APPENDIX D: SOIL REPORT

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GEOTECHNICAL EVALUATION
OAKMONT OF EVERGREEN
3550 SAN FELIPE ROAD
SAN JOSE, CALIFORNIA

PREPARED FOR:

Oakmont Senior Living
9240 Old Redwood Highway, Suite 200
Windsor, California 95492

PREPARED BY:

Ninyo & Moore
Geotechnical and Environmental Sciences Consultants
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San Jose, California 95131

May 13, 2016
Project No. 402679001

May 13, 2016
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Ms. Hannah Daugherty
Project Manager
Oakmont Senior Living
9240 Old Redwood Highway, Suite 200
Windsor, California 95492

Subject: Geotechnical Evaluation
Oakmont of Evergreen
3550 San Felipe Road
San Jose, California

Dear Ms. Daugherty:

In accordance with your request, we have prepared this geotechnical evaluation for the proposed Oakmont of Evergreen Project to be constructed at 3550 San Felipe Road in San Jose, California. This report presents our geotechnical findings, conclusions, and recommendations regarding the proposed project.

As an integral part of our role as the geotechnical engineer-of-record, we request the opportunity to review the construction plans before they go to bid and to provide follow-up construction observation and testing services.

We appreciate the opportunity to be of service on this project.

Sincerely,
NINYO & MOORE

REGISTERED PROFESSIONAL ENGINEER
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1. INTRODUCTION

In accordance with your request, we have prepared this geotechnical evaluation for the proposed Oakmont of Evergreen Project to be constructed at 3550 San Felipe Road in San Jose, California (Figure 1). The purpose of our study was to evaluate geotechnical conditions and provide recommendations for the design and construction of this project.

2. SCOPE OF SERVICES

Ninyo & Moore's scope of services for this project generally included review of pertinent geologic and geotechnical background data, performance of a geologic reconnaissance, subsurface evaluation, laboratory testing, engineering analysis with regard to the proposed construction, and preparation of this report. Specifically, we performed the following tasks:

- Review of background data listed in the References section of this report. The data reviewed included topographic maps, geologic data and maps, fault and seismic hazard maps, flood hazard maps, and a conceptual site plan for the project.
- Geologic reconnaissance to observe site conditions and surficial geologic conditions.
- Mark out of the proposed exploratory boring locations prior to contacting Underground Service Alert.
- Field percolation testing at two locations to evaluate percolation characteristics for design of stormwater management facilities.
- Subsurface exploration consisting of drilling, logging, and sampling seven small-diameter borings advanced to depths of up to about 35½ feet. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk and relatively undisturbed soil samples for laboratory tests. The borings and soundings were backfilled with cement grout and drill cuttings.
- Laboratory testing of selected soil samples was performed to evaluate the geotechnical properties of the subsurface materials including in-situ soil moisture content and density, grain size distribution, Atterberg limits, shear strength, expansion index, soil corrosivity, and R-value.
- Compilation of the field and laboratory data and engineering analysis.

- Preparation of this report presenting our findings and conclusions regarding the geotechnical conditions encountered at the project site, and our recommendations for the design and construction of the proposed project.

3. SITE DESCRIPTION

The site of the proposed Oakmont of Evergreen facility is in Santa Clara Valley on the northeast side of San Felipe Road between Fowler Road and Yerba Buena Avenue in San Jose, California (Figure 1). The site is mostly undeveloped covering approximately 4.3 acres with a ground elevation of about 242 feet above Mean Sea Level (BRCE, 2016). The site and vicinity generally slopes down to the northwest with an overall average gradient of approximately 1½ percent (Google Earth, 2015). The site is bounded to the north by Evergreen Valley United Methodist Church, to the east by residential properties, to the south by commercial properties and to the west by San Felipe Road.

4. PROJECT DESCRIPTION

We understand that the project will consist of a new two-story assisted living facility with 79 units and a footprint area of approximately 49,000 square feet (Figure 2). Other project improvements will consist of a detached garage, a covered trash enclosure, parking areas, fire access lanes, underground utilities, driveways, pedestrian flatwork, landscaping, and a stormwater management system. The site is currently occupied by several mobile homes and the historic Smith House. We understand that the mobile homes will be removed and the historic Smith House will be relocated approximately 120 feet to the west of its current location. We anticipate that the finish floor elevations will be within a foot or two of the existing grade.

5. FIELD EXPLORATION AND LABORATORY TESTING

Our field exploration included a geologic reconnaissance, percolation testing, and subsurface exploration of the project site. The subsurface exploration was conducted on April 14, 2016 and consisted of drilling, logging, and sampling seven small-diameter borings advanced to depths of up to about 35½ feet below the existing ground surface. The locations of the borings and percolation tests are presented on Figure 2.

A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk and relatively undisturbed soil samples from the borings. The samples were then transported to our geotechnical laboratory for testing. The borings and soundings were backfilled with cement grout and drill cuttings shortly after completion. Detailed logs of the borings are presented in Appendix A.

Laboratory testing of soil samples recovered from the borings included tests to evaluate in-place soil moisture content and density, grain size distribution, Atterberg limits, shear strength, expansion index, soil corrosivity, and R-value. The results of the in-place soil moisture and density are shown at the corresponding sample depths on the boring logs in Appendix A. The results of the other laboratory tests are presented in Appendix B.

Percolation testing was performed in Borings B-6 and B-7 at a depth of about 3 feet below ground surface using a double wall infiltrometer. A description of the infiltration test procedure and the results of the testing are presented in Appendix C.

6. GEOLOGY AND SUBSURFACE CONDITIONS

Our findings regarding regional and site geology, and subsurface soil and groundwater conditions at the subject site are provided in the following sections.

6.1. Regional Geologic Setting

The site is located south of San Francisco Bay in the Coast Ranges geomorphic province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill structural valleys and line continental margins. The San Francisco Bay Area has several mountain ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras (Figure 3). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement.

6.2. Site Geology

Published geologic maps indicate that the site is underlain by Holocene surficial deposits consisting of alluvial gravel, sand and clay soil of valley areas including low-sloping alluvial fan gravel and sand near foothills (Dibblee, 2005). Wentworth, et al. (1999) indicates that the site is underlain by Upper Pleistocene alluvial fan deposits. Wentworth describes this unit as tan to reddish brown gravel with cobble-size clasts in a clayey and sandy matrix. The unit is crudely bedded with spatial relation to depositing streams typically still evident. Helley, et al. (1994) indicates that site is underlain by Pleistocene alluvial terrace deposits described as crudely-bedded, clast-supported, gravels, cobbles, and boulders with a sandy matrix. Clasts with an intermediate diameter of up to approximately 35 cm are typical. Coarse sand lenses may also be present. A map of regional geology is presented as Figure 4.

6.3. Subsurface Conditions

The following sections provide a generalized description of the geologic units encountered during our subsurface evaluation at the project site. More detailed descriptions are presented on the boring logs in Appendix A.

6.3.1. Asphalt Pavement

Boring B-1 was drilled through an asphalt pavement. The pavement section encountered was approximately 2 inches thick. Variations in the thickness of the pavement may be encountered due to past maintenance, utility work, or other factors.

6.3.2. Alluvium

Alluvium was encountered in the borings from the ground surface or below the asphalt pavement to the depths explored. The alluvium generally consisted of dry to moist, stiff to hard, lean clay; dry to moist, medium dense to very dense, sand and clayey sand; and dry, very dense gravel.

6.4. Groundwater

Groundwater was not encountered in our exploratory borings. The seismic hazard zone report for the San Jose East 7.5-minute Quadrangle (CDMG, 2000) indicates that the depth to historic high groundwater is around 20 feet below the ground surface. Fluctuations in the groundwater level may occur due to seasonal precipitation, variations in topography or subsurface hydrogeologic conditions, or as a result of changes to nearby irrigation practices or groundwater pumping. In addition, seeps may be encountered at elevations above the historic high groundwater levels due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration. Piezometers can be installed to further evaluate the depth to groundwater in the study area and fluctuation in groundwater levels.

7. GEOLOGIC HAZARDS AND GEOTECHNICAL CONSIDERATIONS

This study considered a number of potential issues relevant to the proposed construction on the subject site, including seismic hazards, landsliding, expansive soil, settlement of compressible soil layers, potential of on-site soil to corrode ferrous metals and promote sulfate attack on concrete, and excavation characteristics. These issues are discussed in the following subsections.

7.1. Seismic Hazards

The project site is located within a seismically active region. The seismic hazards considered in this study include the potential for ground surface rupture and ground shaking resulting from seismic activity, seismically induced liquefaction, dynamic settlement, sand-boil-induced ground subsidence, and lateral spreading. These potential hazards are discussed in the following subsections.

7.1.1. Faulting and Ground Surface Rupture

There are numerous recognized faults in California. Selected characteristics, as evaluated by the Working Group on California Earthquake Predictions (WGCEP, 2008), for recognized and postulated faults (Caltrans, 2012a) near the site are presented in Table 1. The fault characteristics in the table are presented in order of decreasing PGA for the site based on a deterministic seismic hazard analysis utilizing the Chiou & Youngs (2008) and Campbell & Bozorgnia (2008) attenuation relationships.

Table 1 – Parameters for Nearby Faults

Fault Name (Section)	ID	Type	Max Moment Magnitude	Distance to Site (kilometers)
Silver Creek	148	Reverse	6.9	1.5
Hayward (Southern extension)	149	Strike-Slip	6.7	2.6
Calaveras (Central) 2011 CFM	151	Strike-Slip	6.9	7.1
San Andreas (Santa Cruz Mts)	158	Strike-Slip	8.0	24.0
Cascade fault	153	Reverse	6.7	10.8
San Andreas (Peninsula) 2011 CFM	134	Strike-Slip	8.0	29.0
Monte Vista-Shannon	154	Reverse	6.4	11.2
Calaveras (No) 2011 CFM	130	Strike-Slip	6.9	16.7
Hayward (South)	137	Strike-Slip	7.3	22.4
Sargent fault (northwestern section)	164	Strike-Slip	7.0	23.1

The site is not located within an Alquist-Priolo Fault-Rupture Hazard Zone established by the state geologist (CDMG, 1982) to delineate regions of potential ground surface rupture adjacent to active faults and the site is not within a fault rupture hazard zone established by the county geologist (SCC, 2012). As defined by the California Geological Survey (CGS), active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,000 years (CGS, 2007). The closest known active fault is the Evergreen fault. The fault rupture hazard zone associated with this fault is approximately 1,800 feet from the site to the northeast (SCC, 2012). The locations of significant regional faults near the site are noted on Figure 4.

Based on our review of the referenced geologic maps, there are no known faults at the project site and the site is not located within a fault rupture hazard zone. Therefore, the probability of damage from surface fault rupture is considered to be low.

