Wickham, Jerry, Env. Health

From: Wickham, Jerry, Env. Health
Sent: Tuesday, June 03, 2014 9:21 AM
To: 'Peter Sims'
Subject: RO3122: Ashland Housing Project Fill Import

Peter,

Based on the information provided, Alameda County Environmental Health has no objections to the use of imported fill from the Evelyn-Marshall site in Sunnyvale, CA described in the submitted documents. We request that the imported fill be periodically inspected by a qualified environmental professional to assure that the imported fill is consistent with the assumed conditions. Undocumented fill or fill that is not consistent with the described native soil from the Evelyn-Marshall site is not to be placed at the Ashland Housing site. The inspections and placement of the fill are to be documented and described in the Remedial Action Completion Report that is to be submitted following completion of site grading.

Regards,
Jerry Wickham
Alameda County Environmental Health
1131 Harbor Bay Parkway
Alameda, CA 94502-6577
phone: 510-567-6791
jerry.wickham@acgov.org

From: Peter Sims [mailto:psims@ninyoandmoore.com]
Sent: Monday, June 02, 2014 3:25 PM
To: Wickham, Jerry, Env. Health
Subject: Ashland Housing Project Fill Import

Hi Jerry,

Ashland has found a source that appears to me to be acceptable for import. Please review the attached data as well as the answers to your list of required information below:

1) Some background on environmental conditions at the site where the fill comes from. Some documentation such as a Phase I report or other information from a qualified professional indicating whether the site has any known or suspected environmental conditions.

Phase I ESA is attached.

2) The sample location and volume that each sample represents such as does the sample go with a stockpile of a certain volume.

Sample location map is attached. 1,500 cubic yards of soil will be imported from the northern property (Evelyn-Marshall). The soil samples were collected in-situ. The fill soil will be excavated from 0 to 5 feet bgs and will be direct loaded at the source property and exported to the Ashland site.

3) The type of samples - composite or discrete and how they were collected.

Discrete in-situ samples were collected from the source property.

4) The type of fill and the heterogeneity.

The fill is clayey sand or sandy clay from 0 to 5 feet bgs with some gravel at shallower depths (see attached geotechnical report for soil descriptions/boring logs).

5) Whether the fill contains any debris, construction material, baserock, or other non-native materials.

Non-native materials were not reported in the boring logs of the geotechnical report.

6) Whether any staining or odor was observed.

Staining and odor were not reported.

7) Where the soil is to be used at the site. In this case, will the soil be used in housing areas or under a street?

Fill soil is to be used across the site beneath both housing and parking/street areas.

8) Whether this is a variance from the Work Plan.

The Work Plan did not address import fill.

9) Laboratory analytical results.

See attached.

Peter D. Sims, LEED AP
Project Environmental Geologist
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants
1956 Webster Street, Suite 400
Oakland, California 94612
(510) 343-3000 x15216 (Office)
(510) 327-9335 (Cell Phone)
(510) 343-3001 (Fax)
psims@ninyoandmoore.com

New San Jose office
2149 O'Toole Avenue, Suite 10
San Jose, CA 95131
(408) 435-9000
(408) 435-9006 (Fax)

Experience · Quality · Commitment

Geotechnical Investigation

Evelyn Avenue Apartments
457 and 477 East Evelyn Avenue
Sunnyvale, California

Report No. 189340 has been prepared for:

Prometheus Real Estate Group

1900 South Norfolk Street, Suite 150, San Mateo, California

January 10, 2012

Alberto Cortez Staff Engineer	Scott M. Leck, P.E., G.E. Principal Geotechnical Engineer	Erin L. Steiner, P.E., G.E. Senior Project Engineer Quality Assurance Reviewer

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GEOTECHNICAL INVESTIGATION
EVELYN AVENUE APARTMENTS
457 AND 477 EAST EVELYN AVENUE
SUNNYVALE, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the Evelyn Avenue Apartments development located in Sunnyvale, California. The site location is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the geologic and subsurface conditions and to provide geotechnical recommendations for design of the proposed project.

For our use, we received a parcel map and a conceptual project site plan titled, “Evelyn Avenue Apartments, Sunnyvale, California,” prepared by Studio T Architects, dated November 15, 2011.

1.1 Project Description

The site is located at 457 and 477 East Evelyn Avenue in Sunnyvale and is currently occupied by two single-story buildings, paved parking areas, driveways and landscaping. The layout of the proposed development is shown on the Site Plan, Figure 2. The site is bordered by the Caltrain Peninsula Corridor to the northeast, an apartment complex to the northwest, Evelyn Avenue to the southwest, and Marshall Avenue and an apartment complex to the southeast.

As presently planned, the project consists of developing the site with apartment homes. The structures will occupy nearly the entire site and will consist of one level of below grade parking with four stories of wood framed units above. The below grade parking level will be approximately 8 feet below the existing site grade. Additional improvements will include pavements, underground utilities, and landscaping. Structural loads have not been provided to us; we assume that structural loads will be representative for this type of construction.

1.2 Scope of Services

Our scope of services was presented in our agreement with you dated December 1, 2011. To accomplish this work, we have provided the following services:

- Exploration of subsurface conditions by drilling two borings in the area of the proposed development and retrieving soil samples for observation and laboratory testing. We also advanced two Cone Penetration Tests (CPTs).

- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.

- Engineering analysis to evaluate building foundations, site earthwork, slabs-on-grade, and pavements.

- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

2.0 SITE CONDITIONS

2.1 Site Reconnaissance

Our Staff Engineer performed a reconnaissance of the site on December 16, 2011. At the time of the reconnaissance, the site was occupied with commercial buildings, paved parking areas and driveways and appeared relatively flat with minor grade variation for drainage purposes.

2.2 Exploration Program

Subsurface exploration was performed on December 16, 2011 using conventional, hollow-stem auger truck-mounted drilling equipment and CPT equipment to investigate, sample, and log subsurface soils. Two hollow-stem auger exploratory boring were drilled to depths of 45 feet. Also, Two CPTs were advanced to depths of 45 feet. Our borings were backfilled in accordance with the Santa Clara Valley Water District guidelines. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. The logs of these borings and details regarding our field investigation are included in Appendix A; laboratory tests are discussed in Appendix B.

2.3 Subsurface Conditions

Our borings encountered a pavement section consisting of 1½ inches of asphalt concrete underlain by 2½ to 7 inches of aggregate base. Soils encountered in our CPTs were generally interpreted to include interbedded layers consisting of clay, silty clay, sandy silt, silty sand, clayey sand, gravelly sand, and poorly graded sand. The CPT interpretations were generally consistent with our experience in the area, and the conditions encountered in our borings. Undrained shear strengths indicated soft to hard silty and clay soils. Granular layers encountered were interpreted as medium dense to hard. Our exploratory borings generally encountered interbedded layers consisting of stiff sandy silt, medium stiff to very stiff lean clay, stiff to very stiff sandy lean clay, medium dense to dense clayey sand, medium dense poorly graded gravel, and medium dense to very dense poorly graded sand to a depth of 45 feet, the maximum depth explored. Sieve analysis indicated percent passing the -#200 sieve ranged from 20 to 45 percent for the tested sand samples indicating sands with significant amount of fines.

A Plasticity Index (PI) test was performed on a representative clayey soil sample in boring EB-2 at a depth of approximately 2 feet. The tests resulted in a PI of 10, indicating low plasticity and expansion potential of the near surface soils.

2.4 Ground Water

Free ground water was encountered during subsurface exploration in both exploratory borings at a depth of approximately 31½ feet. Based on pore pressure dissipation measurements, our CPTs encountered groundwater at a depth of 28½ feet below grade. The ground water depth was measured at the time of drilling and may not reflect a stabilized level. According to the depth to historically high ground water map, prepared by the California Geological Survey for the Mountain View Quadrangle (CGS, 2006), the depth to historically high ground water levels in the site vicinity are on the order of 25 feet below ground surface (bgs). Based on the above information, we judge a design ground water depth of 25 feet below the ground surface to be appropriate for geotechnical design purposes. All borings were backfilled immediately after drilling. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone). The site is also not located in a Santa Clara County Fault Rupture Hazard Zone (2002) for fault rupture. As shown on Figure 3, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Maximum Estimated Ground Shaking

The current California Building Code (CBC) method indicates a peak ground surface acceleration of 0.40g, which is equal to the Design Spectral Response Acceleration Parameter (S_{DS}) divided by 2.5 as discussed in Section 1803.5.12 of the 2010 CBC.

