

The design pressures above are based on fully drained walls. Water can accumulate behind the walls from perched groundwater and other sources, such as rainfall, irrigation, and broken water lines. One acceptable method for backdraining the wall is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the backside of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the wall. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140NC or equivalent). The perforated collector pipe should be sloped at an inclination of at least one percent to the discharge location.

Wall backfill material and compaction should conform to the recommendations presented previously in Section 8.3. The upper foot of backfill should consist of low-permeability backfill material to limit surface water infiltration into the backfill. Lightweight compaction equipment should be used to reduce stresses induced on the retaining walls during fill placement unless the walls are appropriately braced. Retaining walls should be backfilled before framing or subsequent construction to minimize effects of initial wall deflections from backfill placement.

If moisture migration through the basement walls is undesirable, we recommend waterproofing be installed and water stops be placed at all construction joints. The waterproofing system should be designed by others.

8.7 Concrete Slab-on-Grade Floors

If it is desirable to reduce water vapor transmission through floor slabs, we recommend installing a capillary moisture break and a water vapor retarder beneath the floor. In general, water vapor transmission through the floor slab should be reduced where there is potential for finished floor coverings to be adversely affected by moisture. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 2.

TABLE 2
GRADATION REQUIREMENTS FOR CAPILLARY MOISTURE BREAK

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be moist, but not saturated, at the time concrete is placed. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured.

Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

8.8 Asphalt Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We expect the final soil subgrade in asphalt-paved areas will generally consist of sand with varying amounts of fines. On the basis of our laboratory test results on this soil, we selected an R-value of 50 for design. If the existing subgrade will be raised beneath the paved areas, the fill material should have the same or higher R-value than the native soil. Therefore, additional tests should be performed on the proposed fill to measure its R-value. Depending on the results of the tests, the pavement design may need to be revised.

For our calculations, we assumed a Traffic Index (TI) of 4.5 for automobile parking areas with occasional trucks, and 6.0 for driveways and truck-use areas; these TIs should be confirmed by the project civil engineer. Table 3 presents our recommendations for asphalt pavement sections.

TABLE 3
PAVEMENT SECTION DESIGN

TI	Asphalt Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4.5	2.5	8
6.0	3.5	8

Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base should be compacted to at least 95 percent relative compaction.

8.9 Soil Corrosivity

On the basis of the resistivity measurements provided by Sunland Analytical, the soil sample tested is classified as non-corrosive to buried steel and concrete. However, the chemical test data presented in Appendix B should be evaluated by a corrosion consultant prior to final design.

8.10 Seismic Design

For preliminary seismic design in accordance with the provisions of 2010 San Francisco Building Code (SFBC) we recommend the following:

- Maximum Considered Earthquake (MCE) S_s and S_1 of 1.789g and 0.918g, respectively.
- Site Class D
- Site Coefficients F_A and F_V of 1.000 and 1.500
- MCE spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.789g and 1.377g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.192g and 0.918g, respectively.

9.0 LIMITATIONS

This preliminary geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The preliminary conclusions and recommendations made in this report are based on the assumption that the subsurface soil, rock, and groundwater conditions do not deviate appreciably from those disclosed in the CPTs. A supplemental investigation and final geotechnical report should be prepared prior to finalizing any design. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

REFERENCES

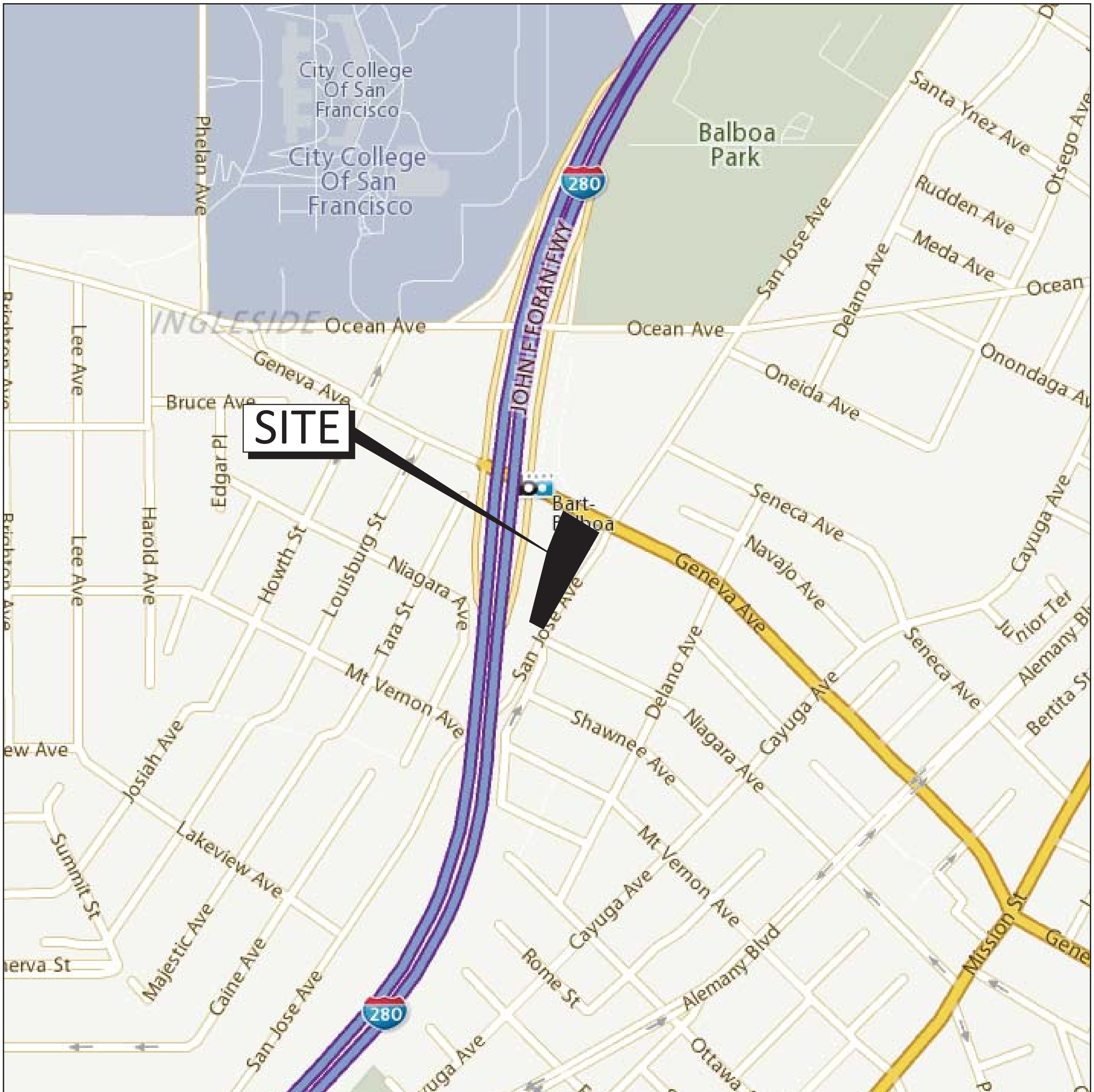
California Division of Mines and Geology, (1996), *Probabilistic Seismic Hazard Assessment for the State of California*, DMG Open-File Report 96-08.