7.1.2. Seismic Ground Motion

Based on historic activity, the potential for future strong ground motion at the site is considered significant. Design recommendations for structures to address seismic shaking are provided in Section 9.2. The 2013 CBC specifies that the potential for liquefaction and soil strength loss should be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with an adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated to be 0.56g using the United States Geological Survey (USGS, 2016) seismic design tool to evaluate the mapped MCE_G peak ground acceleration (0.56g) and corresponding site coefficient (F_{PGA}) of 1.00 for Site Class D.

7.1.3. Liquefaction and Strain Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soils of low plasticity (liquefaction) or in wet, sensitive, cohesive soils (strain softening). Liquefaction and strain softening can result in a loss of foundation bearing capacity or lateral spreading of sloping or unconfined ground. Liquefaction can also generate sand boils leading to subsidence at the ground surface. Liquefaction (or strain softening) is generally not a concern at depths more than 50 feet below ground surface.

The project site is not located within a liquefaction hazard zone (Figure 5) as mapped by the state geologist (CDMG, 2001) or by the county geologist (SCC, 2012). During our subsurface exploration, we encountered medium dense sand in Boring B-2 near historic high groundwater levels that could be susceptible to liquefaction. We evaluated the liquefaction susceptibility of these deposits in accordance with the method presented by Idriss & Boulanger (2008) using the penetration resistance data collected during our subsurface exploration and considering a PGA of 0.56g corresponding to a Magnitude 6.9 earthquake on the Silver Creek fault (Caltrans, 2012a). For the liquefaction analysis, we assumed a groundwater depth of 20 feet. The results of our analysis, presented in Appendix D, indicate that a sand layer in Boring B-2 extending from approximately 20 to 23 feet below the existing ground surface will liquefy under the considered ground motion based on a factor of safety against liquefaction of less than one. However due to the depth of the liquefiable layer encountered, we do not regard the potential for liquefaction-induced reduction in the bearing capacity of shallow foundations as a design consideration for the project. Other consequences of liquefaction, including dynamic settlement, sand-boil-induced ground subsidence, and lateral spreading, are addressed in the following sections.

The cohesive soils that we encountered during our subsurface exploration are not known to be particularly sensitive. We do not regard seismically-induced strain-softening behavior as a design consideration.

7.1.4. Dynamic Settlement

The strong vibratory motion associated with earthquakes can also dynamically compact loose granular soil leading to surficial settlements. Dynamic settlement is not limited to the near surface environment and may occur in both dry and saturated sand and silt. Cohesive soil is not typically susceptible to dynamic settlement.

During our subsurface exploration, we encountered deposits of medium to very dense sand that could dynamically compact following an earthquake. We evaluated the potential for dynamic settlement using the penetration resistance data collected during our subsurface exploration in accordance with the method presented by Tokimatsu and Seed (1987) for saturated sand and the method presented by Pradel (1998) for dry sand. Our analysis considered a Magnitude 6.9 earthquake producing a PGA of 0.56g with groundwater at 20 feet below grade. The results of our analyses indicate that the total dynamic settlement will be relatively minor, approximately ⅔ inch (Boring B-2) following the considered seismic event. Differential dynamic settlement is estimated to be on the order of about ⅓ inch over a horizontal distance of approximately 30 feet. Recommendations for shallow footing foundations are provided.

7.1.5. Ground Subsidence

Sand boils that occur when liquefied, near-surface soil escapes to the ground surface, can result in ground subsidence due to loss of material that is in addition to dynamic settlement. We do not consider sand boils to be a design consideration at the site due to the relative density of the liquefiable deposits encountered.

7.1.6. Lateral Spreading

In addition to vertical displacements, seismic ground shaking can induce horizontal displacements as surficial soil deposits spread laterally by floating atop liquefied subsurface layers. Lateral spread can occur on sloping ground or on flat ground adjacent to an exposed face. The topography of the project site is relatively flat and a free-face condition does not exist near the proposed improvements. In addition, the lateral extent of the liquefiable soil layers is relatively discontinuous. Consequently, we do not regard lateral spreading as a design consideration.

7.2. Landsliding and Slope Stability

The site and surrounding areas are relatively flat and are not located within a landslide hazard zone as mapped by the state geologist (CDMG, 2001) or the county geologist (SCC, 2012). As such, we do not regard landsliding as a design consideration. Recommendations for allowable gradients of temporary and constructed slopes are provided.

7.3. Expansive Soil

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soil containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. Laboratory testing was performed on a sample of the near-surface soil to evaluate the expansion index. The tests were performed in general accordance with the American Society of Testing and Materials (ASTM) Standard D 4829 (Expansion Index). The results of our laboratory test indicate that the expansion index of the sample tested was 32. These results are indicative of a low expansion characteristic. The recommendations for subgrade preparation and foundation embedment provided in Section 9 of this report are provided with consideration for the low expansion characteristics of the site soil.

7.4. Static Settlement

Based on our review of the preliminary grading plan (BRCE, 2016), we understand that a significant increase in the site grade is not proposed and anticipate that the structural loads for the building will be moderate. Based on the subsurface conditions encountered during our evaluation, we anticipate that static settlement due to sustained loads will be tolerable for foundations designed and constructed in accordance with the recommendations in this report.

7.5. Corrosive/Deleterious Soil

An evaluation of the corrosivity of the on-site materials was conducted to assess the impact to concrete and metals. The corrosion impact was evaluated using the results of limited laboratory testing on samples obtained during our subsurface study. Laboratory testing to quantify pH, resistivity, chloride, and soluble sulfate contents was performed on a sample of the fill. The results of the corrosivity tests are presented in Appendix B. California Department of Transportation (Caltrans) defines a corrosive environment as an area within 1,000 feet of brackish water or where the soil contains more than 500 parts per million (ppm) of chlorides, sulfates of 0.2 (2,000 ppm) percent or more, or pH of 5.5 or less

(Caltrans, 2012b). The criteria used to evaluate the deleterious nature of soil on concrete are listed in Table 2. Based on these criteria, the inland location of the site, and the results of our testing, the site does not meet the definition of a corrosive environment and the sulfate exposure to concrete is negligible. Exposed ferrous metals will undergo corrosion but conventional measures to mitigate corrosion, such as galvanization or reliance on a corrosion allowance, should be effective. A corrosion engineer may be consulted to provide specific guidance. Recommendations to mitigate the impact of corrosive/deleterious soil on concrete structures are presented in Section 9.6.

Table 2 – Criteria for Deleterious Soil on Concrete

Sulfate Content Percent by Weight	Sulfate Exposure
0.0 to 0.1	Negligible
0.1 to 0.2	Moderate
0.2 to 2.0	Severe
> 2.0	Very Severe
Reference: American Concrete Institute (ACI) Committee 318 Table 4.3.1 (ACI, 2016)	

7.6. Excavation Characteristics

We anticipate that construction of the project may involve excavations of up to about several feet in depth for foundation construction and utility installation. Our subsurface exploration generally encountered stiff to hard, lean clay and medium dense sand and clayey sand over this interval. We anticipate that heavy earthmoving equipment in good working condition should be able to make the proposed excavations.

Near-vertical cuts in these materials may not be stable, particularly if the excavation encounters granular soil, is exposed to water, or if the sidewall is disturbed during construction operations. Excavation subgrade may become unstable if exposed to wet conditions. Recommendations for excavation stabilization are presented.

8. CONCLUSIONS

Based on our review of the referenced background data, our geologic field reconnaissance, subsurface evaluation, and laboratory testing, it is our opinion that construction of the proposed assisted living facility is feasible from a geotechnical standpoint. Geotechnical considerations include the following:

- Our subsurface evaluation indicated that the project site is underlain by alluvium consisting of stiff to hard clay, and medium to very dense sand, clayey sand, and gravel.
- Groundwater was not encountered during our subsurface exploration. Regional records of historic groundwater levels indicate that the depth to historic high groundwater is around 20 feet. Fluctuations in the groundwater levels should be anticipated as discussed in Section 6.4.
- The earth materials underlying the site should be excavatable with conventional earth moving equipment in good working condition. Near-vertical excavations in granular materials should be considered unstable. Recommendations for excavation stabilization are presented.
- The site could experience a relatively large degree of ground shaking during a significant earthquake event on a nearby fault.
- Liquefaction and liquefaction-related seismic hazards (e.g., loss of foundation bearing capacity, seismic strain softening, sand boil induced ground subsidence, and lateral spreading) are not design considerations for the project.
- Dynamic settlement due to seismic ground shaking is anticipated to be minor. The total dynamic settlement due to the assumed ground motion is estimated to be approximately ⅔ inch with a differential dynamic settlement of about ⅓ inch over a lateral distance of approximately 30 feet.
- Recommendations for shallow footing foundations are provided to address the potential for settlement due to sustained building loads.
- Landsliding is not a design consideration. Recommendations for allowable gradients of temporary and constructed slopes are provided.
- Test results indicate that the onsite soil has a low expansion characteristic.
- The site does not meet the definition of a corrosive environment (Caltrans, 2012b).

9. RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements should be designed constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

9.1. Earthwork

The earthwork should be conducted in accordance with the relevant grading ordinances having jurisdiction and the following recommendations. The geotechnical engineer should observe earthwork operations. Evaluations performed by the geotechnical engineer during the course of field operations may result in new recommendations, which could supersede the recommendations in this section.

9.1.1. Pre-Construction Conference

We recommend that a pre-construction conference be held to discuss the grading recommendations presented in the report. The owner and/or their representative, the architect, the engineer, Ninyo & Moore, and the contractor should be in attendance to discuss project schedule and earthwork requirements.

9.1.2. Site Preparation

Site preparation should begin with the removal of vegetation, utility lines, debris and other deleterious materials from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. Rubble and excavated materials that do not meet criteria for use as fill should be disposed of in an appropriate landfill. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout.

Excavations resulting from removal of buried utilities, tree stumps, or obstructions should be backfilled with compacted fill in accordance with the recommendations in the following sections.

9.1.3. Observation and Removals

Prior to placement of fill, erection of forms or placement of reinforcement for foundations, the client should request an evaluation of the exposed subgrade by Ninyo & Moore. Materials that are considered unsuitable shall be excavated under the observation of the geotechnical engineer in accordance with the recommendations in this section or the field recommendations of the geotechnical engineer.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil; and undocumented or otherwise deleterious fill materials. Unsuitable materials should be removed from trench bottoms and below bearing surfaces to a depth at which suitable foundation subgrade, as evaluated in the field by the geotechnical engineer, is exposed.

9.1.4. Material Recommendations

Materials used during earthwork, grading, and paving operations should comply with the requirements listed in Table 3. Materials should be evaluated by the geotechnical engineer for suitability prior to use. The contractor should notify the geotechnical consultant 72 hours prior to import of materials or use of on-site materials to permit time for sampling, testing, and evaluation of the proposed materials. On-site materials may need to be dried out before re-use as fill. The contractor should be responsible for the consistency of import material brought to the site.