3.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (WGCEP, 2007) estimates there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the San Francisco Bay region between 2007 and 2037. This result is an important outcome of WGCEP's work because any major earthquake can cause damage throughout the region. The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault epicenter.

Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.4 Liquefaction

The site is adjacent to, but not located within an area zoned by the State of California as having potential for seismically induced liquefaction hazards (CGS, 2006). The site is also not located within an area zoned by the Santa Clara County Geologic Liquefaction Hazard Zones (2002). During cyclic ground shaking, such as during earthquakes, cyclically induced stresses may cause increased pore water pressures within the soil matrix, resulting in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure under moderate to high shear stresses, such as beneath foundations or sloping ground (NCEER/NSF, 2001), and in many ways may behave more like a liquid than a solid. Liquefied soil can also settle (compact) as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, in some cases, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured.

Soils most susceptible to liquefaction are loose to moderately dense, saturated non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

As mentioned above, because the site is located adjacent to a liquefaction hazard zone, we evaluated the site for potential liquefaction if it were to occur. We performed our liquefaction analyses following the methods presented by the 1998 NCEER Workshops (Youd et al., 2001) in accordance with guidelines set forth in the CGS Special Publication 117A (2008). The NCEER methods for Standard Penetration Tests (SPT) and CPT analysis update simplified procedures presented by Seed and Idriss (1971). In broad terms, these methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

Based on our explorations and the depth to historic high ground water map prepared by the CGS, a design ground water level at 25 feet below the existing site grade was used for our liquefaction analysis. As discussed in the subsurface description above, some medium dense sand layers were encountered below the design ground water depth. These layers were evaluated to assess liquefaction potential and the effects liquefaction may have on the proposed development. No liquefaction analyses were performed on layers above the design ground water depth.

Our analyses indicate that some thin sand layers below the design ground water depth may potentially theoretically liquefy resulting in up to ½-inch of total settlement. Volumetric change and settlement were estimated using the Ishihara and Yoshimine (1990) method. As discussed in the Southern California Earthquake Center report (SCEC, 1999), differential movement for level ground deep soil sites will be on the order of half the total estimated settlement. We estimate differential settlements from potential liquefaction if it were to occur to be on the order of ¼-inch in 50 feet.

3.5 Dry Seismic Settlement

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform densification of loose to medium dense cohesionless soil strata. This results in movement of the near-surface soils. Based on the corrected SPT blow counts and laboratory testing data, it appears that there are several medium dense sand above the design groundwater depth that may densify during a strong earthquake.

We performed dry sand settlement calculations following the Tokimatsu and Seed method (1987) and using a peak ground acceleration of 0.40g. Our calculations indicated that during a seismic event the medium dense sand and gravel layers in our borings may densify and settle less than ¼-inch.

3.6 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or “free” face such as an open body of water, channel or excavation. In soils, this movement is generally due to failure along a weak plane and may often be associated with liquefaction. There are no creeks or open bodies of water within an appropriate distance from the site for lateral spreading to occur. For this reason, the probability of lateral spreading occurring at the site during a seismic event is low.

4.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical engineering viewpoint, the proposed residential development may be constructed as planned, in our opinion, provided the design and construction are performed in accordance with the recommendations presented in this report.

The primary geotechnical and geologic concerns at the site are as follows:

- Strong seismic shaking
- Liquefaction settlement
- Demolition of the existing buildings and pavements prior to site development
- Differential settlement between at-grade and below grade portions of the structure

We have prepared a brief description of the issues and present typical approaches to manage potential concerns associated with the long-term performance of the development.

4.1.1 Strong Seismic Shaking

We recommend that, at a minimum, the proposed residential structures be designed in accordance with the seismic design criteria of the 2010 CBC. Site seismic coefficients are presented in the "Foundations" section below.

4.1.2 Liquefaction Settlement

Liquefaction could result in an additional ½-inch of settlement with differential settlements of approximately ¼-inch across a horizontal distance of 50 feet. The proposed structures should be designed to accommodate the resulting seismic and static settlements. Detailed recommendations are presented in the Foundation section of this report.

4.1.3 Demolition Debris

A significant amount of construction debris both above and below grade is anticipated as a result of the site demolition required prior to site grading. The debris should be either: 1) collected and off-hauled to an appropriate facility prior to beginning the earthwork for the project, or 2) the concrete crushed and re-used as fill at the site. It has been our experience that some debris will remain in the soil on-site after the demolition contractor has completed their work. Therefore, it should be anticipated that some debris would be encountered in excavations for underground utilities and foundations. It has been our experience that some coordination between the demolition contractor, grading contractor and geotechnical engineer is needed to identify the scope of the excavation backfill and other similar work items. Recommendations for re-use of recycled materials are presented in the Earthwork section of this report.

4.1.4 Differential Settlement

It is possible that differential settlement could occur between the at-grade and below-grade portions of the structure that could adversely affect the driveway ramp(s). We recommend that the ramps be designed to accommodate at least ½ inch of movement between the at-grade portions of the site and below-grade slab.

4.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and confirm the geotechnical specifications of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation and, if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

5.0 EARTHWORK

5.1 Clearing and Site Preparation

The proposed project area should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, utilities, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. All existing landscaping and any associated root systems should be completely removed. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the “Compaction” section of this report. We recommend that backfilling of holes or pits results from demolition and removal of existing building foundations, buried structures or other improvements be carried out under our observation and that the backfill be observed and tested during placement.

After clearing, any vegetated areas within the proposed improvements should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

5.2 Removal of Undocumented Fill

If undocumented fill is encountered, it should be removed down to the native soil. If the fill material meets the requirements in the “Material for Fill” section below, it may be reused an engineered fill. Side slopes of fill excavations in building and pavement areas should be sloped at inclinations no steeper than 3:1 (horizontal:vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the “Compaction” section of this report.

5.3 Abandoned Utilities

Abandoned utilities within the proposed building areas should be removed in their entirety. Utilities within the proposed building area would only be considered for in-place abandonment provided they do not conflict with new improvements, that the ends and all laterals are located and completely grouted, and the previous fills associated with the utility do not pose a risk to the structure.

Utilities outside the building areas should be removed or abandoned in-place by grouting or plugging the ends with concrete. Fills associated with utilities abandoned in-place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged.

5.4 Reuse of On-site Recycled Materials

We anticipate that significant amounts of asphalt concrete/aggregate base grindings and concrete will be generated during removal of the existing pavements and buildings. If it is desired to reuse the grindings for new site pavement structural support, we recommend the asphalt concrete be pulverized and mixed with the underlying aggregate base to meet Caltrans Class 2 Aggregate Base requirements. If laboratory testing of the recycled material indicates that it meets Caltrans Class 2 specifications, it may be used as Class 2 aggregate base beneath pavements and sidewalks. Recycled material containing asphalt concrete grindings should not be used in building areas. Laboratory testing may be performed on initial grindings generated to evaluate the material further and refine the pavement recommendations.

5.5 Subgrade Preparation

After the site has been properly cleared, stripped and necessary excavations have been made, exposed surface soils in those areas to receive fill or pavements should be scarified to a depth of 8 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the “Compaction” section. The finished compacted subgrade should be firm and relatively non-yielding under the weight of compaction equipment.

5.6 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension.

Import fill material should be inorganic, have a PI of 15 or less and should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Samples of the proposed import fill should be submitted to us at least 10 working days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials. Import soils should not be more corrosive than the on-site native materials, including pH, soluble sulfates, chlorides and resistivity.

5.7 Compaction

All fill, as well as scarified surface soils in those areas to receive fill, should be uniformly compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content at least 1 percent over laboratory optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and relatively non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition), except for the native clays, which should be compacted as noted above. Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum moisture content.

5.8 Wet Soils and Wet Weather Conditions

Earthwork such as subgrade preparation, fill placement and trench backfill may be difficult for soil containing high moisture content or during wet weather. If the soil is significantly above its optimum moisture content, it will become soft, yielding, and difficult to compact. Based on the results of our laboratory tests, the in-situ moisture contents of the near surface soils are generally near optimum moisture contents. If saturated soils are encountered, aerating or blending with drier soils to achieve a workable moisture content may be required. We recommend that earthwork be performed during periods of suitable weather conditions, such as the “summer” construction season.