California Division of Mines and Geology, (1997), *Fault Rupture Hazard Zones in California*, Special Publication 42.

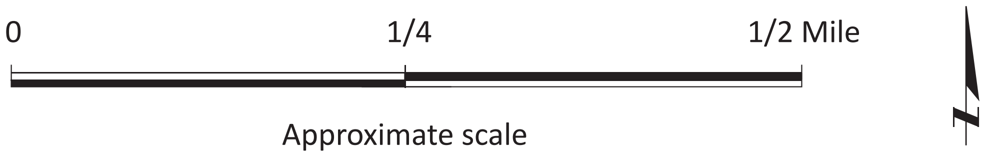
California Geological Survey, (2008), *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117.

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

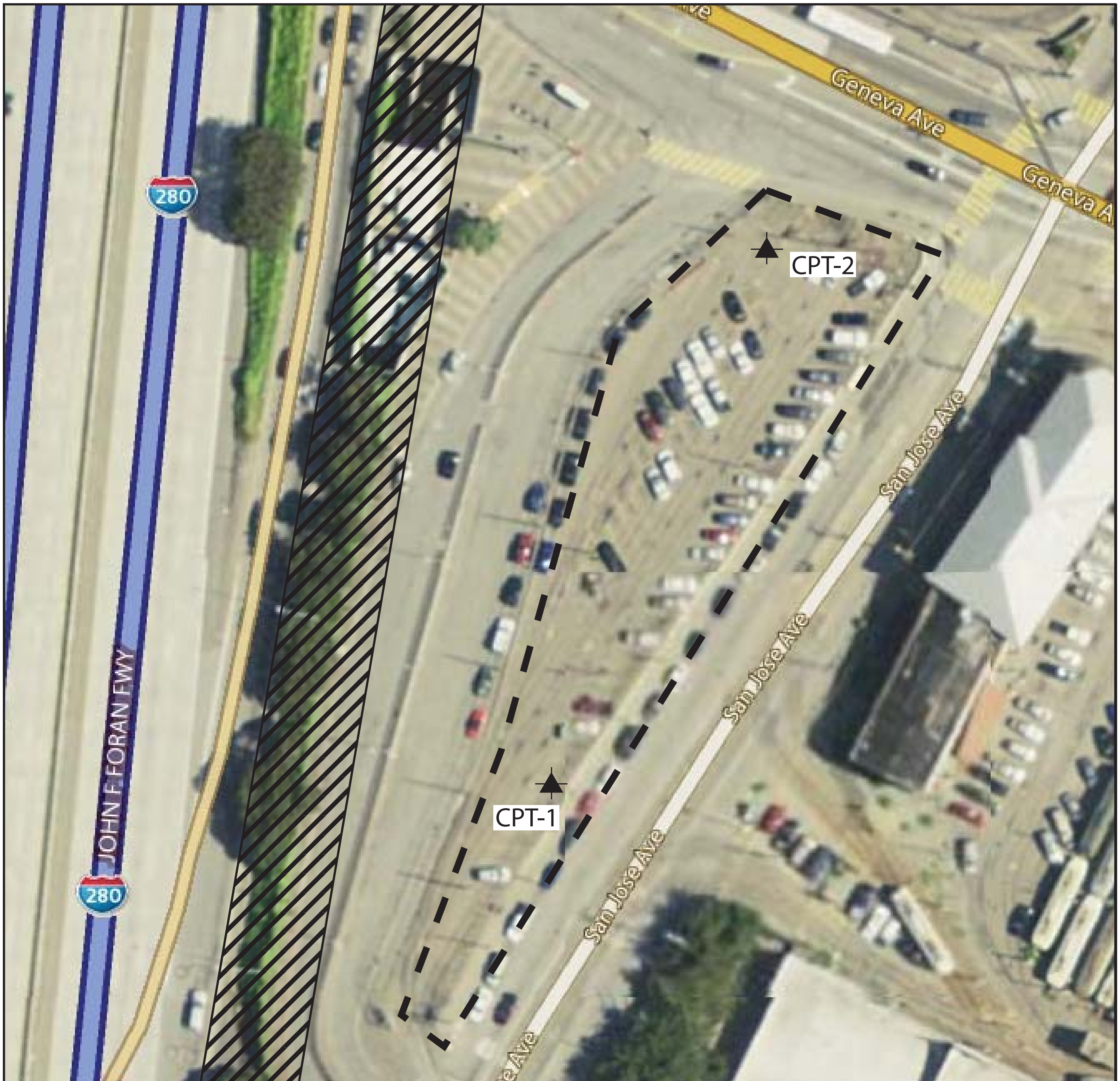
FIGURES



Base map: (c) 2013 Rand McNally



BALBOA PARK BART UPPER YARD San Francisco, California	SITE LOCATION MAP		
DC / RR JOINT VENTURE	Date 5/31/13	Project No. 13-050202	Figure 1



CPT-1 — Approximate location of cone penetration test performed by DC/RG JV, 29 March 2013

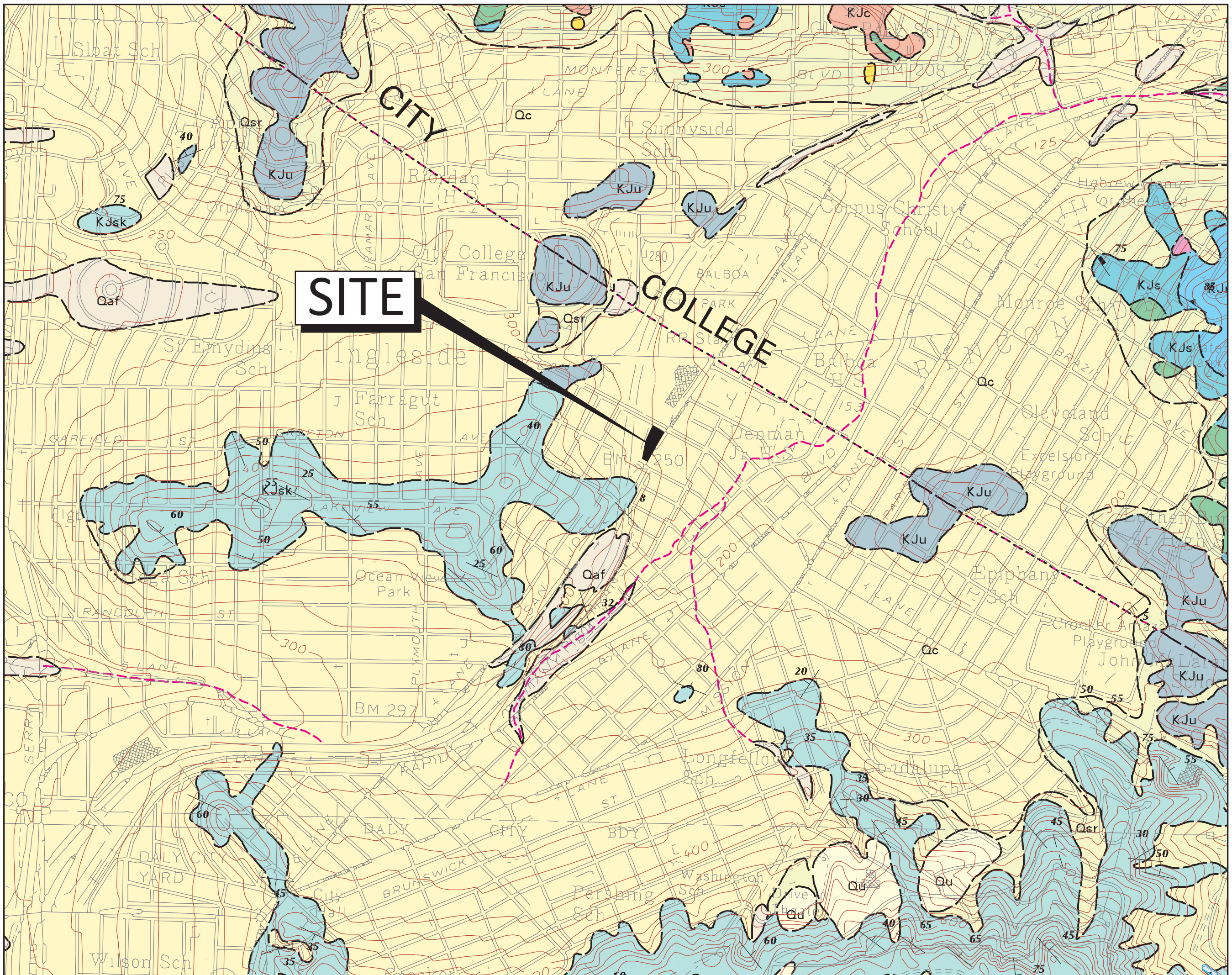
Approximate site boundry

Approximate location of BART tunnel

0 — 70 Feet
Approximate scale

Base map: (c) 2013 Rand McNally

BALBOA PARK BART UPPER YARD San Francisco, California	**SITE PLAN**
DC/RR JOINT VENTURE	Date 5/31/13 | Project No. 13-050202 | Figure 2



Qaf	Artificial fill
Qc	Colma formation
Qsr	Slope debris and ravine fill
Qu	Sedimentary deposits, undifferentiated
KJs	Sandstone and shale
KJc	Chert
KJg	Greenstone
KJsk	Sandstone and shale
KJu	Sheared rocks
— — —	Contact, approximately located
- - - - -	1800s shoreline and stream channels

0 2000 4000 feet

Approximate scale

Reference: 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, by M. G. Bonilla, 1998.