Table 3 – Recommended Material Requirements

Material and Use	Source	Requirements[1,2]
General Fill - below/adjacent to structures - for uses not otherwise specified	Import	Close-graded with 35 percent or more passing No. 4 sieve and either: Expansion Index of 50 or less, Plasticity Index of 12 or less, or less than 10 percent, by dry weight, passing No. 200 sieve
	On-site borrow	No additional requirements[1]
Aggregate Base for pavements	Import	Class II; CSS[4] Section 26-1.02
Asphalt Concrete for pavements	Import	Type A; CSS[4] Section 39-2
Permeable Aggregate - capillary break gravel	Import	Open-graded, clean, compactable crushed rock or angular gravel; nominal size ¾ inch or less
Pipe/Conduit Bedding and Pipe Zone Material -material below conduit invert to 12 inches above conduit	Import	90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve
Trench Backfill - above bedding material	Import or on-site borrow	As per general fill and excluding rock/lumps retained on 4-inch sieve or 2-inch sieve in top 12 inches

1 In general, fill should be free of rocks or lumps in excess of 6-inches diameter, trash, debris, roots, vegetation or other deleterious material.

2 In general, import fill should be tested or documented to be non-corrosive[3] and free from hazardous materials in concentrations above levels of concern.

3 Non-corrosive as defined by the Corrosion Guidelines (Caltrans, 2012b).

4 CSS is California Standard Specifications (Caltrans, 2015b).

9.1.5. Subgrade Preparation

Subgrade in trenches and below slabs, footings, pavements, or fill, should be prepared as per the recommendations in Table 4. Prepared subgrade should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill or construction of footings and slabs. Subgrade that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture conditioned, and recompacted as per the requirements above. A thin layer (approximately 3 inches) of lean concrete or controlled low strength material (CLSM) may be poured over prepared subgrade for footings or slabs to maintain the appropriate moisture condition during erections of forms and placement of reinforcing steel.

Table 4 – Subgrade Preparation Recommendations

Subgrade Location	Preparation Recommendations
Utility Trenches	• Check for unsuitable materials as per Section 9.1.3. • Remove or compact loose/soft material.
Below Pavements	• Clear & grub to remove unsuitable materials. • Scarify 8 inches then moisture condition and compact as per Section 9.1.6. • Proof roll compacted subgrade with loaded water truck under the observation of the geotechnical engineer. Mitigate yielding areas in accordance with the recommendations of the engineer. • Keep in moist condition by sprinkling water.
Below Fill, Slabs, & Flatwork	• Check for unsuitable materials as per Section 9.1.3 • Scarify 8 inches then moisture condition and compact as per Section 9.1.6. • Keep in moist condition by sprinkling water.
Below Footings	• Check for unsuitable materials as per Section 9.1.3. • Scarify and moisture condition exposed subgrade as-needed to achieve a moisture content approximately 2 points above the optimum as evaluated by ASTM D1557. Compact exposed subgrade to 90 percent of the reference density as evaluated by ASTM D1557. • Keep in moist condition by sprinkling water. • (Optional) Place layer of lean concrete or CLSM over prepared subgrade.

9.1.6. Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 5. The allowable uncompacted thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness.

Table 5 – Recommended Compaction Requirements

Fill Type	Location	Compacted Density[1]	Moisture Content[2]
Subgrade	Below pavements	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Bedding and Pipe Zone Fill	Below invert to 12 inches above pipe	90 percent	+ 2 percent
Trench Backfill	Below pavements	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
General Fill	Within 2 feet of finish grade for pavements	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Aggregate Base	Under pavement and flatwork	95 percent	+ 2 percent
Asphalt Concrete	Pavement Section	91 to 97 percent	Not Applicable

[1] Expressed as percent relative compaction or ratio of field density to reference density (typically on a dry density basis for soil and aggregate and on a wet density basis for asphalt concrete). The reference density of soil and aggregate should be evaluated by ASTM D 1557. The reference density of asphalt concrete should be evaluated by ASTM D 2041.

[2] Target moisture content at compaction relative to the optimum as evaluated by ASTM D 1557.

Compacted fill should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill or construction of footings and slabs. Fill that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture conditioned, and recompacted as per the requirements above.

9.1.7. Excavation Stabilization and Shoring

Excavations, including foundation and utility excavations, should be stabilized by shoring sidewalls or laying slopes back in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations [CFR], Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA).

Table 6 lists the OSHA material type classifications and corresponding allowable temporary slope layback inclinations for soil deposits that may be encountered on site. Alternatively, cantilever or internally-braced shoring systems may be used to stabilize excavation sidewalls during construction. The lateral earth pressures listed in Table 6 may be used to design or select an internally-braced shoring system or trench shield conforming to the OSHA guidelines. Our recommendations for lateral earth pressures and allowable slope gradients are based upon the limited subsurface data provided by our exploratory borings and reflect the influence of the environmental conditions that existed at the time of our exploration. Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse. Dewatering pits or sumps should be used to depress the groundwater level (if encountered) below the bottom of the excavation.

Table 6 – OSHA Material Classifications and Allowable Slopes

Formation	OSHA Classification	Allowable Temporary Slope[1,2,3]	Lateral Earth Pressure on Shoring[4] (psf)
Alluvium (above groundwater)	Type C	1 ½ h:1v (34°)	80·D + 72

[1] Allowable slope for excavations less than 20 feet deep. Excavation sidewalls in cohesive soil may be benched to meet the allowable slope criteria (measured from the bottom edge of the excavation). The allowable bench height is 4 feet. The bench at the bottom of the excavation may protrude above the allowable slope criteria.

[2] In layered soil, layers shall not be sloped steeper than the layer below.

[3] Temporary excavations less than 5 feet deep may be made with vertical side slopes and remain unshored if judged to be stable by a competent person (29 CFR, Part 1926.650).

[4] 'D' is depth of excavation for excavations up to 20 feet deep. Includes a surface surcharge equivalent to two feet of soil.

The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor. The shoring parameters presented in this report are preliminary design criteria, and the designer should evaluate the adequacy of these parameters and make appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

Excavations made in close proximity to existing structures may undermine the foundation of those structures and/or cause soil movement related distress to the existing structures. Stabilization techniques for excavations in close proximity to existing structures will need to account for the additional loads imposed on the shoring system and appropriate setback distances for temporary slopes. The geotechnical engineer should be consulted for additional recommendations if the proposed excavations cross below a plane extending down and away from the foundation bearing surfaces of the adjacent structure at an angle of 2:1 (horizontal to vertical).

The excavation bottoms may encounter wet, loose material, which may be subject to pumping under heavy equipment loads. The contractor should be prepared to stabilize the bottom of the excavations. In general, unstable bottom conditions may be mitigated by using a stabilizing geogrid, overexcavating the excavation bottom to suitable depths and replacing with compacted fill, or other suitable method. Additionally, aeration of wet soils should be anticipated.

9.1.8. Construction Dewatering

Water intrusion into the excavations may occur as a result of groundwater seepage or surface runoff. The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations. Sump pits, trenches, or similar measures should be used to depress the water level below the bottom of the excavation. Considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement, and groundwater discharge. Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board.

9.1.9. Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with our recommendations in Section 9.1.7. Utility trenches should be backfilled with materials that conform to our recommendations in Section 9.1.4. Trench backfill, bedding, and pipe zone fill should be compacted in accordance with Section 9.1.6 of this report. Bedding and pipe zone fill should be shoveled under pipe haunches and compacted by manual or mechanical, hand-held tampers. Trench backfill should be compacted by mechanical means. Densification of trench backfill by flooding or jetting should not be permitted.

To reduce potential for moisture intrusion into the building envelope, we recommend plugging utility trenches at locations where the trench excavations cross under the building perimeter. The trench plug should be constructed of a compacted, fine-grained, cohesive soil that fills the cross-sectional area of the trench for a distance equivalent to the depth of the excavation. Alternatively, the plug may be constructed of concrete or CLSM.

9.1.10. Rainy Weather Considerations

We recommend that the construction be performed during the period between approximately April 15 and October 15 to avoid the rainy season. In the event that grading is performed during the rainy season, the plans for the project should be supplemented to include a stormwater management plan prepared in accordance with the requirements of the relevant agency having jurisdiction. The plan should include details of measures to protect the subject property and adjoining off-site properties from damage by erosion, flooding or the deposition of mud, debris, or construction-related pollutants, which may originate from the site or result from the grading operation. The protective measures should be installed by the commencement of grading, or prior to the start of the rainy season. The protective measures should be maintained in good working order unless the project drainage system is installed by that date and approval has been granted by the building official to remove the temporary devices.

In addition, construction activities performed during rainy weather may impact the stability of excavation subgrade and exposed ground. Temporary swales should be constructed to divert surface runoff away from excavations and slopes. Steep temporary slopes should be covered with plastic sheeting during significant rains. The geotechnical consultant should be consulted for recommendations to stabilize the site as-needed. A thin layer (approximately 3 inches) of lean concrete or CLSM may be poured over prepared subgrade for footings or slabs to maintain the appropriate moisture condition during erections of forms and placement of reinforcing steel.

9.2. Seismic Design Considerations

Seismic design of the proposed improvements should be performed in accordance with the requirements of the governing jurisdictions and applicable building codes. Table 7 presents the MCE_R spectral ground motion response accelerations consistent with the 2013 CBC and corresponding site-adjusted and design level spectral response accelerations for structures with a fundamental period of ½ second or less.

Table 7 – 2013 California Building Code Seismic Design Criteria

Seismic Design Parameters Evaluated for 37.31078° North Latitude, 121.78527° West Longitude	**Value**
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Mapped Spectral Acceleration at 0.2-second Period, S_s	1.502 g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.600 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.502 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.900 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.002 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.600 g
Seismic Design Category for Risk Category I, II, or III	D

9.3. Foundations

The new assisted living facility may be supported on spread footings with a slab-on-grade floor. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in design of the structures.

9.3.1. Spread Footings

Footing bearing on subgrade prepared in accordance with the recommendations in Section 9.1.5 may be designed using the criteria listed in Table 8. The geotechnical engineer should observe the footing excavations to evaluate bearing materials and subgrade condition before the exposed subgrade is covered.

Table 8 – Recommended Bearing Design Parameters for Footings

Footing	Sustained Loads	Footing Width	Bearing Depth[1]	Allowable Bearing Capacity[2]	Static Settlement
Wall Footing	12.5 kips/foot or less	12 inches or more	2 feet	2,500 psf	1 inch total ½ inch differential over 30 feet
Column Footing	100 kips or less	2 feet or more	2 feet	3,000 psf	1 inch total ½ inch differential over 30 feet

[1] Below the adjacent grade.
[2] Net allowable bearing capacity in pounds per square foot. Listed value includes a Factor of Safety of 3 or more. Allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic loads.