There are several alternatives to facilitate subgrade preparation, fill placement and trench backfill if the soil is wet or earthwork is performed during the wet winter season.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction,
- Over-excavate the fill and replace with suitable on-site or import materials with an appropriate moisture content,
- Install a layer of geo-synthetic (geotextile or geogrid) to reduce surface yielding and bridge over soft fill,
- Chemically treat the higher moisture content soils with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill.

The implementation of these methods should be reviewed on a case-by-case basis so that a cost effective approach may be used for the specific conditions at the time of construction.

5.9 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer’s recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the “Material for Fill” section of this report. Contractors should plan on drying the native clay soils prior to reuse as engineered fill. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the footing bearing surface to the bottom edge of the trench. Where utility trenches will cross beneath footing bearing planes, the footing concrete should be deepened to encase the pipe or the utility trench should be backfilled with sand/cement slurry or lean concrete within the foundation-bearing plane.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas, and coming into contact with expansive subgrade soils.

5.10 Temporary Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

5.11 Temporary Shoring Support System

As previously discussed, excavations on the order of 8 feet are planned to construct the proposed apartments with a one level of below grade parking. The excavations could potentially be temporarily supported by several methods including tiebacks, soil nailing, braced shoring, temporary slopes if space is adequate, or potentially other methods. Where shoring is required, restrained shoring will most likely be necessary to limit deflections and disruption to nearby improvements. It has been our experience that cantilever shoring might be feasible for temporary shoring to a height of about 10 to 12 feet where allowable deflections are limited. The choice of shoring method should be left to the contractor’s judgment since economic considerations and/or the individual contractor’s construction experience may determine which method is more economical and/or appropriate. However, other factors such as the location of nearby utilities and encroachment on adjacent properties may influence the choice of support.

The temporary shoring should be designed for additional surcharges due to adjacent loads such as from construction vehicles, street traffic and adjacent sound wall. To prevent excessive surcharging of the walls, we recommend that heavy loads such as construction equipment and stockpiles of materials be kept at least 15 feet from the top of the excavations. If this is not possible, the shoring must be designed to resist the additional anticipated lateral loads. Shoring systems should be designed with sufficient rigidity to prevent detrimental lateral displacements. Minimum geotechnical parameters for design of a temporary shoring system are given in Table 1.

Table 1. Temporary Shoring System Design Parameter

Design Parameter	Design Value (psf)
Minimum Lateral Wall Surcharge[1]	120 psf
Earth Pressure – Cantilever Wall	40 pcf
Earth Pressure – Restrained Wall[2] From ground surface to H/4 (ft)	Increase from 0 to 25H psf
Earth Pressure – Restrained Wall Below H/4 (ft)	Uniform pressure of 25H psf
Passive Pressure[3]	400 pcf up to 2,000 psf max

Note: 1 For the upper 5 feet (minimum for incidental loading)
2 Where H equals height of excavation
3 Can assume to act over 2 times the diameter of soldier piles, neglecting the upper foot

To limit potential movements of the shoring system, the shoring designer and contractor should consider several design and construction issues. For the movements of shoring to be reduced, the designer will have to provide for a uniform and timely mobilization of the soil pressures. Tiebacks or internal bracing should be loaded to the design loads prior to excavation of the adjacent soil so that load induced strains in the retaining system will not result in the system moving toward the excavation. In addition, a relatively stiff shoring system should be designed to limit deflections under loading. In general, we recommend designing a shoring system to deflect less than 1-inch.

In addition, ground subsidence and deflections can be caused by other factors such as voids created behind the shoring system by over-excavation, soil sloughing, erosion of sand or silt layers due to perched water, etc. All voids behind the shoring system should be filled as soon as feasible by grouting to minimize potential problems during installation of the shoring system.

Since we drilled our borings with hollow-stem auger drilling equipment, we are not able to evaluate the potential for caving of on-site soils, which may become a factor during soldier pile and/or tieback installation. The contractor is responsible for evaluating excavation difficulties prior to construction. Pilot holes using proposed production drilling equipment may be prudent, to evaluate possible excavation difficulties such as caving soils, cobbles, boulders and/or other excavation difficulties.

In conjunction with the shoring installation, a monitoring program should be set up and carried out by the contractor to determine the effects of the construction on the adjacent sound wall, street and other improvements such as sidewalks and utilities. As a minimum, we recommend horizontal and vertical surveying of reference points on the shoring and on the adjacent street, sound wall and other improvements in addition to an initial crack survey. We also recommend that all supported and/or sensitive utilities be located and monitored by the contractor. Reference points should be set up and read prior to the start of construction activities. Points should also be set on the shoring as soon as initial installations are made.

This report is intended for use by the design team. The contractor should perform additional subsurface exploration and/or geotechnical studies as they deem necessary for the chosen shoring system. The contractor is also responsible for site safety and the means and methods of construction, including temporary shoring. Temporary shoring must be designed by a licensed California Civil or Structural Engineer. Prior to construction, we recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for preconstruction review.

5.12 Surface Drainage

Positive surface water drainage gradients, at least 2 percent in landscaping and 0.5 percent in pavement areas, should be provided to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

5.13 Landscaping Considerations

We recommend restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low flow rate sprinkler heads, or drip irrigation systems
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,

- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that the landscape architect consider these items when developing the landscaping plans.

5.14 Construction Observation

A representative from our company should observe the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your construction schedule we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

6.0 FOUNDATIONS

Provided that the site is prepared in accordance with the “Earthwork” section of this report and the proposed apartment structures with one level of below grade parking can be designed to accommodate the following estimated amounts of settlement, the structures may be supported on spread footings or reinforced mat foundations as discussed in the sections below.

6.1 2010 CBC Site Coefficients and Site Seismic Coefficients

Chapter 16 of the 2010 California Building Code (CBC) outlines the procedure for seismic design of structures. Based on our explorations, and review of available geologic maps (Rogers and Williams, 1974), the site is underlain by medium stiff to hard soils extending to depths on the order of greater than 500 feet, which corresponds to a soil profile type D. Based on the above information and local seismic sources, the site may be characterized for design using the information in Table 2 below.

Table 2. 2010 CBC Site Class and Site Seismic Coefficients

Latitude: 37.3764 N **Longitude: 122.0251 W**	**CBC Table/ Figure**	**Factor/ Coefficient**	**Value**
Soil Profile Type	Table 1613.5.2	Site Class	D
Mapped Spectral Response Acceleration for MCE at 0.2 second Period	Figure 1613.5(3)	S_s	1.5
Mapped Spectral Response Acceleration for MCE at 1 Second Period	Figure 1613.5(4)	S_1	0.61
Site Coefficient	Table 1613.5.3(1)	F_a	1.0
Site Coefficient	Table 1613.5.3(2)	F_v	1.5

Adjusted MCE Spectral Response Parameter	Equation 16A-37	S_{MS}	1.5
Adjusted MCE Spectral Response Parameter	Equation 16A-38	S_{M1}	0.91
Design Spectral Response Acceleration Parameter	Equation 16A-39	S_{DS}	1.0
Design Spectral Response Acceleration Parameter	Equation 16A-40	S_{D1}	0.61

6.2 Footings

The proposed apartment structures may be supported on conventional spread footings bearing on natural, undisturbed soil or compacted engineered fill. All footings should have a minimum width of at least 18 inches and footing bottoms should extend at least 24 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower.

Footings constructed on native soil or engineered fill in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. We should observe all footing excavations before reinforcing steel is placed.

6.3 Foundation Settlement

Structural loads were not available for our review at the time of our investigation. Therefore, we assumed typical interior column dead plus live loads on the order of 300 kips for our settlement analysis. Based on the assumed load and the maximum allowable bearing pressures recommended above, we estimate that total static settlement for footings will be approximately ½-inch, with differential settlements of ¼-inch over a horizontal distance of 50 feet. We estimate differential settlements from liquefaction will be on the order of ¼-inch in 50 feet. We should be retained to review the final foundation plans and structural loads to verify the above settlement estimates.

6.4 Reinforced Mat Foundations

The proposed apartment structures may alternatively be supported on conventionally-reinforced mat foundations. Based on the subsurface conditions, the mat may be designed for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads with maximum localized allowable bearing pressures of 3,000 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes.

The mat should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to permit spanning of local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that we observe the subgrade of the mat foundation prior to placement of reinforcing steel.