BALBOA PARK BART UPPER YARD
San Francisco, California

DC/R JOINT VENTURE

REGIONAL GEOLOGIC MAP

Date 05/31/13	Project No. 13-020202	Figure 3

I Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII Frightens everyone. General alarm, and everyone runs outdoors.
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII General fright, and alarm approaches panic.
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX Panic is general.
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X Panic is general.
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI Panic is general.
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII Panic is general.
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

BALBOA PARK BART UPPER YARD
San Francisco, California

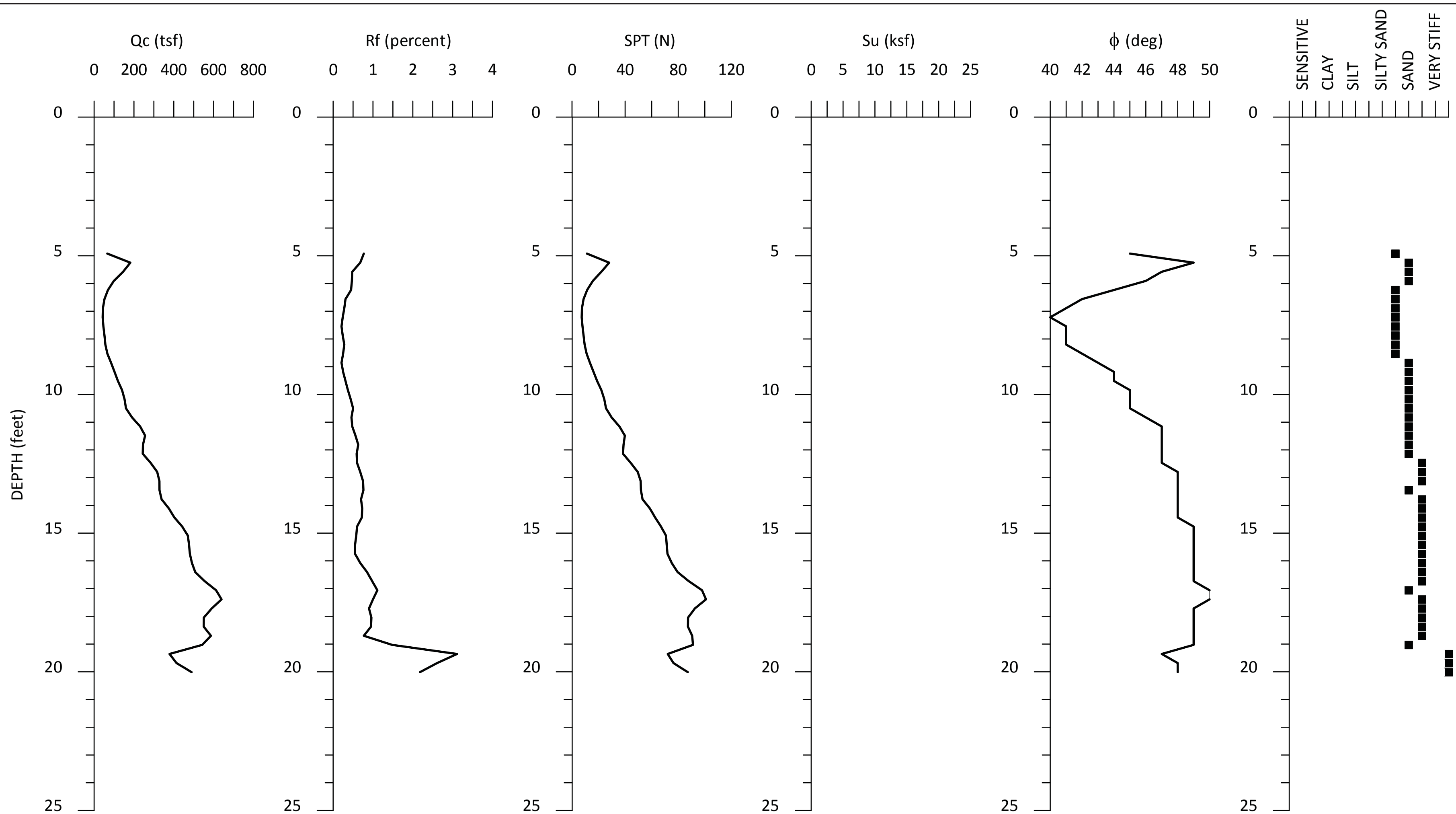
DC / ЯR JOINT VENTURE

MODIFIED MERCALLI INTENSITY SCALE

Date 5/31/13	Project No.12-050202	Figure 4

APPENDIX A

Cone Penetration Test Results



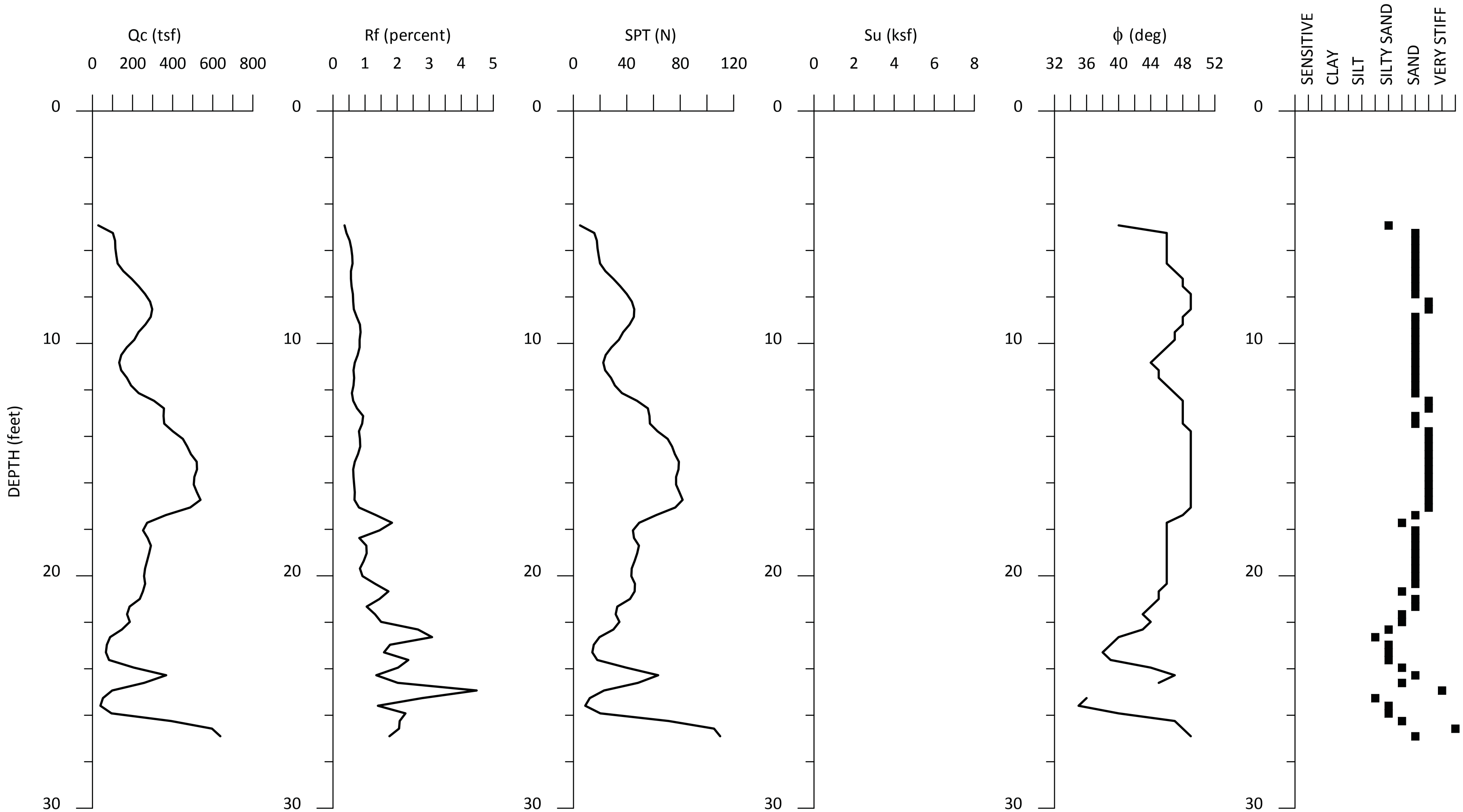
Terminated at 20 feet
Groundwater measured at 10 feet.
Date performed: 3/29/13

BALBOA PARK BART UPPER YARD
San Francisco, California

CPT-1

Date 5/31/13	Project 13-050202	Figure A-1

DC/R JOINT VENTURE



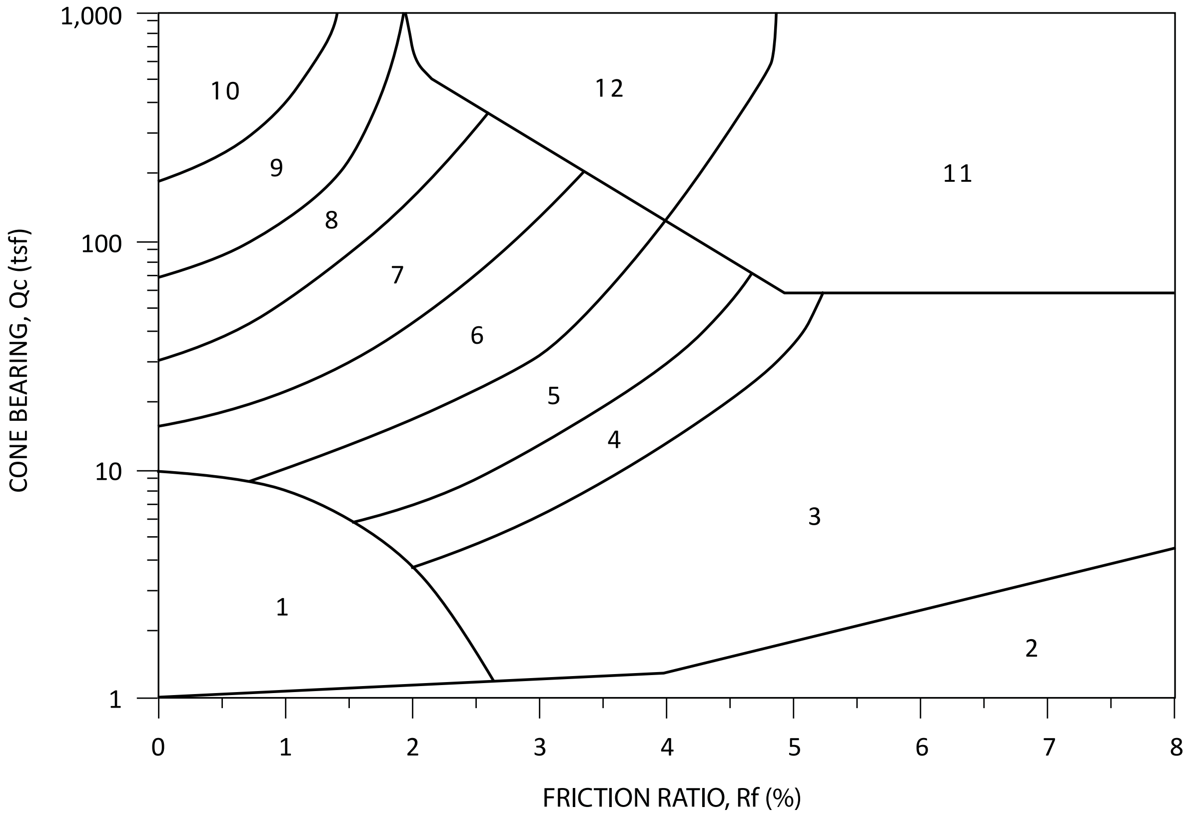
Terminated at 20 feet
Groundwater measured at 10 feet.
Date performed: 3/29/13