Structures supported on footings consistent with these recommendations should be designed for the total and differential settlements listed in Table 8 for sustained loads plus an additional ⅔ inch with a differential dynamic settlement of about ⅓ inch over a lateral span of 30 feet. Footing settlement due to static loads may be further evaluated using a modulus of subgrade reaction. Recommended values for the modulus of subgrade reaction are provided in Table 9. The designer may interpolate between the values in the table for intermediate footing widths.

Table 9 – Footing Modulus of Subgrade Reaction

Footing	Footing Width				
	1 foot	2 feet	3 feet	5 feet	10 feet
Wall Footing	49 pci	29 pci	23 pci	18 pci	--
Column Footing	--	45 pci	32 pci	23 pci	16 pci
Notes: Modulus of Subgrade Reaction in pounds per cubic inch (pci).					

The spread footings should be reinforced with deformed steel bars as detailed by the project structural engineer. Where footings are located adjacent to utility trenches or other excavations, the footing bearing surfaces should bear below an imaginary plane extending upward from the bottom edge of the adjacent trench/excavation at a 2:1 (horizontal to vertical) angle. Footings should be deepened or excavation depths reduced as-needed.

A lateral bearing pressure of 300 psf per foot of depth up to 3,000 psf may be used to evaluate the resistance of footings to lateral loads. The recommended lateral bearing pressure is for level ground conditions where the ground adjacent to the foundation is approximately level for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. The lateral bearing pressure should be neglected to a depth of 1 foot where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces. A friction coefficient of 0.35 may be assumed for evaluating frictional resistance to lateral loads.

9.3.2. Slabs-on-Grade

Building floor slabs should be designed by the project structural engineer based on the anticipated loading conditions. Recommendations for slabs subject to vehicular traffic are provided in Section 9.5.2. The slab should be reinforced with deformed steel bars. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of slab reinforcement in the upper half of the slab. Refer to Section 9.5 for the recommended concrete cover over reinforcing steel. A vapor retarder is recommended in areas where moisture-sensitive floor coverings or conditioned environments are anticipated. See Section 9.7 for vapor retarding system recommendations. Joints consistent with ACI guidelines (ACI, 2016) should be constructed at periodic intervals to reduce the potential for random cracking of the slab.

9.4. Retaining Walls

Minor retaining walls (wall height above footing of 5 feet or less) may be designed for active, at-rest, and passive equivalent fluid earth pressures of 40, 60, and 300 psf per foot depth for level backfill conditions and an active equivalent fluid earth pressures of 65 psf per foot depth for 2:1 (horizontal to vertical) sloping backfill conditions (interpolation can be used for intermediate backfill slope angles). Lateral forces may be resisted by friction at the base of the wall footing and passive earth pressure acting on the embedded wall, wall footing, or wall key, if present. Passive earth pressure should be neglected to a depth of 1 foot below the ground surface when evaluating lateral load resistance where the ground surface is not covered by pavement or flatwork. Gravity and semi-gravity cantilever walls may be designed for a coefficient of friction of 0.35 to resist lateral loads and an allowable bearing capacity of 2,500 psf for a 12-inch footing width and 12 inches of embedment below the adjacent grade plus 100 psf per additional foot of width and 400 psf per additional foot of embedment up to 4,000 psf.

Walls should be designed to withstand a total static settlement of 1 inch with a differential of ½ inch over a 20-foot span. We recommend that the wall and the wall footing be reinforced. Footings should be designed by the structural engineer based on the anticipated loading and usage. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of footing reinforcement. Refer to Section 9.6 for the recommended concrete cover over reinforcing steel.

Cantilever semi-gravity walls that yield or deflect may be designed for active earth pressures. Wall deflection equivalent to about 1 percent of wall height may be needed to reduce at-rest earth pressures to active earth pressures.

Walls retaining level ground should be designed to resist construction or live load surcharges on the backfill. The lateral earth pressure due to a backfill surcharge should be a uniform horizontal surcharge of 72 psf. An additional backfill surface and lateral earth pressure for adjacent footings should be considered, as applicable, where the adjacent footings bear above an imaginary plane that rises up and away from the bottom edge of the wall at a 2:1 (horizontal to vertical) gradient.

Hydrostatic pressures may be neglected, provided that suitable drainage of the retained soil is provided. The retained soil should be drained by weep holes or a subdrain at the base of the wall stem consisting of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be capped by a pavement or 12 inches of native soil and drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar). The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the wall. Alternatively, geocomposite drain panels (Miradrain 6000XL, or similar) placed against the back of the wall may be used to supplement a smaller subdrain located near the base of the wall. Measures to reduce the rate of moisture or vapor intrusion through the wall may be advisable for walls where the discoloration resulting from moisture intrusion would be undesirable. Such measures might include use of concrete with a low water-to-cementitious-materials ratio, and/or the placement of an asphalt emulsion or 10-mil thick plastic membrane to the back surface of the wall.

9.5. Pavements

Recommendations for rigid and flexible pavements are presented in the following sections. The design R-value used for evaluate the pavement sections was selected based on R-value testing performed on a sample collected during our subsurface exploration. The pavement subgrade should be observed by the geotechnical engineer during grading to check that the exposed materials are consistent with the findings from our subsurface exploration and the support characteristics assumed for pavement design. Additional R-value testing may be needed, based on these observations, with subsequent revision to the pavement sections.

Projected traffic and anticipated vehicle loading data were not available at the time of our pavement evaluation and we did not evaluate a traffic index for the project. Pavement sections were evaluated for a range of traffic indexes. The designer may interpolate between the values provided once a traffic index has been selected.

9.5.1. Asphalt Concrete Pavement

Ninyo & Moore conducted an analysis to evaluate appropriate asphalt pavement structural sections following the methodology presented in the Highway Design Manual (Caltrans, 2015a). Alternative sections were evaluated. The pavement sections were designed for a 20-year service life presuming that periodic maintenance, including crack sealing and resurfacing will be performed during the service life of the pavement. Premature deterioration may occur without periodic maintenance. Our recommendations for the pavement sections are presented in Table 10. Recommendations for subgrade preparation are presented in Sections 9.1.5.

Table 10 –Asphalt Concrete Pavement Structural Sections

Design R-Value	Traffic Index	Alternative 1	Alternative 2
5	5	3 inches AC 10 inches AB	3 inches AC 8 inches AB SEG
	6	3½ inches AC 13 inches AB	3½ inches AC 10 inches AB SEG
	7	4 inches AC 16 inches AB	4 inches AC 12 inches AB SEG

Notes:
[1] AC is Type A, Dense-Graded Hot Mix Asphalt complying with Caltrans Standard Specification 39-2 (2015b).
[2] AB is Class II Aggregate Base complying with Caltrans Standard Specification 26-1.02 (2015b).
[3] SEG is subgrade enhancement geotextile such as Mirafi 600X or equivalent.

AC and AB should conform to the material recommendations made in Section 9.1.4 and should be placed and compacted in accordance with the recommendations in Section 9.1.6. Concentrated runoff should not flow over the pavement as this can result in early deterioration of the pavement. We recommend that the paving operations be observed and tested by Ninyo & Moore.

9.5.2. Portland Cement Concrete Pavement

Portland cement concrete may be used in lieu of asphalt concrete for the proposed pavement sections. Our recommended pavement sections based on methodologies developed by the Portland Cement Associate (PCA) are presented in Table 11 for a 20-year design period with appropriate periodic maintenance. The recommended sections presume that the concrete will have a 28-day flexural strength of 600 psi or an equivalent compressive strength of 5,000 psi at 28 days.

Table 11 – Portland Cement Concrete Pavement Sections

Loading Condition[1]	Design Period	Subgrade Modulus[2]	Concrete Pavement Section
90 Annual Trucks (equivalent Traffic Index = 5)	20 years	75 pci	7½ inches PCC[3] 8 inches AB[4]
1,400 Annual Trucks (equivalent Traffic Index = 7)	20 years	75 pci	8½ inches PCC[3] 8 inches AB[4]

1 Assumes HS20 loading with one 8-kip and two 32-kip single axle loads.
2 Modulus of Subgrade Reaction in pounds per cubic inch (pci).
3 PCC is Portland Cement Concrete complying with Caltrans Standard Specification Section 90 (2015)
4 AB is Class II Aggregate Base complying with Caltrans Standard Specification Section 26 (2015)

Appropriate jointing of the concrete pavement can reduce the random occurrence of cracks. Joints should be laid out in a regular square pattern. Contraction, construction, and isolation joints should be detailed and constructed in accordance with the guidelines of the American Concrete Institute (ACI) Committee 302 (Manual of Concrete Practice [MCP], 2016). We recommend spacing contraction joints at 15 feet or less. Contraction joints formed by premolded inserts, grooving plastic concrete, or saw-cutting at initial hardening, should extend to a depth equivalent to 25 percent of the slab thickness and 1 inch or more for thin slabs. Contraction joints should be reinforced with smooth, 1-inch diameter, 14-inch long dowels placed across the joint at mid-slab height and spaced at 12 inches on center along the joint. However, contraction joints that are parallel and adjacent to pavement edges that are unrestrained by curbs or adjacent pavements should instead be reinforced with 30-inch long, No. 6 deformed steel bars placed across the joint at mid-slab height and spaced at 12 inches on center along the joint. Isolation joints subject to traffic loading should be thickened by 20 percent of the nominal thickness at the edge of the pavement with a 40:1 taper (horizontal to vertical) to the nominal slab thickness. Construction joints subject to traffic loading should be reinforced with smooth dowels as for contraction joints. Construction joints within the middle third of the typical joint spacing pattern should be reinforced with 30-inch long, No. 6 deformed steel bars placed across the joint near the middle of the slab and spaced at 30 inches on center. To reduce the potential for subsurface water intrusion into the subgrade and base layer, curbs or similar cutoff devices should be provided and joints should include a formed or sawcut reservoir for placement of foam backer rod and recessed, self-leveling silicone sealant. Periodic maintenance of the pavement should include sealing cracks that develop and replacement of joint sealant as-needed.

Distributed reinforcing steel may be placed to reduce the potential for differential slab movement, should cracking occur between joints. The distributed reinforcing steel should be terminated about 6 inches from contraction or isolation joints and should consist of No. 3 deformed bars at 18 inches on center, both ways near mid-height of the slab.

9.6. Concrete

Laboratory testing indicated that the concentration of sulfate and corresponding potential for sulfate attack on concrete is negligible for the soil tested. However, due to the variability in the on-site soil and the potential future use of reclaimed water at the site, we recommend that Type II/V or Type V cement be used for concrete structures in contact with soil. In addition, we recommend a water-to-cement ratio of no more than 0.45. A 3-inch thick, or thicker, concrete cover should be maintained over reinforcing steel where concrete is in contact with soil in accordance with Section 7.7 of ACI Committee 318 (ACI, 2016).