6.5 Modulus of Subgrade Reaction

For structural design of the mat, we recommend using a subgrade modulus that models the soil response under building loads. In developing the appropriate modulus of subgrade reaction (referred to as the "subgrade modulus"), we considered the varying soil conditions and stress distribution for the planned building layout. We recommend that a subgrade modulus of 30 pounds per cubic inch (pci) be used for design. We would be pleased to provide supplemental consultation in refining the soil subgrade modulus value, if desired. In order to proceed with further analysis, we would need the output from the first iteration of the SAFE analysis or other finite element analysis of the mat.

6.6 Mat Foundation Settlement

Based on the estimated areal mat pressure, we estimate that total settlement will be on the order of ½-inch for mats bearing at 8 feet below grade. If the mat foundation, designed in accordance with the above recommendations, is not capable of resisting up to ½-inch total and ¼-inch of differential movement from the center to the corner of the interior mat, respectively, additional reinforcement may be required.

As discussed in the "Liquefaction" section, differential settlement of mat foundations due to liquefaction settlement may occur during strong ground shaking. To reduce the potential impact of dry seismic settlement, the mats should also be designed to tolerate ¼-inch of differential settlement over a horizontal distance of 50 feet.

6.7 Lateral Loads

Lateral loads may be resisted by friction between the bottom of footings or mats and the supporting subgrade. A maximum allowable frictional resistance of 0.3 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against footings or deepened mat edges poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

6.8 Moisture Protection Considerations

Since the long-term performance of concrete mat foundations depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete mat of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with mat foundation construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 15-mil thick vapor barrier meeting minimum ASTM E 1745, Class A requirements should be placed directly below the mat. The vapor barrier should extend to the edge of the mat. At least 4 inches of free-draining gravel, such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break (no sand). The crushed rock should be consolidated in place with vibratory equipment. The vapor barrier should be sealed at all seams and penetrations.

- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.
- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels should not be permitted.
- All concrete surfaces to receive any type of floor covering should be moist-cured for a minimum of 7 days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.
- Water vapor emission levels and pH should be determined before floor installation as required by the manufacturer of the floor covering materials. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. It should be noted that the application of these guidelines does not affect the geotechnical aspects of the foundation performance.

7.0 CONCRETE SLABS-ON-GRADE

7.1 Garage Slabs (with Footings)

The one-level below-grade parking garage slab should be at least 5 inches thick, have a compressive strength of at least 3,000 pounds per square inch (psi), and supported on at least 6 inches of Class 2 aggregate base compacted to at least 95 percent relative compaction. Adequate slab reinforcement should be provided to satisfy the anticipated use and loading requirements.

8.0 BASEMENT WALLS

8.1 Lateral Earth Pressures

Any proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the wall. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

8.2 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively low permeable compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at 5 feet below existing site grades, or to some other closed or through-wall system. Miradrain panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

We recommend that design details for draining the basement walls above the design ground water level be determined prior to completion of construction documents as this is often a critical feature. A sump will likely be needed for drainage at this elevation unless storm drains are at an elevation that would accept the water by gravity. A suitable prefabricated drainage system designed for this specific use, such as Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting, is typical. The prefabricated drainage system should be installed against the wall (if excavation is laid back) or shoring system and should be installed in at least 4-foot-wide vertical strips at 8 feet on-center around the basement walls. Drainage panels should terminate 24 inches from final exterior grade. The drainage panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil. A horizontal collection system external to the basement walls, or carried inside the basement, should drain to a sump system. Waterproofing should be installed between the drainage system and the basement walls. The project structural engineer should review and approve any notch or penetrations planned in basement walls.

8.3 Backfill

Backfill placed behind the walls should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy compaction equipment is used, the walls should be temporarily braced.

8.4 Foundation

Basement walls may be supported on continuous spread footing or reinforced mat foundation designed in accordance with the recommendations presented in the “Footings” and “Reinforced Mat Foundation” sections of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations in the “Lateral Loads” section.

9.0 PAVEMENTS

9.1 Asphalt Concrete

Based on the near surface soils encountered and a Plasticity Index (PI) result of 10, we judge an R-value of 20 to be applicable for design. Using estimated traffic indices for various pavement-loading requirements, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 3.

Table 3. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value = 20

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking	4.0	2.5	5.5	8.0
	4.5	2.5	7.0	9.5
Automobile Parking Channel	5.0	3.0	7.0	10.0
	5.5	3.0	9.0	12.0
Truck Access & Parking Areas	6.0	3.5	9.5	13.0
	6.5	4.0	10.5	14.5

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study. Because of the presence of moderately expansive clay at the site, some increased amount of maintenance should be expected.

9.2 Exterior Portland Cement Concrete (PCC) Pavements

Recommendations for exterior PCC pavements are presented below in Table 4. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 4. Recommended Minimum PCC Pavement Thickness

Allowable ADTT	Minimum PCC Pavement Thickness (inches)
0.8	5
13	5½
130	6

Our design is based on an R-value of 20 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the PCC pavements to control the cracking inherent in this construction.

It is possible that differential settlement could occur between the at-grade and below-grade portions of the structure that could adversely affect the driveway ramp(s). We recommend that the ramps be designed to accommodate at least ½ inch of movement between the at-grade portions of the site and below-grade slab.

9.3 Pavement Cutoff

Surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, “Deep-Root Moisture Barrier,” or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cutoff barrier may be eliminated. If desired to install pavement cutoff barriers, they should be considered where pavement areas lay downslope of any landscape areas that are to be sprinkled or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

9.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the “Earthwork” section of this report.

9.5 Flatwork and Sidewalks

We recommend that exterior slabs-on-grade, such as flatwork and sidewalks be at least 4 inches thick and be underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. Subgrade soils should be moisture conditioned near the laboratory optimum moisture content prior to placing baserock. Reinforcing steel would be beneficial in reducing shrinkage cracking and vertical faulting at control and construction joints. We recommend that exterior slabs be isolated from adjacent foundations and that adequate construction and control joints be used in design of the concrete slabs to control cracking inherent in concrete construction. If sidewalks are subject to wheel loads, they should be designed in accordance with the “Exterior Portland Cement Concrete Pavements” section of this report.

10.0 LIMITATIONS

This report has been prepared for the sole use of Prometheus Real Estate Group, specifically for design of the proposed Evelyn Avenue Apartments in Sunnyvale, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC’s report by others. Furthermore, TRC will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