BALBOA PARK BART UPPER YARD
San Francisco, California

CPT-2

Date 5/31/13	Project 13-050202	Figure A-2

DC/SR JOINT VENTURE



ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for Qc 9 tsf)	Sensitive Fine-Grained
2	1	15 (10 for Qc 9 tsf)	Organic Material
3	1	15 (10 for Qc 9 tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
8	4	---	SAND to SILTY SAND
9	5	---	SAND
10	6	---	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	---	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc x 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

References:
1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud Qc £9). Estimated from local experience (ne-grained soils Qc > 9).

BALBOA PARK BART UPPER YARD
San Francisco, California

JOINT VENTURE

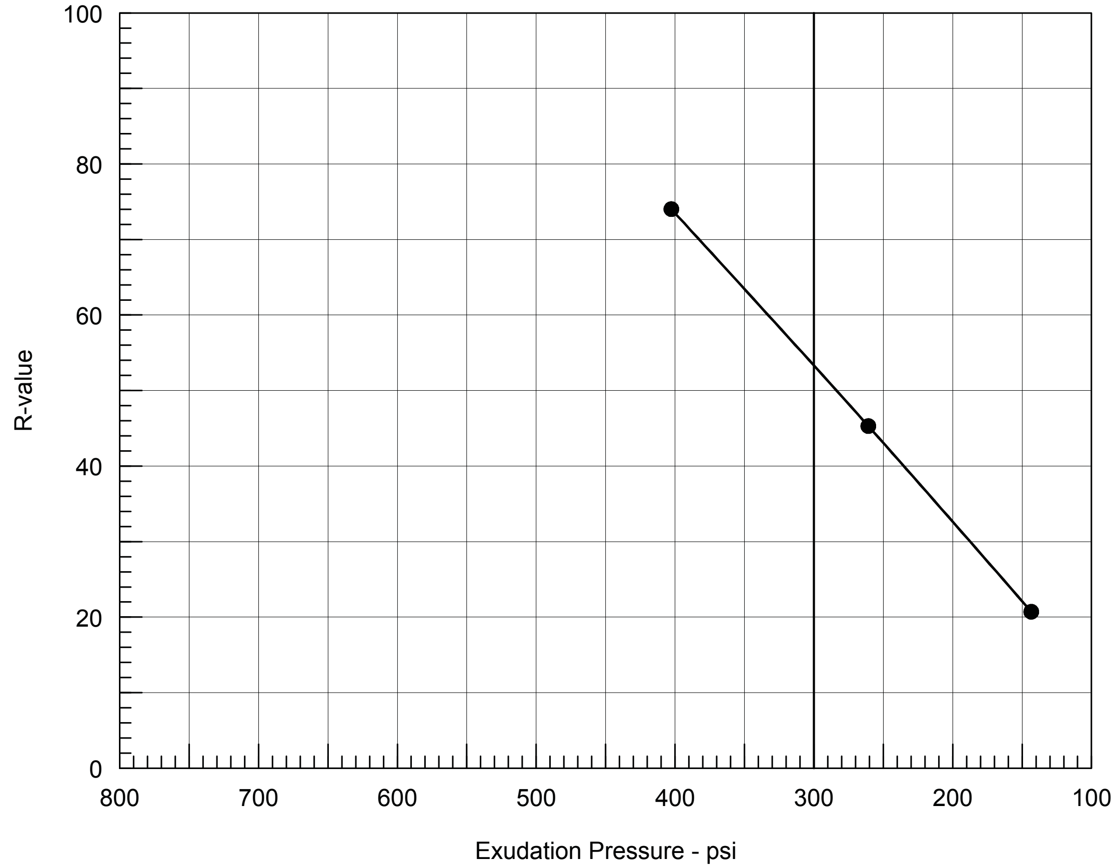
CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 5/31/13 | Project No.13-050202 | Figure A-3

APPENDIX B

Laboratory Test Results

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	200	135.6	7.7	0.00	116	2.49	143	21	21
2	350	127.7	5.9	0.00	32	2.53	403	74	74
3	300	136.2	7.0	0.00	71	2.41	261	48	45

Test Results	Material Description
R-value at 300 psi exudation pressure = 53	Dark Brown Gravelly SAND w/Traces of Asphalt

BALBOA PARK BART UPPER YARD San Francisco, California	**R-VALUE TEST**
DC / RR JOINT VENTURE	Date 5/31/13 | Project No.13-050202 | Figure B-1

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/05/2013
Date Submitted 04/01/2013

To: Linda Liang
Rockridge Geotechnical, Inc.
4379 Piedmont Ave
Oakland, CA 94611

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager \

The reported analysis was requested for the following location:
Location : 13502-BALBOA PARK Site ID : CPT-2-1 @ 0-5.
Thank you for your business.