9.7. Moisture Vapor Retarder

The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. We recommend that the moisture vapor retarding system consist of a 4-inch-thick capillary break, overlain by a plastic membrane 15-mil-thick. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of ¾-inch nominal size. To reduce the potential for slab curling and cracking, an appropriate concrete mix with low shrinkage characteristics and a low water-to-cementitious-materials ratio should be specified. In addition, the concrete should be delivered and placed in accordance with ASTM C94 with attention to concrete temperature and elapsed time from batching to placement, and the slab should be cured in accordance with Section 302.1, 305, or 306 of the MCP (ACI, 2016), as appropriate. The plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. The bottom of the moisture barrier system should be higher in elevation than the exterior grade, if possible. Positive drainage should be established and maintained adjacent to foundations and flatwork. If a moisture vapor retarding system is to be constructed below an interior slab subject to vehicular loading, the blotter sand layer should be omitted or replaced with CLSM to reduce potential for slab pumping under load.

Where the exterior grade is at a higher elevation than the moisture vapor retarding system (including the capillary break layer), consideration should be given to constructing a subdrain around the foundation perimeter. The subdrain should consist of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be capped by a pavement or 12 inches of native soil and drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar). The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the foundation. The pipe should be located below the bottom elevation of the moisture vapor retarding system but above a plane extending down and away from the bottom edge of the foundation at a 2:1 (horizontal to vertical) gradient.

9.8. Surface Drainage and Site Maintenance

Surface drainage on the site should generally be provided so that water is diverted away from structures and is not permitted to pond. Positive drainage consisting of a gradient of 2 percent or more should be established for a distance of 5 feet or more adjacent to structures to divert surface water to an appropriate collector (graded swale, v-ditch, or area drain) with a suitable outlet. Slope, pad, and roof drainage (from adjacent structures) should be collected and diverted to suitable discharge areas away from structures or other slopes by non-erodible devices (e.g., gutters, downspouts, concrete swales, etc.). Graded swales, v-ditches, or curb and gutter should be provided at the site perimeter to restrict flow of surface water onto and off of the site. Slopes should be vegetated or otherwise armored to reduce potential for erosion of soil. Drainage structures should be periodically cleaned out and repaired, as-needed, to maintain appropriate site drainage patterns.

Landscaping adjacent to foundations should include vegetation with low-water demands and irrigation should limited to that which is needed to sustain the plants. Trees should be restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree.

Care should be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices on or adjacent to the project area. Drainage patterns established at the time of grading should be maintained for the life of the project. The property owner and maintenance personnel should be made aware that altering drainage patterns might be detrimental to wall performance.

Pavement surfaces should be crowned and/or sloped to divert water to edge gutters so that it is not permitted to pond or accumulate on the pavement. Edge gutters should be sloped to provide positive drainage to drop inlets or other drainage devices. Cracks that form in the pavement surface should be periodically sealed to reduce moisture intrusion into the aggregate base section.

9.9. Review of Construction Plans

The recommendations provided in this report are based on preliminary design information for the proposed construction. We recommend that a copy of the plans be provided to Ninyo & Moore for review before bidding to check the interpretation of our recommendations and that the designed improvements are consistent with our assumptions. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified to meet the project requirements.

9.10. Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions encountered in relatively widely spaced exploratory borings. During construction, the geotechnical engineer or his representative in the field should be allowed to check the exposed subsurface conditions. During construction, the geotechnical engineer or his representative should be allowed to:

- Observe preparation and compaction of subgrade.
- Check and test imported materials prior to use as fill.
- Observe placement and compaction of fill, aggregate base, and asphalt concrete.
- Perform field density tests to evaluate fill and subgrade compaction.
- Observe foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel and concrete.
- Observe placement of reinforcing steel in footings and slabs.
- Observe condition of water vapor retarding system prior to concrete placement.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, we request that the selected consultant provide a letter to the architect and the owner (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the recommendations contained in this report.

10. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur because of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

11. REFERENCES

American Concrete Institute, 2016, ACI Manual of Concrete Practice.

American Society for Testing and Materials (ASTM), 2015, Annual Book of ASTM Standards, West Conshohocken, Pennsylvania.

American Society of Civil Engineer (ASCE), 2010, Minimum Design Loads for Buildings and Other Structures, Standard 7-10.

Brelje & Race Consulting Civil Engineers (BRCE), 2016, Preliminary Grading Plan, Dated January.

California Building Standards Commission (CBSC), 2013, California Building Code (CBC), Title 24, Part 2, Volumes 1 and 2, based upon the 2012 International Building Code (IBC).

California Department of Transportation (Caltrans), 2012a, Caltrans Fault Database for ARS Online, Version 2b: dated December 13.

California Department of Transportation (Caltrans), 2012b, Corrosion Guidelines, Version 2.0, Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Technology Branch: dated November.

California Department of Transportation (Caltrans), 2015a, Highway Design Manual, http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm: updated December 30.

California Department of Transportation (Caltrans), 2015b, Standard Specifications.

California Division of Mines and Geology (CDMG), 1982, State of California Special Studies Zones, San Jose East Quadrangle: dated January 1: Scale 1:24,000.

California Division of Mines and Geology (CDMG), 2000, Seismic Hazard Zone Report for the San Jose East 7.5-Minute Quadrangle, Santa Clara County, California, Seismic Hazard Zone Report 044.

California Division of Mines and Geology (CDMG), 2001, Seismic Hazard Zone Map for the San Jose East 7.5-Minute Quadrangle, Santa Clara County, California, Scale: 1:24,000, dated January 17.

California Geological Survey (CGS), 2007, Special Publication 42 - Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act With Index to Earthquake Fault Zones Maps, by Bryant, W.A. and Hart, E.W.

Campbell, K., and Bozorgnia, Y., 2008, NGA ground motion model for the geometric mean horizontal component of PGA, PGV, PGD, and 5% damped linear elastic response spectra for periods ranging from 0.01 to 10 s.: Earthquake Spectra, Vol.24, pp.139-172.

Chiou, B., and Youngs, R., 2008, An NGA model for the average horizontal component of peak ground motion and response spectra: Earthquake Spectra, Vol.24, pp.173-216.

Dibblee, T.W., and Minch, J.A., 2005, Geologic map of the San Jose East quadrangle, Santa Clara County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-155, scale 1:24,000

Google Earth, 2015 Version No. 7.1.5.1557.

Helley, E.J., Graymer, R. W., Phelps, G. A.., Showalter, P. K., Wenworth, C. M., 1994, Quaternary Geology Map of the Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California, Scale 1:50000.

Idriss, I.M., and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute (EERI), Monograph MNO-12.

Jennings, C.W. and Bryant, William A., 2010, Fault Activity Map of California and Adjacent Areas: California Division of Mines and Geology, California Geologic Data Map Series, Map No. 6, Scale 1:750,000.

Occupational Safety and Health Administration (OSHA), 1989, Occupational Safety and Health Standards – Excavations, Department of Labor, Title 29 Code of Federal Regulations (CFR) part 1926, dated October 31.

Portland Cement Association (PCA), 1988, Design of Heavy Industrial Concrete Pavements, Publication IS234.01P, Skokie, Illinois.

Pradel, D. J., 1998, Procedure to Evaluate Earthquake Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, (ASCE), Volume 124, No. 4.

Santa Clara County (SCC), 2012, Geologic Hazard Zones, scale 1:24,000, dated: October 26.

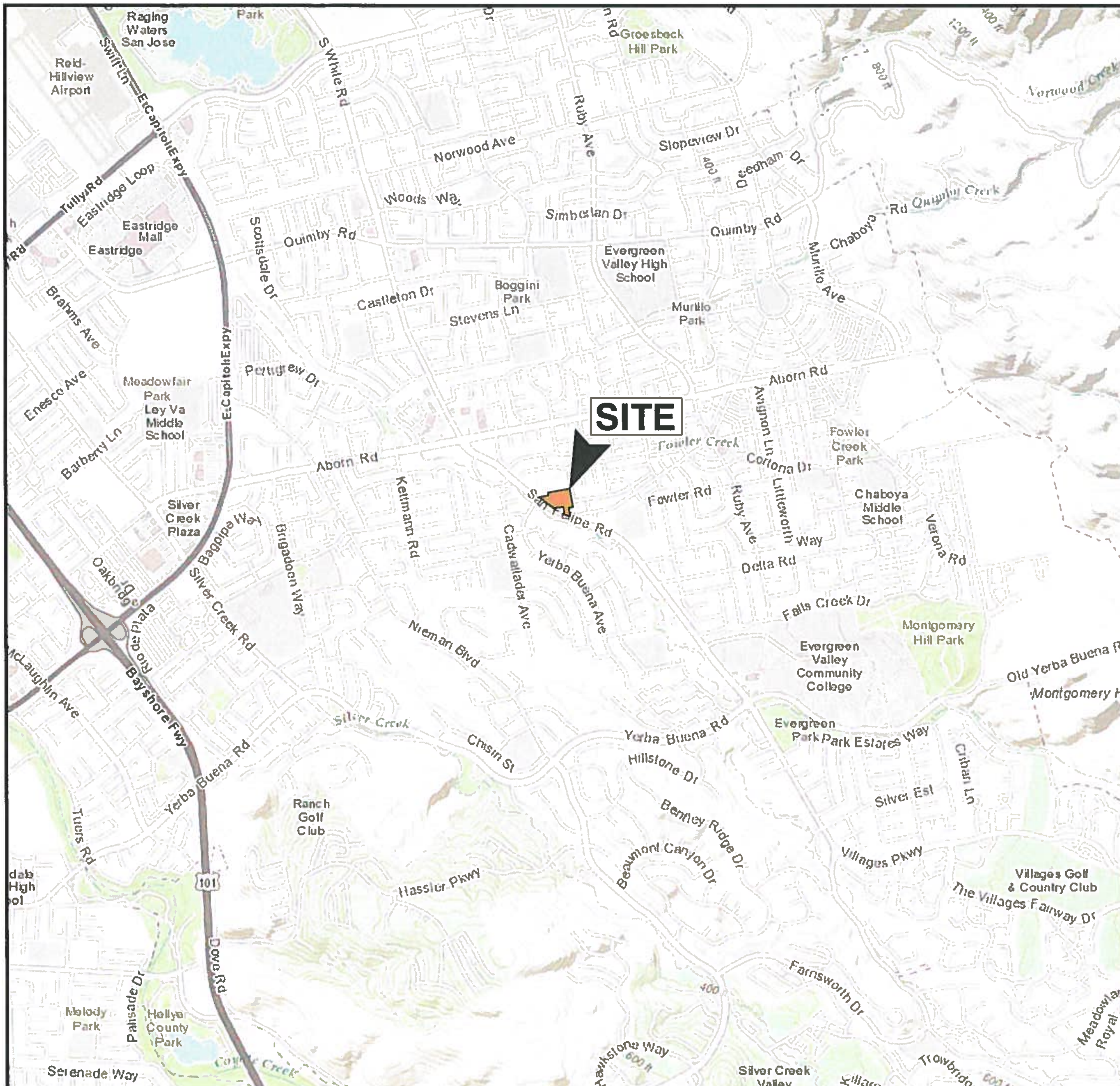
Tokimatsu, K., and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of the Geotechnical Engineering Division, ASCE, Vol. 113, No. 8, pp. 861-878.