11.0 REFERENCES

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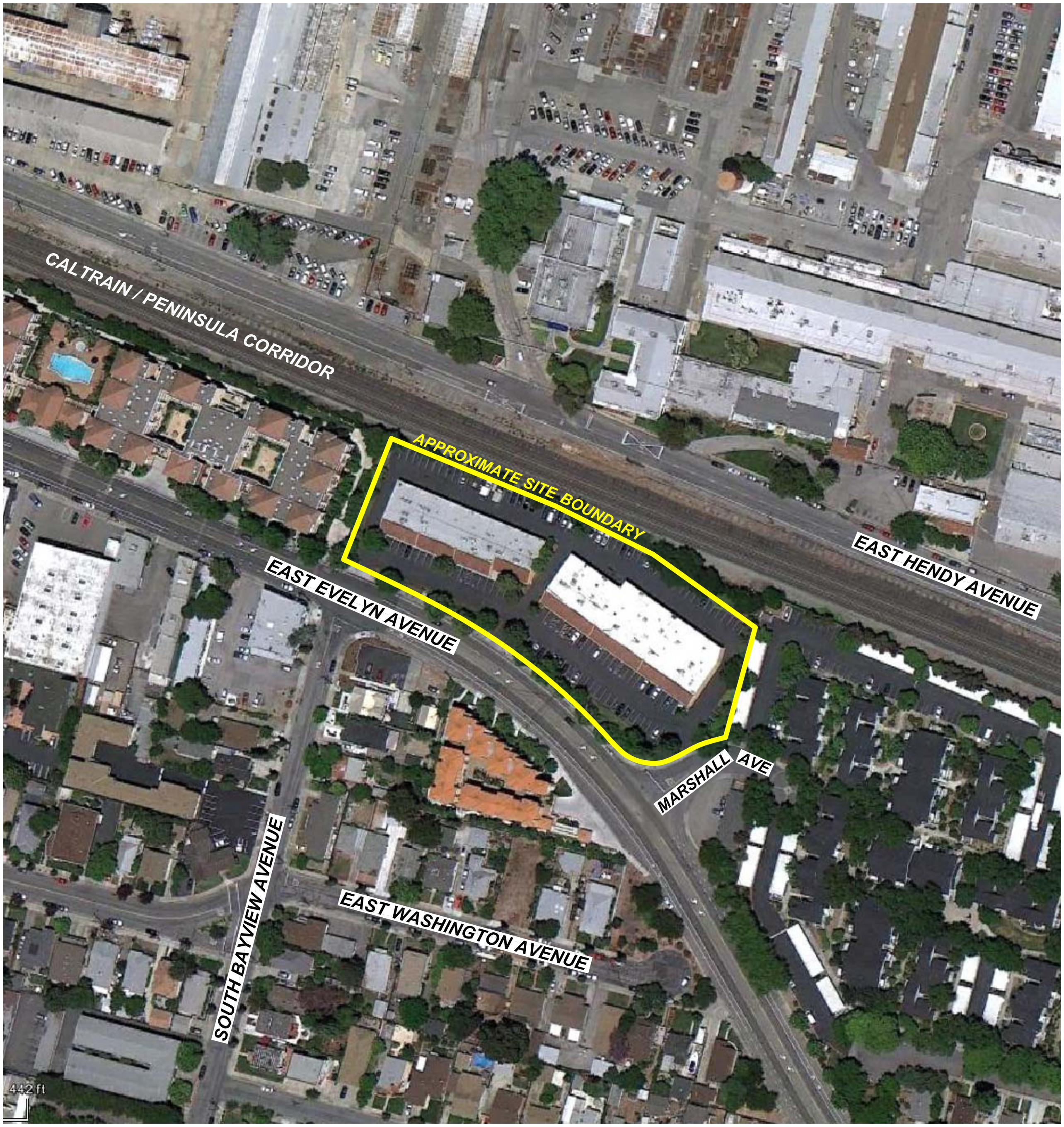
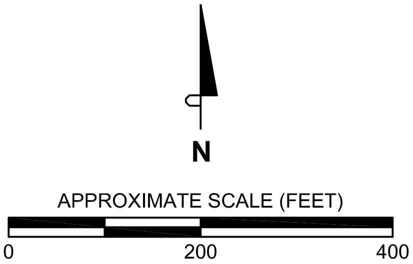




AERIAL PHOTO SOURCE:
Google Earth, June 2011.

VICINITY MAP

Evelyn Avenue Apartments
457 and 477 East Evelyn Avenue
Sunnyvale, California

TRC

189340

FIGURE 1

FILE NAME: Z:\Current\Evelyn Ave Apts_Sunnyvale\Fig1_Vicinity Map.dwg | Layout Tab: 8x11

SOURCE: Site Plan by Studio T Sq. for Prometheus Real Estate Group, November 2011.

LEGEND

- ⊕ Approximate location of exploratory boring
- △ Approximate location of cone penetration test

N

APPROXIMATE SCALE (FEET)

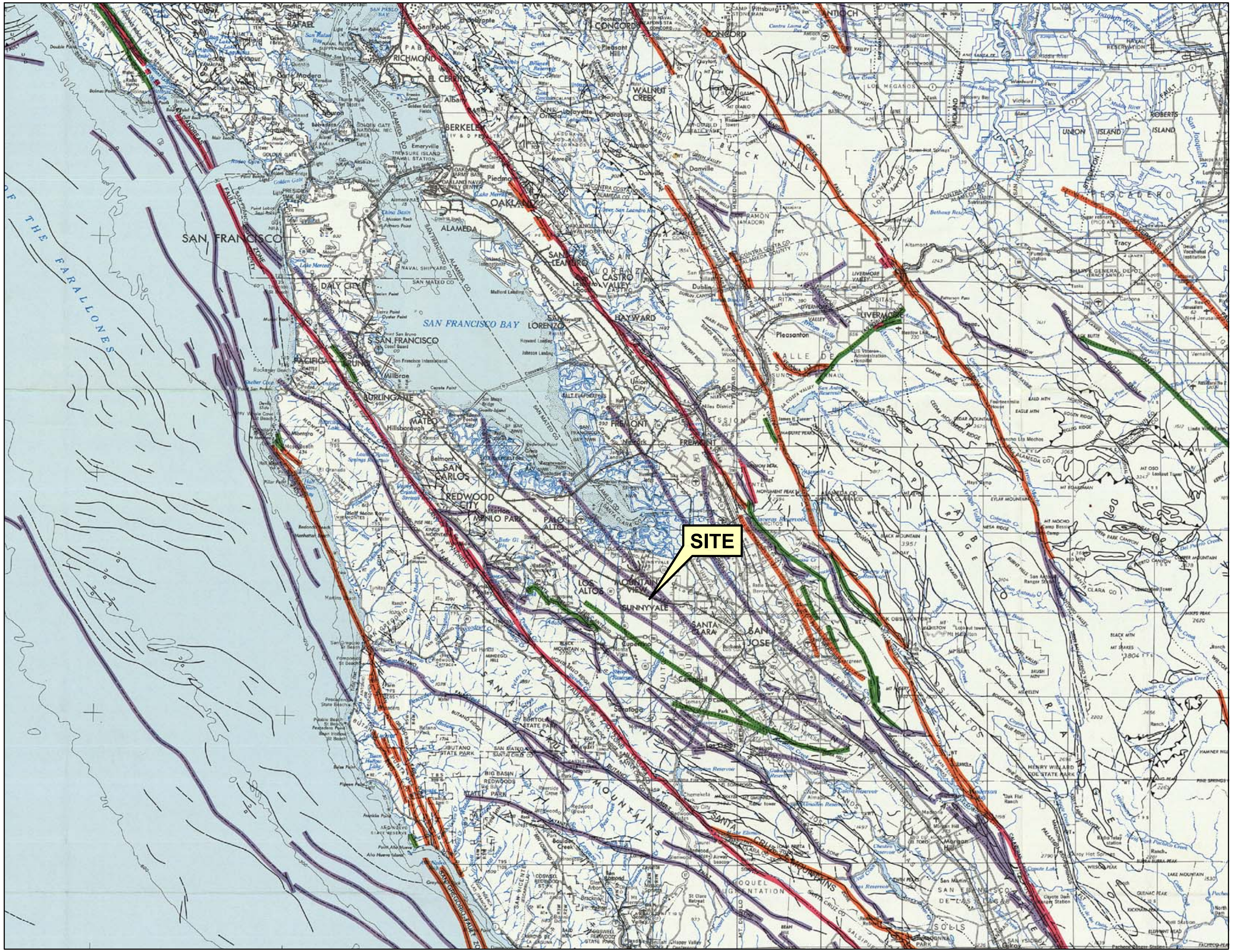
0 60 120

SITE PLAN

Evelyn Avenue Apartments
457 and 477 East Evelyn Avenue
Sunnyvale, California

TRC | 189340 | **FIGURE 2**

FILE NAME: Z:\Current\Evelyn Ave Apts_Sunnyvale\Fig2_Site Plan.dwg | Layout Tab: 8x11



SCALE: 1:500,000

APPROXIMATE GRAPHICAL SCALE (MILES)

0 10 20

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Holocene Historic	200	🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	10,000	🟧		Displacement during Holocene time.[2]
		Pleistocene	700,000	🟩		Faults showing evidence of displacement during late Quaternary time.[3,4]
	Early Quaternary	Pleistocene	2,000,000	🟪		Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary		Pliocene	5,000,000	~		Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.
		Miocene				

NOTES:

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122-A1-TF-250-00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120-A1-TF-250-00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000-meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

REGIONAL FAULT MAP

Evelyn Avenue Apartments
457 and 477 East Evelyn Avenue
Sunnyvale, California

TRC 189340 **FIGURE 3**

FILE NAME: Z:\Current\Evelyn Ave Apts_Sunnyvale\Fig3_Regional Fault Map.dwg | Layout Tab: 11x17

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted, hollow-stem auger drilling and cone penetration test (CPT) equipment. Two 8-inch-diameter exploratory borings were drilled on December 16, 2011 to a maximum depth of 45 feet. Two cone penetration tests (CPTs) were advanced on December 16, 2011 to a maximum depth of 45 feet. The approximate locations of the exploratory borings and CPTs are shown on Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix.