* For future reference to this analysis please use SUN # 64235-132738.

EVALUATION FOR SOIL CORROSION

Soil pH	7.39	
Moisture	3.5 %	
Minimum Resistivity	3.75 ohm-cm (x1000)	
Chloride	3.6 ppm	00.00036 %
Sulfate	42.3 ppm	00.00423 %
Redox Potential	(+) 176 mv	

Sulfate Reducing Bacteria Presence - NEGATIVE

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422
Redox Potential ASTM D1498m, Sulfate Reducing Bacteria AWWA C105-72

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/05/2013
Date Submitted 04/01/2013

To: Linda Liang
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General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 13502-BALBOA PARK Site ID : CPT-2-1 @ 0-5.
Thank you for your business.

* For future reference to this analysis please use SUN # 64235-132738.

Extractable Sulfide Analysis

TYPE OF TEST	RESULTS	UNITS
Sulfide	ND	mg/kg

DETECTION LIMITS
Sulfide 0.05

Method 9031m, ND = Below Detection Limit

DC / RR JOINT VENTURE

APPENDIX C

BART General Guidelines

SAN FRANCISCO BAY AREA RAPID TRANSIT DISTRICT

GENERAL GUIDELINES FOR DESIGN AND CONSTRUCTION OVER OR ADJACENT TO BART'S SUBWAY STRUCTURES

1. Structures over or adjacent to BART's subway structures shall be designed and constructed so as not to impose any temporary or permanent adverse effects on subway. The minimum clearance between any part of the adjacent structures to exterior face of substructures shall be 7'-6". Minimum cover of 8 feet shall be maintained wherever possible.

2. In general, cut-and-cover subway structures were designed with an area surcharge applied at the ground surface both over and adjacent to the structures. The area surcharge was considered static uniform load with the following value:

D (ft)	Additional Average Vertical Loading (psf)
D>20	0
5<D<20	800-40D
D<5	600

Where **D** is the vertical distance from the top of the subway roof to the ground surface.

3. In general, steel-lined tunnels were designed to support the weight of 35 feet of earth above the roof of the tunnel. Whenever the actual depth of cover is less than this amount, construction may be added imposing an additional average vertical loading of 120 lbs. per square foot for each foot of depth of reduced cover. Where basements are excavated, the allowable additional average vertical loading can be increased to the extent that it is balanced by the weight of the removed material. The effects of soil rebound in such cases shall be fully analyzed.

4. Shoring is required for excavations in the Zone of Influence. Zone of Influence is defined as the area above a Line of Influence which is a line from the critical point of substructure at a slope of 1 ½ horizontal to 1 vertical (line sloping towards ground level).

5. Shoring shall be required to maintain at-rest soil condition and monitored for movement.

6. Soil redistribution caused by temporary shoring or permanent foundation system shall be analyzed.

7. Dewatering shall be monitored for changes in groundwater level. Recharging will be required if existing groundwater level is expected to drop more than 2 feet.

8. Piles shall be predrilled to a minimum of 10 feet below the Line of Influence. Piles shall be driven in a sequence away from BART structures. No pile will be allowed between steel-lined tunnels.

9. Subway structures shall be monitored for vibration during pile driving operations for all piles within 100 feet of the structures. Steel –lined tunnels shall also be monitored for movement and deformation. Requirements for monitoring will be provided upon request.

10. Excavation shall be done with extreme care to prevent damage to the waterproofing membrane and the structure itself. Hand excavation shall be performed for the final one foot above the subway roof.