United States Geologic Survey (USGS), 2016, U.S. Seismic Design Maps Application, accessed online at http://earthquake.usgs.gov/designmaps/us/application.php on May 5.

Wentworth, Carl M., Blake, M. Clarke., McLaughlin, Robert J., Graymer, Russell W., 1999, Preliminary Geologic Map of the San Jose 30' x 60' Quadrangle, Scale 1:100000.

Witter, Robert C., Knudsen, Keith L., Sowers, Janet M., Wentworth, Carl M., Koehler, Richard D., Randolph, Carolyn E., 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, Open-File Report 2006-1037, Version 1.1.

Working Group on California Earthquake Probabilities, 2008, Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): U.S. Geological Survey (USGS) Open-File Report 2007-1437 and California Geological Survey (CGS) Special Report 203: http://pubs.usgs.gov/of/2007/1437/.



REFERENCE: ESRI WORLD TOPO, 2016

N

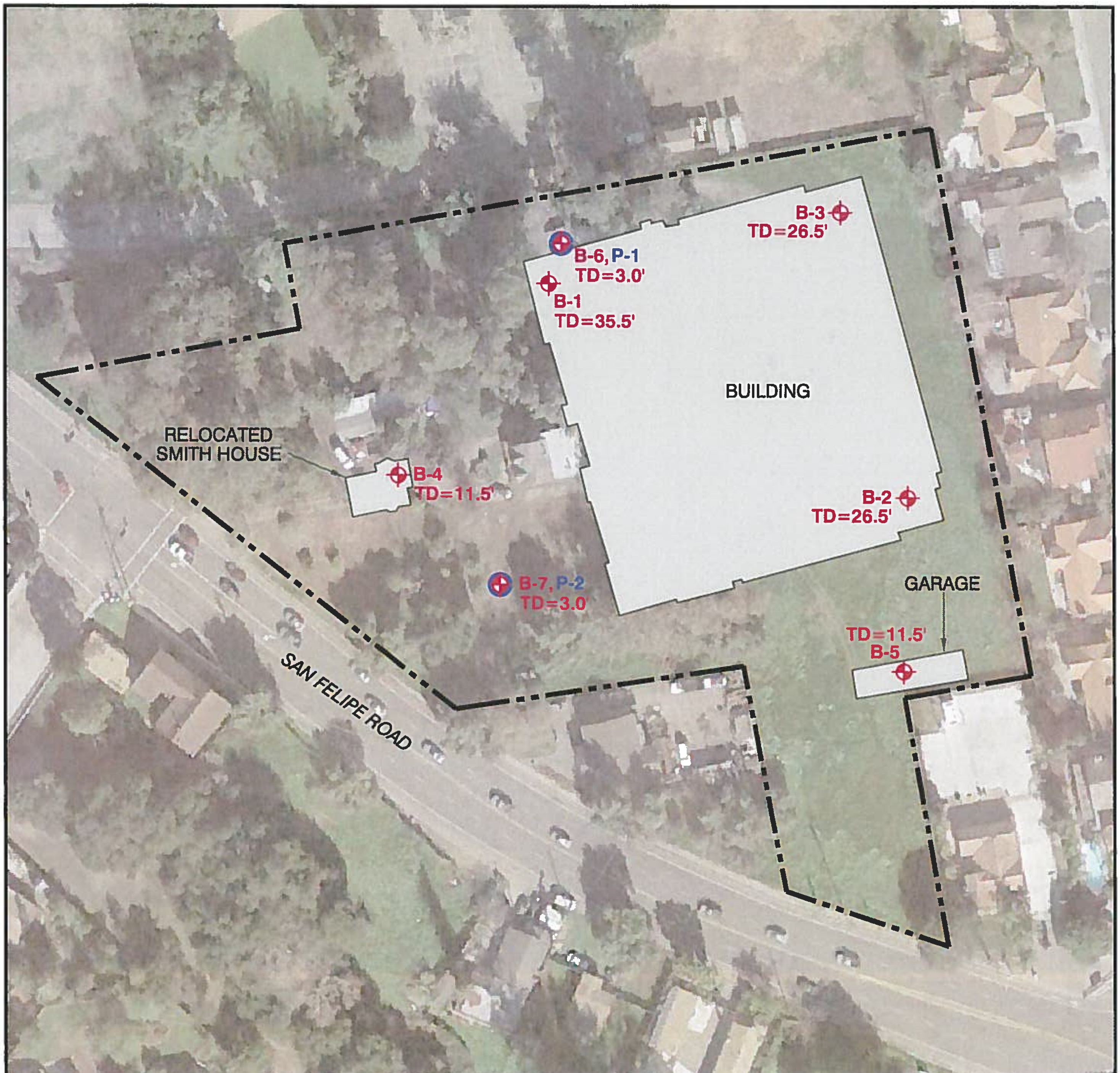
SCALE IN FEET

0 3,000 6,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

Ninyo & Moore		SITE LOCATION	FIGURE
PROJECT NO.	DATE	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	1
402679001	5/16		

1 402679001-SL.dwg May 12, 2016 10:54am SN



REFERENCE: GOOGLE EARTH IMAGERY, 2016.

N

SCALE IN FEET

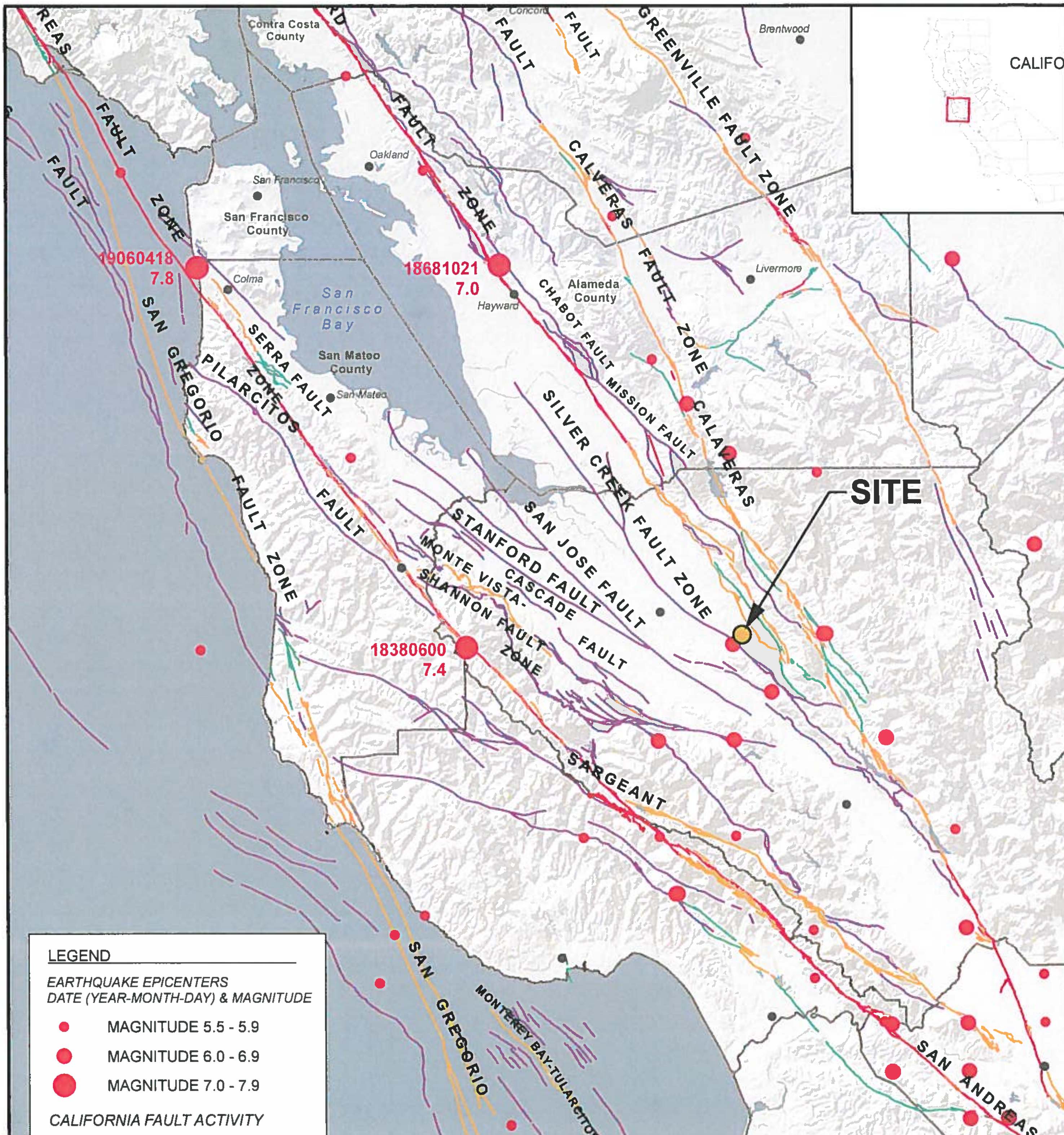
0 100 200

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND	
— · · —	SITE BOUNDARY
B-7 TD=3.0'	BORING LOCATION TD=TOTAL DEPTH IN FEET
P-2	PERCOLATION TEST LOCATION

Ninyo & Moore		BORING AND PERCOLATION TEST LOCATIONS	FIGURE
PROJECT NO.	DATE	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	2
402679001	5/16		

2. 402679001-BL.dwg, May 12, 2016, 10:52am, SN



LEGEND

EARTHQUAKE EPICENTERS
DATE (YEAR-MONTH-DAY) & MAGNITUDE

- MAGNITUDE 5.5 - 5.9
- MAGNITUDE 6.0 - 6.9
- MAGNITUDE 7.0 - 7.9

CALIFORNIA FAULT ACTIVITY

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- LATE QUATERNARY (POTENTIALLY ACTIVE)
- QUATERNARY (POTENTIALLY ACTIVE)
- STATE/COUNTY BOUNDARY

SOURCES: FAULTS - USGS & CGS, 2006, QUATERNARY FOLD AND FAULT DATABASE FOR THE UNITED STATES.
EARTHQUAKES - CALIFORNIA GEOLOGICAL SURVEY, 2000, MAP SHEET MS-49.