The locations of borings and CPTs were approximately determined by pacing from existing site boundaries. Elevations of the borings were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D.) samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the sum of the last two 6-inch increments is the uncorrected SPT measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

The attached boring and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * * * * *

EXPLORATORY BORING: EB-1

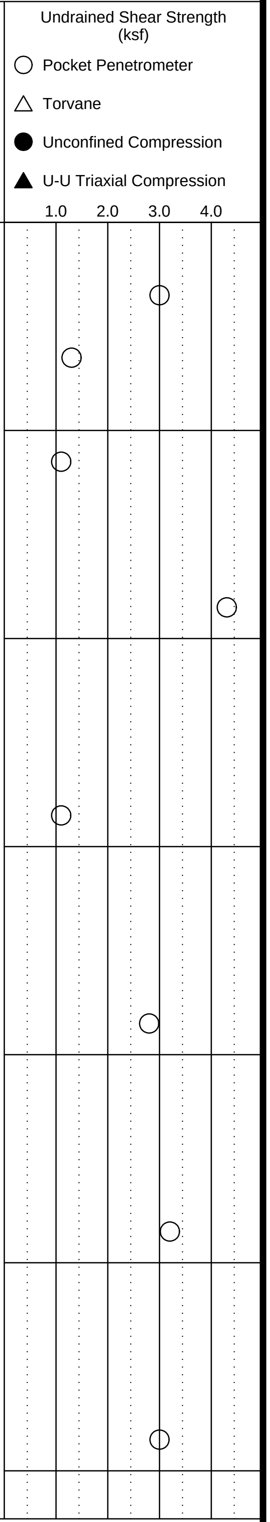
Sheet 1 of 2

DRILL RIG: MOBILE B-53
BORING TYPE: 8-INCH HOLLOW
LOGGED BY: AC
START DATE: 12-16-11 FINISH DATE: 12-16-11

PROJECT NO: 189340
PROJECT: Evelyn Ave Apartments
LOCATION: Sunnyvale, CA
COMPLETION DEPTH: 45.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	**1.5" of AC over 7" of AB**	AC					
		CLAYEY SAND WITH GRAVEL (SC) medium dense, moist, brown, fine to coarse sand, fine to coarse gravel (sub-angular)	SC	40	⧗	9	116	
				17	⧗	15	94	
		CLAYEY SAND (SC) medium dense, moist, brown, fine sand, trace medium sand, fine gravel (sub-angular)						
	5			18	⧗	16	93	
			SC					
				17	⧗	14	99	
	10	trace medium to coarse sand, fine gravel (sub-angular)						
		SANDY SILT (ML) stiff, moist, brown, low plasticity, fine sand	ML					
		SILTY SAND (SM) medium dense, moist, brown, fine sand		18	⧗	9	107	20
	15		SM					
		LEAN CLAY (CL) very stiff, moist, brown, low to moderate plasticity, trace fine to medium sand						
			CL	52	⧗	18	101	
	20							
		CLAYEY SAND (SC) medium dense, moist, brown, fine to coarse sand, fine gravel (sub-angular)	SC					
				42	⧗	15	108	24
	25	**SANDY LEAN CLAY (CL)** very stiff, moist, brown, low plasticity, fine sand						
			CL					
				22	⧗	23	103	
	30		CL					



Continued Next Page

GROUND WATER OBSERVATIONS:
$\underline{\nabla}$: FREE GROUND WATER MEASURED DURING DRILLING AT 31.5 FEET

EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-53	PROJECT NO: 189340
BORING TYPE: 8-INCH HOLLOW	PROJECT: Evelyn Ave Apartments
LOGGED BY: AC	LOCATION: Sunnyvale, CA
START DATE: 12-16-11 FINISH DATE: 12-16-11	COMPLETION DEPTH: 45.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression; scale 1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
∇	30	▨	**LEAN CLAY WITH SAND (CL)** very stiff, moist, greenish-gray to brown, low plasticity, fine sand	CL						
		▨	**POORLY GRADED GRAVEL WITH SAND (GP-GC)** medium dense, wet, brown, fine to coarse sand, fine gravel (sub-angular)	GP-GC	14	⊠	26			○
	35	▨	**LEAN CLAY (CL)** stiff, moist, blue to greenish gray, low to moderate plasticity, trace fine sand	CL						
		⁘	**POORLY GRADED SAND (SP)** very dense, wet, brown, fine to coarse sand, fine gravel (sub-angular)	SP	50	⊠				
	40	⁘	**POORLY GRADED SAND WITH CLAY (SP-SC)** very dense, wet, brown, fine to medium sand, trace coarse sand	SP						
		▨	**SANDY LEAN CLAY (CL)** stiff, moist, brown to greenish-gray, low plasticity, fine sand	CL	32	⧓	16			○
	45	▨	**CLAYEY SAND WITH GRAVEL (SC)** dense, wet, brown, fine to coarse sand, fine gravel (sub-angular) Bottom of boring at 45 feet	SC						
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:

∇ : FREE GROUND WATER MEASURED DURING DRILLING AT 31.5 FEET

LA_CORP.GDT 1/6/12 MV* JPT

EXPLORATORY BORING: EB-2

Sheet 1 of 2

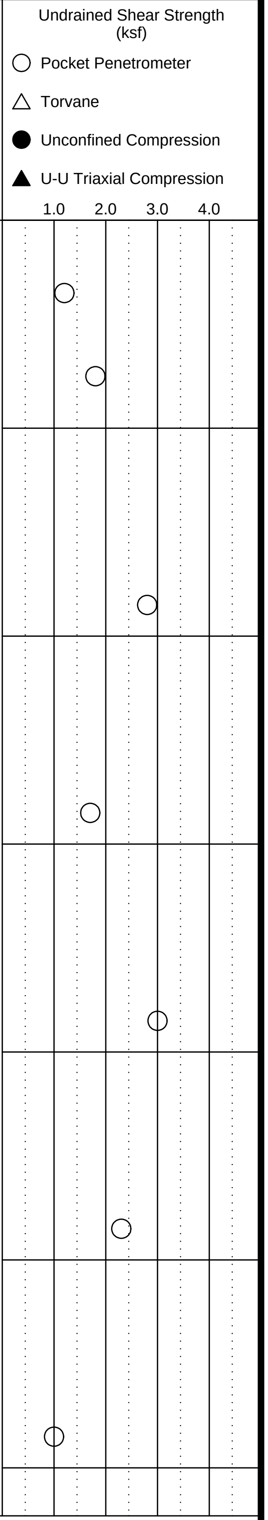
DRILL RIG: MOBILE B-53
BORING TYPE: 8-INCH HOLLOW
LOGGED BY: AC
START DATE: 12-16-11 FINISH DATE: 12-16-11

PROJECT NO: 189340
PROJECT: Evelyn Ave Apartments
LOCATION: Sunnyvale, CA
COMPLETION DEPTH: 45.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	**1.5" of AC over 2.5" of AB**	AC				
		SANDY LEAN CLAY (CL) stiff, moist, brown, low plasticity, fine sand Plasticity Index = 10, Liquid Limit = 27	CL	16	14	91	
		very stiff		13	13	96	
	5	**CLAYEY SAND (SC)** medium dense, moist, brown, fine sand, trace medium sand fine to coarse sand, fine gravel (sub-angular)	SC	13	8	103	
	10	**SANDY LEAN CLAY (CL)** very stiff, moist, brown, low plasticity, fine sand	CL	15	13	91	
		CLAYEY SAND (SC) medium dense, moist, brown, fine sand, trace medium sand	SC				
		SANDY LEAN CLAY (CL) stiff, moist, brown, low plasticity, fine sand		26	11	110	34
	15	**CLAYEY SAND (SC)** medium dense, moist, brown, fine to coarse sand	SC				
	20	**LEAN CLAY WITH SAND (CL)** very stiff, moist, brown, low plasticity, fine sand	CL	21	19	110	
	25	**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity, trace fine sand	CL	35	23	101	
		SANDY SILT (ML) very stiff, moist, light brown, low plasticity, fine sand	ML				
	30	**LEAN CLAY WITH SAND (CL)**	CL	30	19	109	35