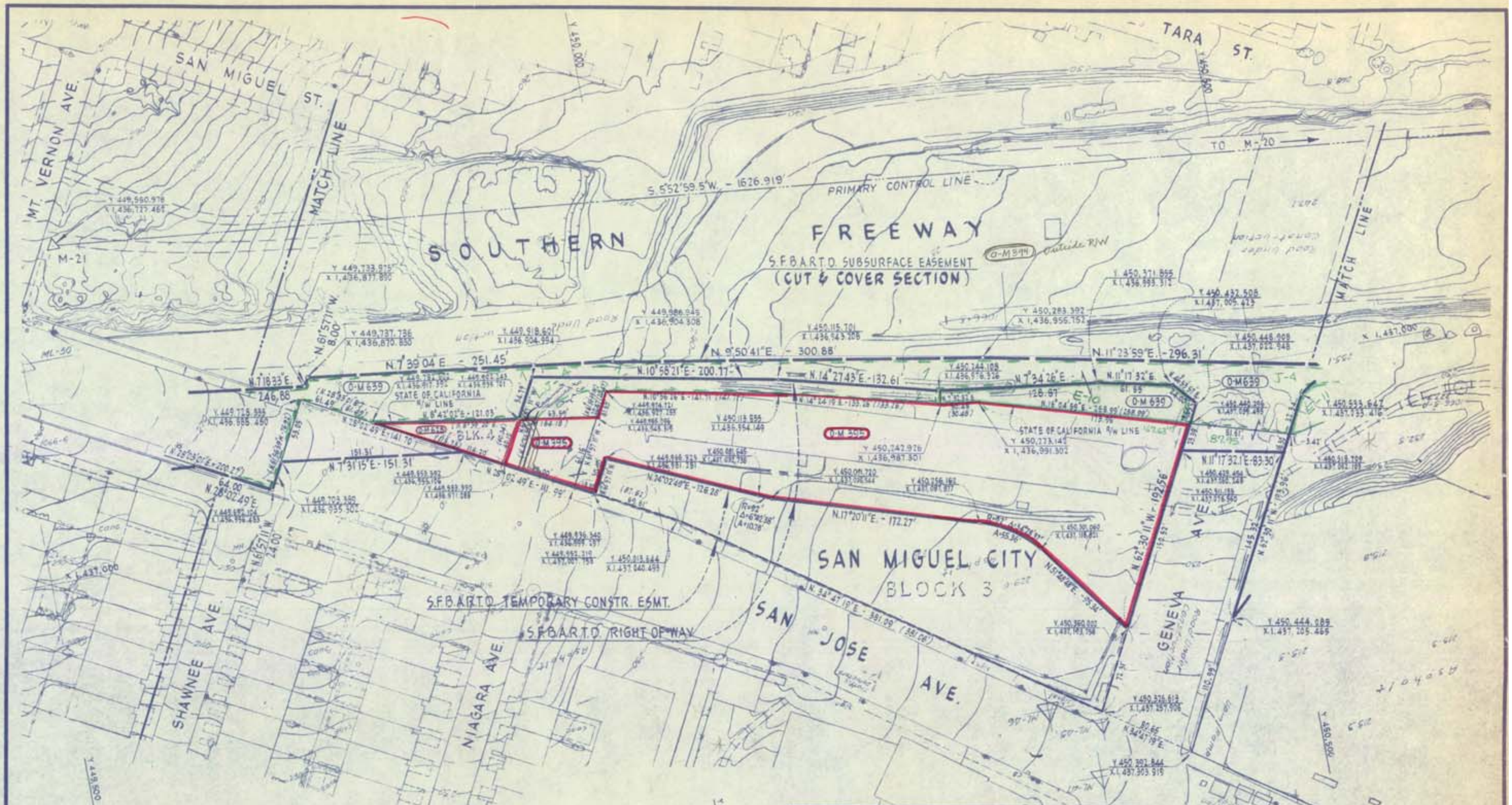
The above shall be considered as general information only and is not intended to cover all situations. Notwithstanding these guidelines, pertinent design and construction documents shall be submitted to BART for review and approval. In addition, the following shall be submitted as applicable:

- Geologic Hazards Evaluation and Geotechnical Investigation reports. The reports shall include engineering geology map, site plan showing the location of subway structures, BART easement, soil reworking plan and the geological conclusion and recommendations.
- Dewatering monitoring and recharging plans.
- Vibration monitoring plan and/or movement and deformation monitoring plans for steel-lined tunnels. Plans shall include locations and details of instruments in subways.
- Foundation plan showing the anticipated total foundation loads.
- Excavation plan for area within the Zone of Influence showing excavation slope or shoring system.
- Procedures and control of soil compaction operation.

DC / RR JOINT VENTURE

APPENDIX D

BART Easement Map



PARCEL No	NAME	TOTAL AREA	TAKE	REMAINING AREA LEFT	REMAINING AREA RIGHT
O-M 395	City & County of San Francisco	79,361 sf	48,658 sf		
	Temporary Construction Esmt		30,703 sf		
O-M 639	See Sheet MRW 9 JUA				
O-M 628		2,290 sf	2,290 sf	[illegible]	

DATE	REVISION	SUBMITTED	APPROVED
2-20-68	Changed Title	O.L.	
1-18-68	Added Temp. Constr. Esmt - Pcl. O-M 395	O.L.	
8-25-67	Revised Pcl. O-M 628	O.L.	
8-17-67	Added Pcl. O-M 628	O.L.	
7-25-67	Changed Dimension O.M 639	O.L.	
7-6-67	Changed Right of Way - Niagara To Geneva Ave.	O.L.	
9-12-66	Corrected Coordinate - Y 449,755.389 / X 1,436,735.305	O.L.	
8-29-66	Added Note "Cut & Cover Sect." to Subsurface Esmt.	O.L.	
8-1-66	Changed Sub-Surface Esmt bet. Shawnee & Niagara Ave.	O.L.	
6-13-66	Revised Pcl. "Area Schedule"	O.L.	
1-28-66	Changed Right of Way - Niagara To Geneva Ave.	O.L.	
11-2-65	Changed Package No		
6-29-65	Bearing & Dist Corrected from [illegible] To [illegible]		
6-22-65	Bearing & Dist. corrected from [illegible] to [illegible]		

Daniel P. Coleman

Bearings and distances are in the California Coordinate System, Zone III
Multiply distances by 1.0000755 to obtain ground level distances.

SAN FRANCISCO BAY AREA RAPID TRANSIT DISTRICT
RIGHT OF WAY MAP

BALBOA PARK (5 ½)

SAN FRANCISCO - MISSION LINE

MODOC AVE. TO COLONIAL WAY

PRIMARY TRAVERSE STA TO STA BALBOA STA
PARSONS BRINCKERHOFF - TUDOR - BECHTEL ENGINEERS
DATE APRIL 30, 1965 SCALE 1" = 50'
DANIEL COLEMAN ENGINEERING CO. SURVEYORS
PACKAGE M005 CONTRACT M 505 SHEET MRW 10