SCALE IN MILES

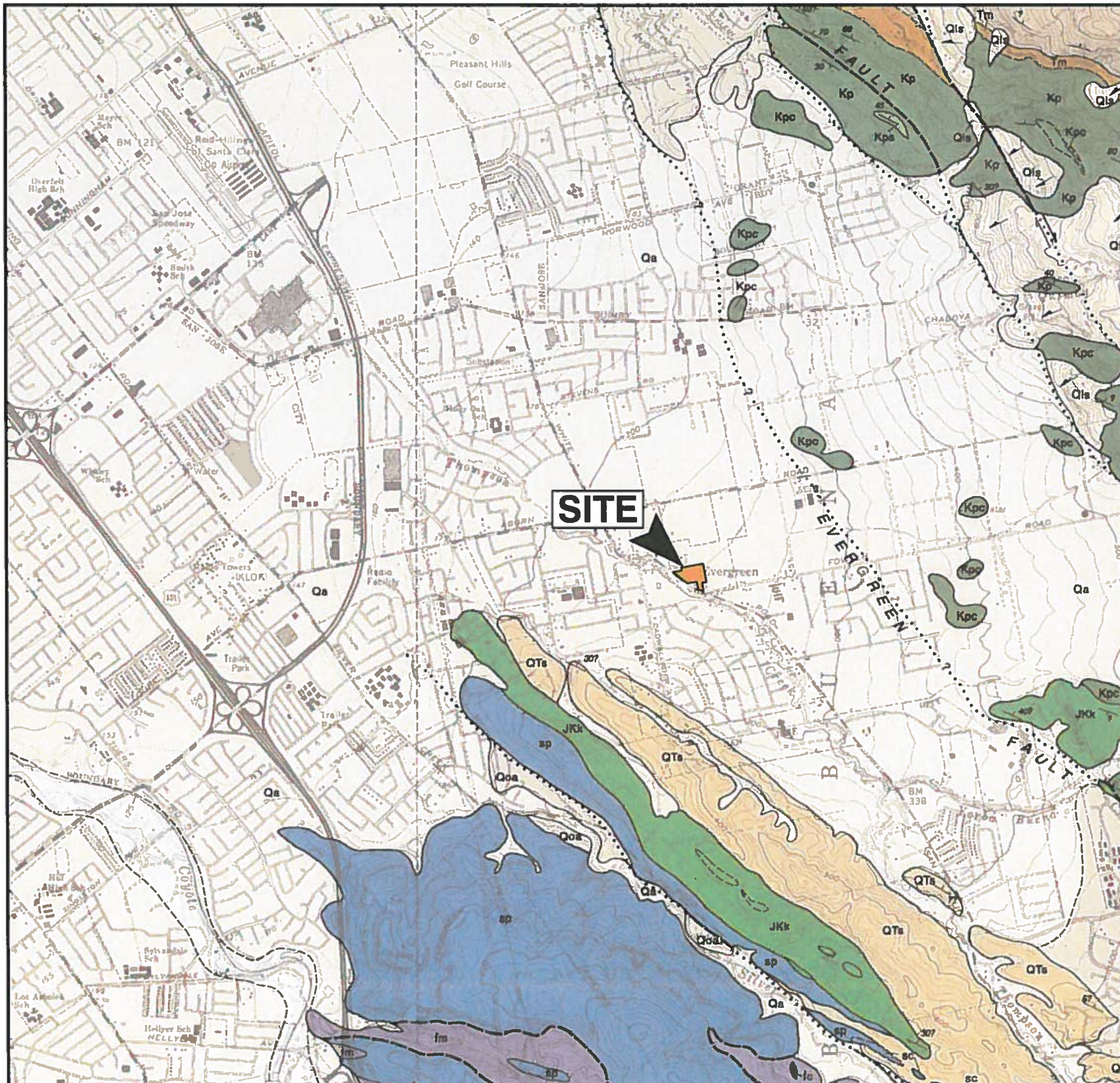
0 10 20

fault_loc_EPIC_OAK_central_93.mxd SN

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore

PROJECT NO.	DATE	FAULT LOCATIONS AND EARTHQUAKE EPICENTERS	FIGURE
402679001	5/16	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	3



REFERENCE: DIBBLEE, JR., T.W., 2005, GEOLOGIC MAP OF THE SAN JOSE EAST QUADRANGLE, UNITED STATES GEOLOGICAL SURVEY, SCALE 1:24,000, DATED APRIL.

N

SCALE IN FEET

0 3,000 6,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

Symbol	Description
Qa	SURFICIAL SEDIMENTS
Qoa	OLDER SURFICIAL SEDIMENTS
QTs	SANTA CLARA FORMATION
Kpc	PANOCHE FORMATION
JKk	KNOXVILLE FORMATION
sp	COAST RANGE OPHIOLITE COMPLEX

GEOLOGIC CONTACT

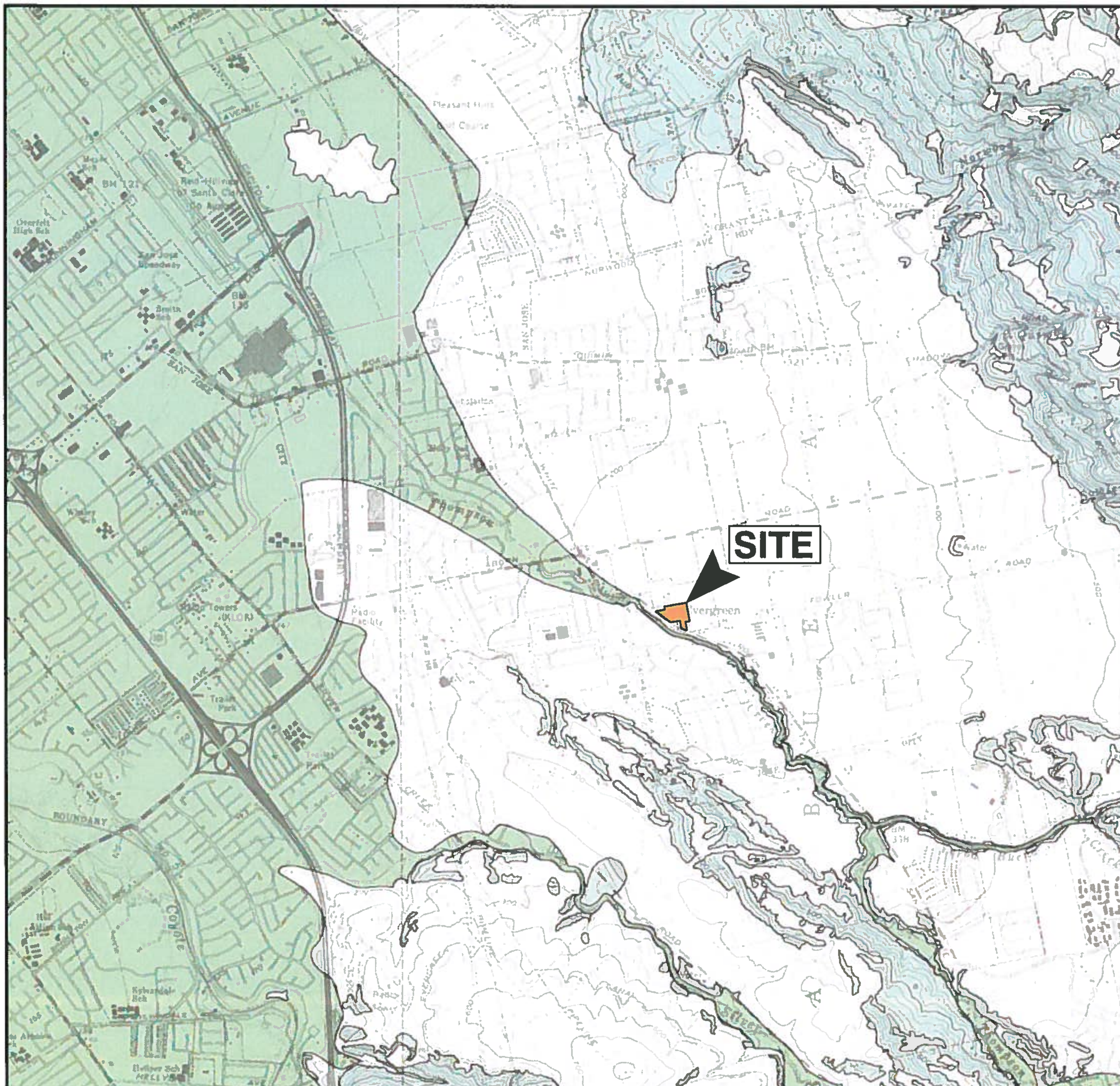
45 STRIKE AND DIP

ANTICLINE

FAULT

Ninyo & Moore	REGIONAL GEOLOGY	FIGURE
PROJECT NO. 402679001	DATE 5/16	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	4

4_402679001-RG.dwg, May 12, 2016, 10:43am, SN



REFERENCE: CALIFORNIA GEOLOGICAL SURVEY, 2001, SEISMIC HAZARD ZONES, SAN JOSE EAST QUADRANGLE, SCALE 1:24,000.

N

SCALE IN FEET

0 3,000 6,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

LIQUEFACTION:
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

EARTHQUAKE-INDUCED LANDSLIDES:
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Ninyo & Moore	SEISMIC HAZARD ZONES	FIGURE
PROJECT NO. 402679001 / DATE 5/16	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	5

5_402679001-SHZ.dwg, May 12, 2016, 10:54am, SN

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following method.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with a 6-inch long, thin brass liners with an inside diameter of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring log as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

Field Testing

The following tests were performed in the field to evaluate soil properties.

Static Cone Penetrometer

A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 1.5 square centimeters was manually pushed 6 inches into the soil. The penetrometer was instrumented to measure the Cone Penetration Index (Q_c) computed as the peak force on the cone divided by the cone base area. The Cone Penetration Index is reported in kilograms per square centimeter (ksc) on the boring log at the depth of the test as a measure of the relative density or consistency of the soil encountered.

Pocket Penetrometer

A pocket penetrometer was inserted into soil samples collected from split spoon samplers to evaluate the unconfined compressive strength of the soil. The unconfined compressive strength as evaluated by the pocket penetrometer (PP) is reported on the boring logs in tons per square foot (tsf).