Continued Next Page



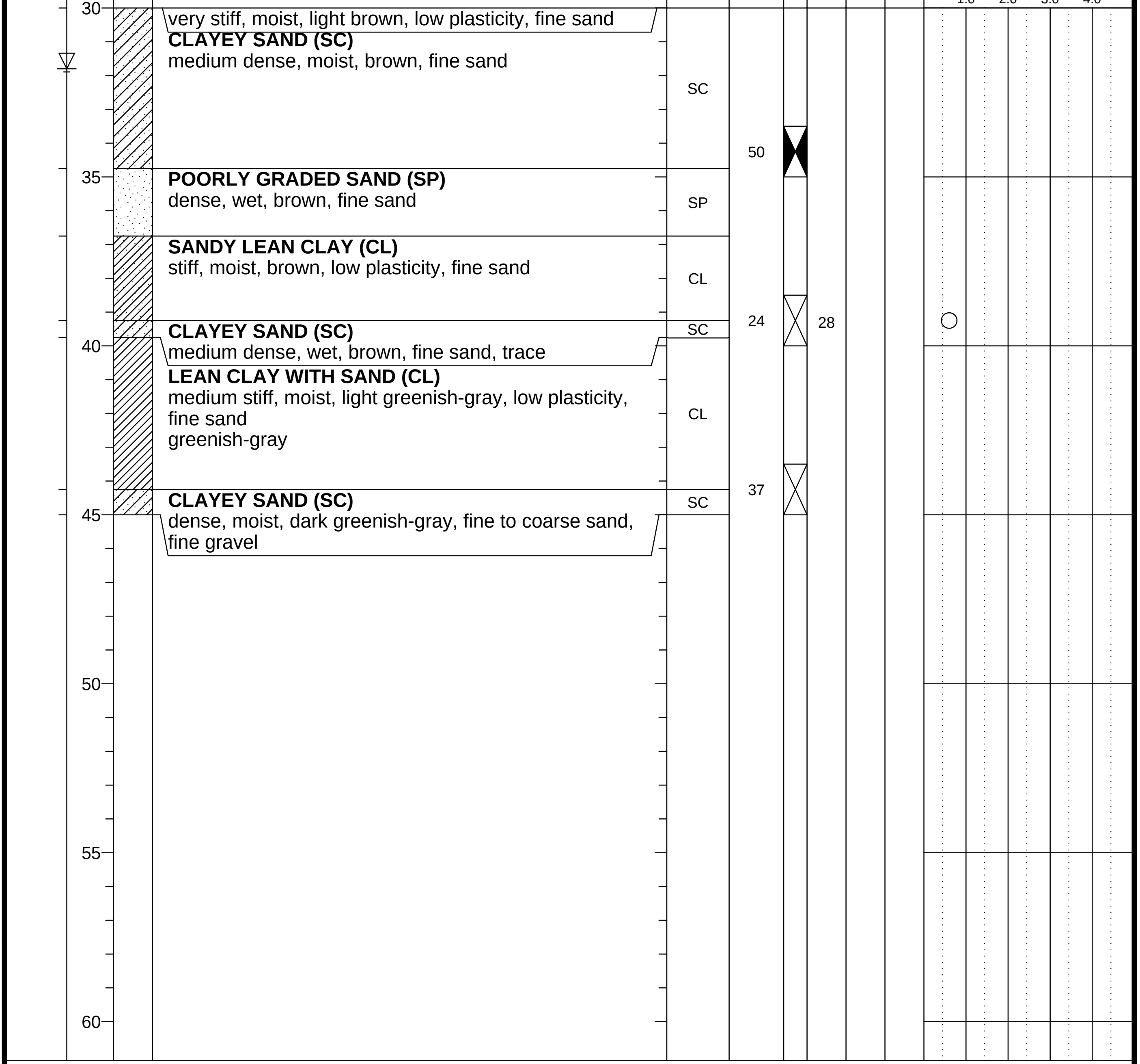
GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 31.8 FEET

EXPLORATORY BORING: EB-2 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-53	PROJECT NO: 189340
BORING TYPE: 8-INCH HOLLOW	PROJECT: Evelyn Ave Apartments
LOGGED BY: AC	LOCATION: Sunnyvale, CA
START DATE: 12-16-11 FINISH DATE: 12-16-11	COMPLETION DEPTH: 45.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.



ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	30		very stiff, moist, light brown, low plasticity, fine sand							
			CLAYEY SAND (SC) medium dense, moist, brown, fine sand	SC	50					
	35		**POORLY GRADED SAND (SP)** dense, wet, brown, fine sand	SP						
			SANDY LEAN CLAY (CL) stiff, moist, brown, low plasticity, fine sand	CL						
			CLAYEY SAND (SC) medium dense, wet, brown, fine sand, trace	SC	24		28			○
	40		**LEAN CLAY WITH SAND (CL)** medium stiff, moist, light greenish-gray, low plasticity, fine sand greenish-gray	CL						
			CLAYEY SAND (SC) dense, moist, dark greenish-gray, fine to coarse sand, fine gravel	SC	37					
	45									

Undrained Shear Strength (ksf):
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression

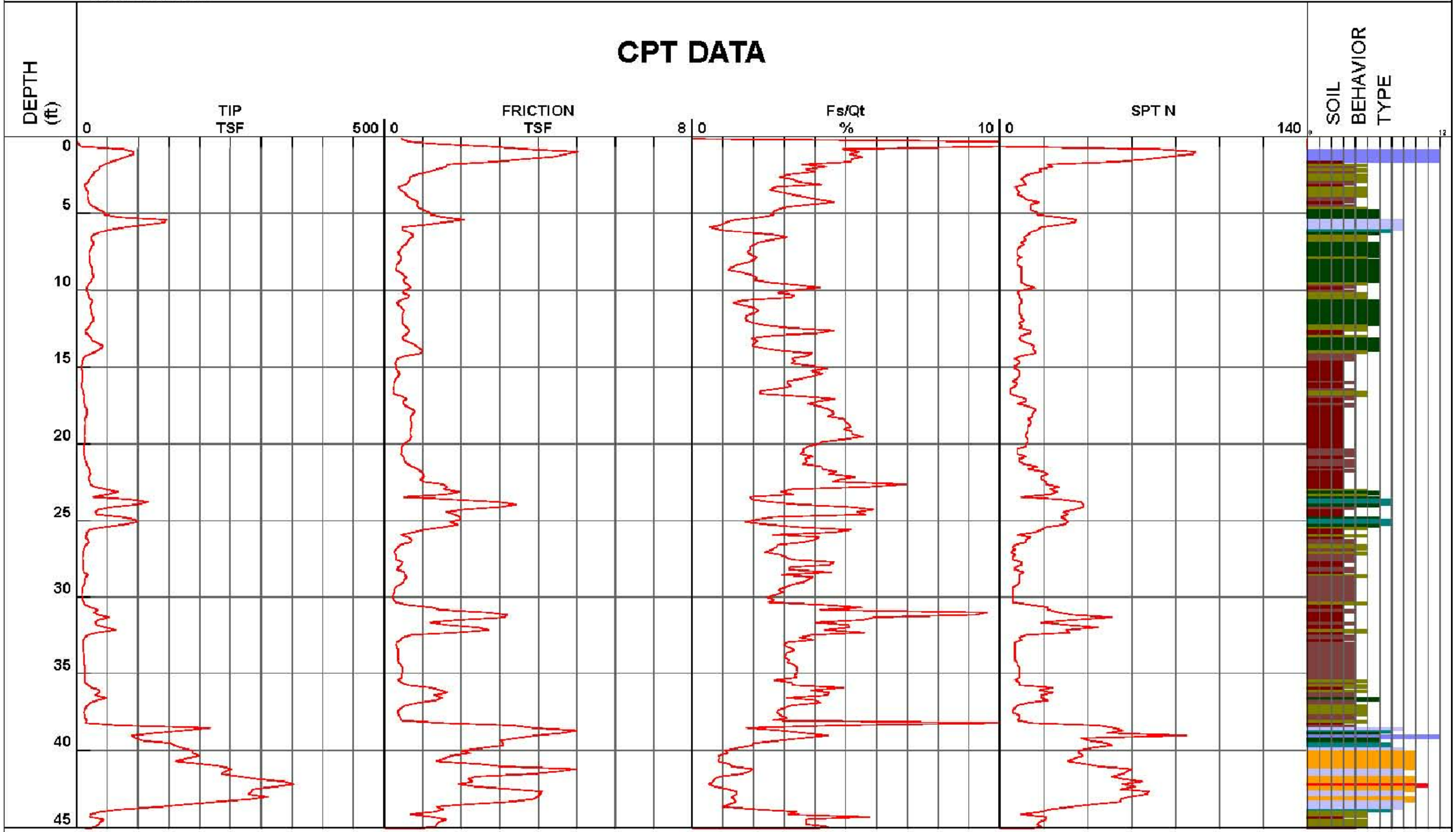
GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 31.8 FEET

LA_CORP.GDT 1/6/12 MV* JPT

TRC

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay

■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand

■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

*Soil behavior type and SPT based on data from UBC-1983

CONE PENETRATION TEST

Evelyn Avenue Apartments
457 and 477 East Evelyn Avenue
Sunnyvale, California

TRC

189340

CPT-1

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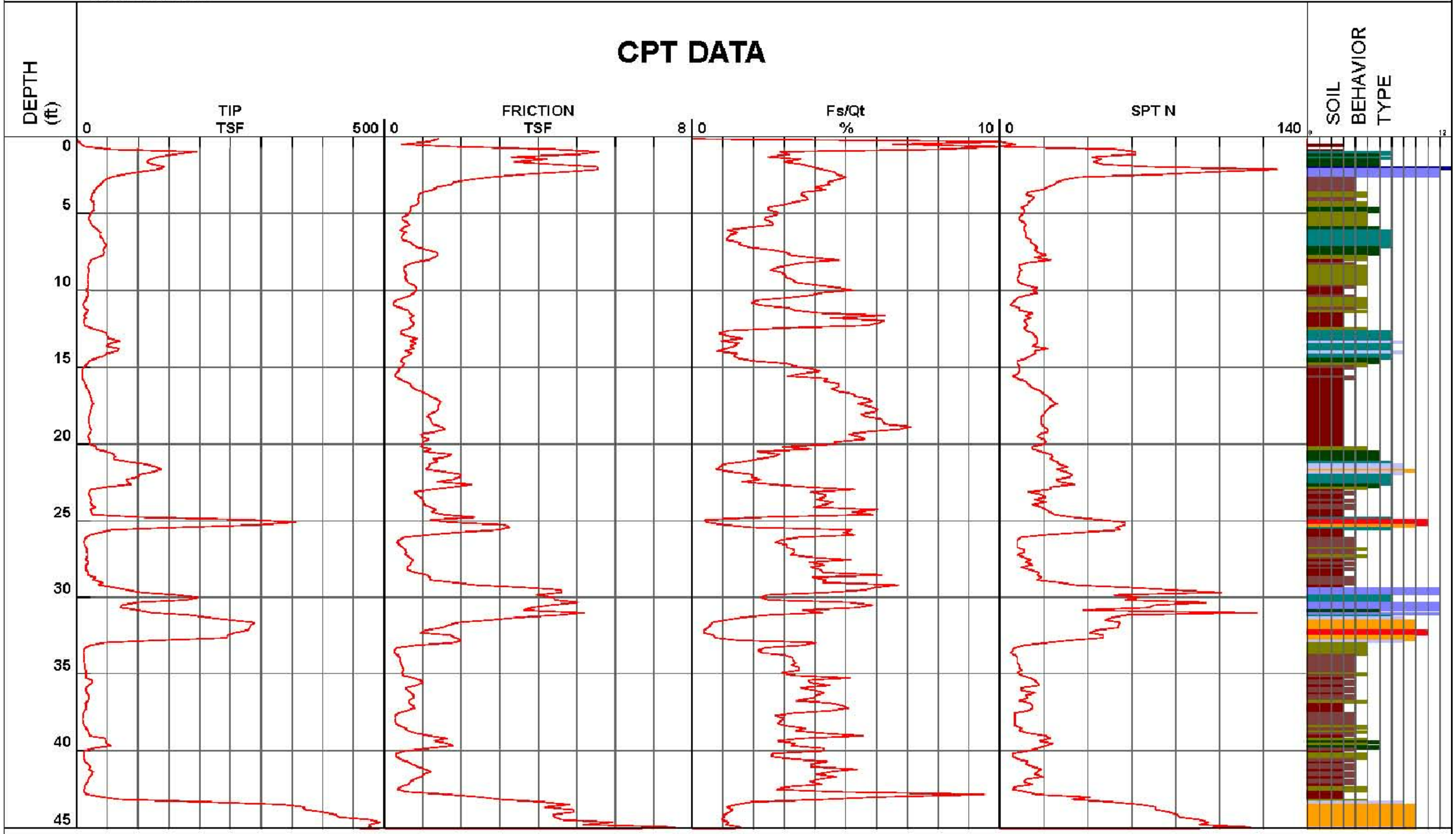
Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

*Soil behavior type and SPT based on data from UBC-1983

CONE PENETRATION TEST

Evelyn Avenue Apartments
457 and 477 East Evelyn Avenue
Sunnyvale, California

TRC

189340

CPT-2

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APPENDIX B

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was measured (ASTM D2216) on 19 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density tests (ASTM D2937) were performed on 16 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soils to measure the range of water contents over which this materials exhibit plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are presented on the Plasticity Chart of this appendix and on the log of the boring at the appropriate sample depth.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was performed on four samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

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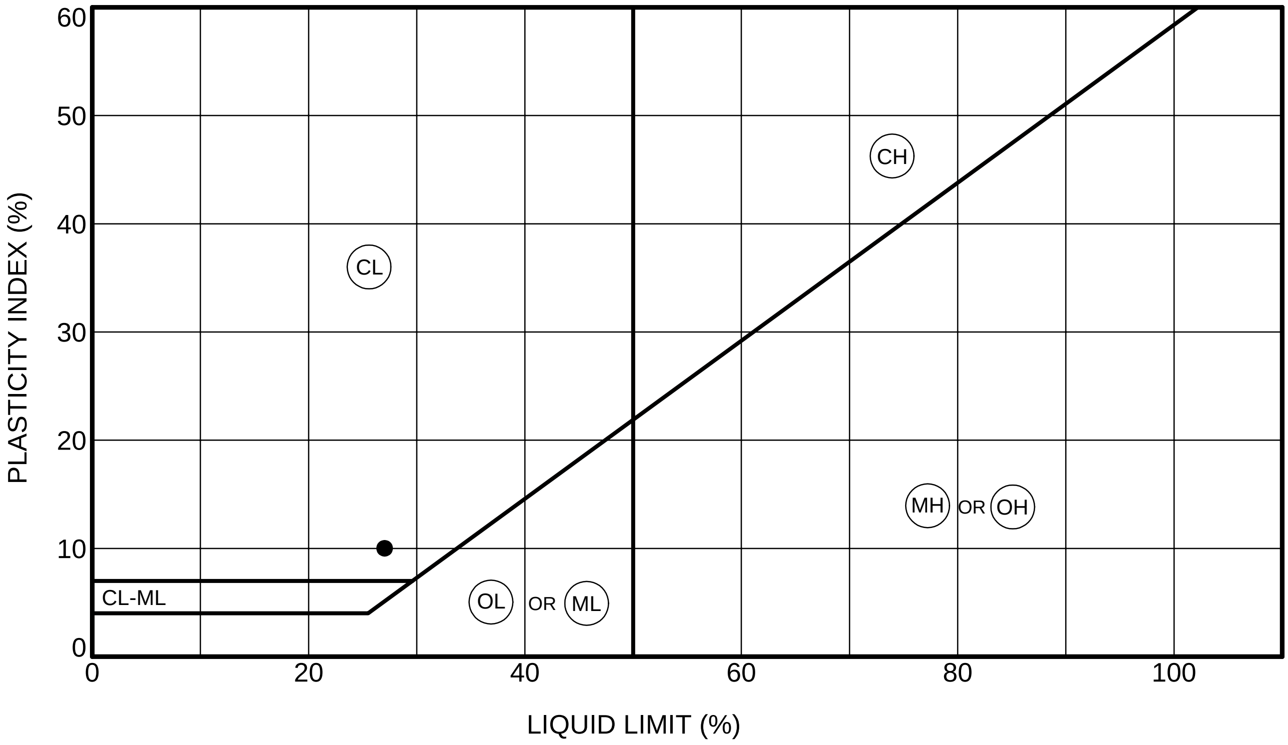

Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	EB-2	2.0		27	17	10		

PLASTICITY CHART AND DATA

Project: Evelyn Ave Apartments

Location: Sunnyvale, CA

Project No.: 189340

TRC