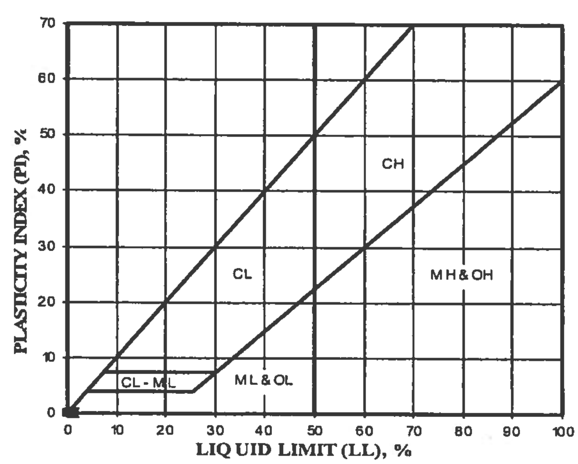
U.S.C.S. METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (More than 1/2 of soil >No. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction <No. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 1/2 of soil <No. 200 sieve size)	SILTS & CLAYS Liquid Limit <50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit >50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

GRAIN SIZE CHART

CLASSIFICATION	RANGE OF GRAIN SIZE	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL Coarse Fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND Coarse Medium Fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.074 4.76 to 2.00 2.00 to 0.420 0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

PLASTICITY CHART



Ninyo & Moore | U.S.C.S. METHOD OF SOIL CLASSIFICATION

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	BORING LOG EXPLANATION SHEET
0								Bulk sample.
								Modified split-barrel drive sampler.
								2-inch inner diameter split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.
								Sample retained by others.
5								Standard Penetration Test (SPT).
								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
10								Seepage.
								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	<u>MAJOR MATERIAL TYPE (SOIL)</u>: Solid line denotes unit change.
							CL	Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

Ninyo & Moore

BORING LOG		
Explanation of Boring Log Symbols		
PROJECT NO.	DATE	FIGURE

DATE DRILLED	4/14/16
BORING NO.	B-1
GROUND ELEVATION	240' ± (MSL)
SHEET	1 OF 2
METHOD OF DRILLING	8" HSA, B-53 Truck-Mounted Drill Rig, Exploration Geoservices
DRIVE WEIGHT	140 LBS (Wireline)
DROP	30"
SAMPLED BY	RH
LOGGED BY	RH
REVIEWED BY	PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	FIELD TESTS	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								SC	ASPHALT PAVEMENT: Approximately 2 inches thick.
	X					Qc>50			ALLUVIUM: Yellowish brown, dry, medium dense, clayey SAND; some gravel.
	X					Qc>50			
	X					Qc>50			
	X					Qc>50			
		X	42			PP>5.0			
								CL	Yellowish brown, dry, hard, lean CLAY with sand; trace gravel.
10		X	69			PP>5.0			
		X	52			PP>5.0			Brown.
								SW	Brown, dry, dense, well-graded SAND; some gravel.
20		X	72						
		X	59						Very dense.
30		X	43						Dense.
		X	53						Very dense.
									Total depth = 35.5 feet.
									Backfilled with soil cuttings and cement grout on 4/14/16.
40									

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BORING LOG

OAKMONT OF EVERGREEN
3550 SAN FELIPE ROAD, SAN JOSE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402679001	5/16	A-1

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	FIELD TESTS	SYMBOL	CLASSIFICATION U.S.C.S.

DATE DRILLED 4/14/16 BORING NO. B-1

GROUND ELEVATION 240' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" HSA, B-53 Truck-Mounted Drill Rig, Exploration Geoservices

DRIVE WEIGHT 140 LBS (Wireline) DROP 30"

SAMPLED BY RH LOGGED BY RH REVIEWED BY PCC

DESCRIPTION/INTERPRETATION

40

Notes:

Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation show above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation including topographic maps provided by BRCE. It is not sufficiently accurate for preparing construction bids and design documents.

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Ninyo & Moore

BORING LOG

OAKMONT OF EVERGREEN
3550 SAN FELIPE ROAD, SAN JOSE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402679001	5/16	A-2

DATE DRILLED 4/14/16 BORING NO. B-2

GROUND ELEVATION 242' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HSA, B-53 Truck-Mounted Drill Rig, Exploration Geoservices

DRIVE WEIGHT 140 LBS (Wireline) DROP 30"

SAMPLED BY RH LOGGED BY RH REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	FIELD TESTS	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						Qc=15		CL	ALLUVIUM: Brown, moist, stiff, lean CLAY with sand.
						Qc=15			
						Qc=15			
						Qc=20			
						Qc=20			
			12	11.9	109.4	PP=3.0			
10			31	8.1	106.9	PP>5.0			Dry, very stiff.
			29	12.4	100.9	PP>5.0			Light brown, moist.
								SC	Brown, moist, medium dense, clayey SAND; trace gravel.
20			34						
								CL	Brown, moist, very stiff, lean CLAY; trace sand.
			22			PP>5.0			
30									
40									

Total depth = 26.5 feet.

Backfilled with soil cuttings and cement grout on 4/14/16.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation show above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation including topographic maps provided by BRCE. It is not sufficiently accurate for preparing construction bids and design documents.

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BORING LOG

OAKMONT OF EVERGREEN
3550 SAN FELIPE ROAD, SAN JOSE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402679001	5/16	A-3

DATE DRILLED 4/14/16 BORING NO. B-3

GROUND ELEVATION 239' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HSA, B-53 Truck-Mounted Drill Rig, Exploration Geoservices

DRIVE WEIGHT 140 LBS (Wireline) DROP 30"

SAMPLED BY RH LOGGED BY RH REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	FIELD TESTS	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						Qc=30		CL	ALLUVIUM:
						Qc=35			Brown, moist, very stiff, lean CLAY with sand.
						Qc=35			
						Qc=30			
						Qc=35			
			21			PP=4.5			
10			18			PP=4.5			Yellowish brown; stiff; trace sand.
			31			PP=5.0			Very stiff; dry.
20			53			PP>5.0			Brown; hard.
			52					GW	Gray, dry, dense, well-graded GRAVEL.
									Total depth = 26.5 feet.
									Backfilled with soil cuttings and cement grout on 4/14/16.
30									Notes:
									Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
									The ground elevation show above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation including topographic maps provided by BRCE. It is not sufficiently accurate for preparing construction bids and design documents.
40									

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BORING LOG

OAKMONT OF EVERGREEN
3550 SAN FELIPE ROAD, SAN JOSE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402679001	5/16	A-4

DATE DRILLED 4/14/16 BORING NO. B-4

GROUND ELEVATION 240' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HSA, B-53 Truck-Mounted Drill Rig, Exploration Geoservices

DRIVE WEIGHT 140 LBS (Wireline) DROP 30"

SAMPLED BY RH LOGGED BY RH REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	FIELD TESTS	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						Qc>50 Qc>50		CL	ALLUVIUM: Brown, dry, very stiff, lean CLAY with sand.
						Qc>50 Qc>50		SP-SC	Yellowish brown, dry, medium dense, poorly graded SAND with clay.
			37						
								CL	Yellowish brown, dry, hard, lean CLAY; trace sand.
10			68						
									Total depth = 11.5 feet.
20									
30									
40									

Total depth = 11.5 feet.

Backfilled with soil cuttings and cement grout on 4/14/16.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation show above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation including topographic maps provided by BRCE. It is not sufficiently accurate for preparing construction bids and design documents.

DATE DRILLED	4/14/16	BORING NO.	B-5
GROUND ELEVATION	243' ± (MSL)	SHEET	1 OF 1
METHOD OF DRILLING	8" HSA, B-53 Truck-Mounted Drill Rig, Exploration Geoservices		
DRIVE WEIGHT	140 LBS (Wireline)	DROP	30"
SAMPLED BY	RH	LOGGED BY	RH
REVIEWED BY	PCC		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	FIELD TESTS	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						Qc=10		CL	ALLUVIUM:
	X					Qc=15			Brown, moist, stiff, lean CLAY; some sand.
	X					Qc=25			
	X					Qc=20			
	X					Qc=20			
		X	25						Dry; very stiff.
10		X	31						
									Total depth = 11.5 feet.
									Backfilled with soil cuttings and cement grout on 4/14/16.
									Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
20									The ground elevation show above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation including topographic maps provided by BRCE. It is not sufficiently accurate for preparing construction bids and design documents.
30									
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Ninyo & Moore

BORING LOG

OAKMONT OF EVERGREEN
3550 SAN FELIPE ROAD, SAN JOSE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402679001	5/16	A-6

DATE DRILLED 4/14/16								BORING NO. B-6
GROUND ELEVATION 239' ± (MSL)								SHEET 1 OF 1
METHOD OF DRILLING 8" HSA, B-53 Truck-Mounted Drill Rig, Exploration Geoservices								
DRIVE WEIGHT NA								DROP NA
SAMPLED BY RH LOGGED BY RH REVIEWED BY PCC								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	FIELD TESTS	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						Qc=15 Qc=15 Qc=15		CL	ALLUVIUM: Dark brown, moist, stiff, lean CLAY; trace sand.
						Qc=25		SC	Brown, moist, medium dense, clayey SAND; trace gravel.
									Total depth = 3.0 feet. Percolation test performed upon completion. Backfilled with soil cuttings on 4/14/16. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation show above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation including topographic maps provided by BRCE. It is not sufficiently accurate for preparing construction bids and design documents.
10									
20									
30									
40									

Ninyo & Moore

BORING LOG

OAKMONT OF EVERGREEN
3550 SAN FELIPE ROAD, SAN JOSE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402679001	5/16	A-7

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	FIELD TESTS	SYMBOL	CLASSIFICATION U.S.C.S.

DATE DRILLED 4/14/16 BORING NO. B-7

GROUND ELEVATION 241' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HSA, B-53 Truck-Mounted Drill Rig, Exploration Geoservices

DRIVE WEIGHT NA DROP NA

SAMPLED BY RH LOGGED BY RH REVIEWED BY PCC

DESCRIPTION/INTERPRETATION

DEPTH (feet)	FIELD TESTS	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	Qc=20 Qc=15 Qc=15 Qc=15	CL	ALLUVIUM: Dark brown, moist, stiff, lean CLAY; trace sand.
			Total depth = 3.0 feet. Percolation test performed upon completion. Backfilled with soil cuttings on 4/14/16. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation show above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation including topographic maps provided by BRCE. It is not sufficiently accurate for preparing construction bids and design documents.
10			
20			
30			
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Ninyo & Moore

BORING LOG

OAKMONT OF EVERGREEN

3550 SAN FELIPE ROAD, SAN JOSE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402679001	5/16	A-8

APPENDIX B

LABORATORY TESTING

Classification
Soil was visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture Content
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2216. The test results are presented on the logs of the exploratory borings in Appendix A.

In-Place Density Tests
The dry density of relatively undisturbed samples obtained from the exploratory borings was evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the borings in Appendix A

Gradation Analysis
A gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain-size distribution curve is shown on Figure B-1. The test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System (USCS).

Atterberg Limits
Tests were performed on a selected representative soil sample to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classification are shown on Figure B-2.

Direct Shear Tests
A direct shear test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figures B-3.

Expansion Index Test
The expansion index of a selected material was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The test results are presented on Figure B-4.

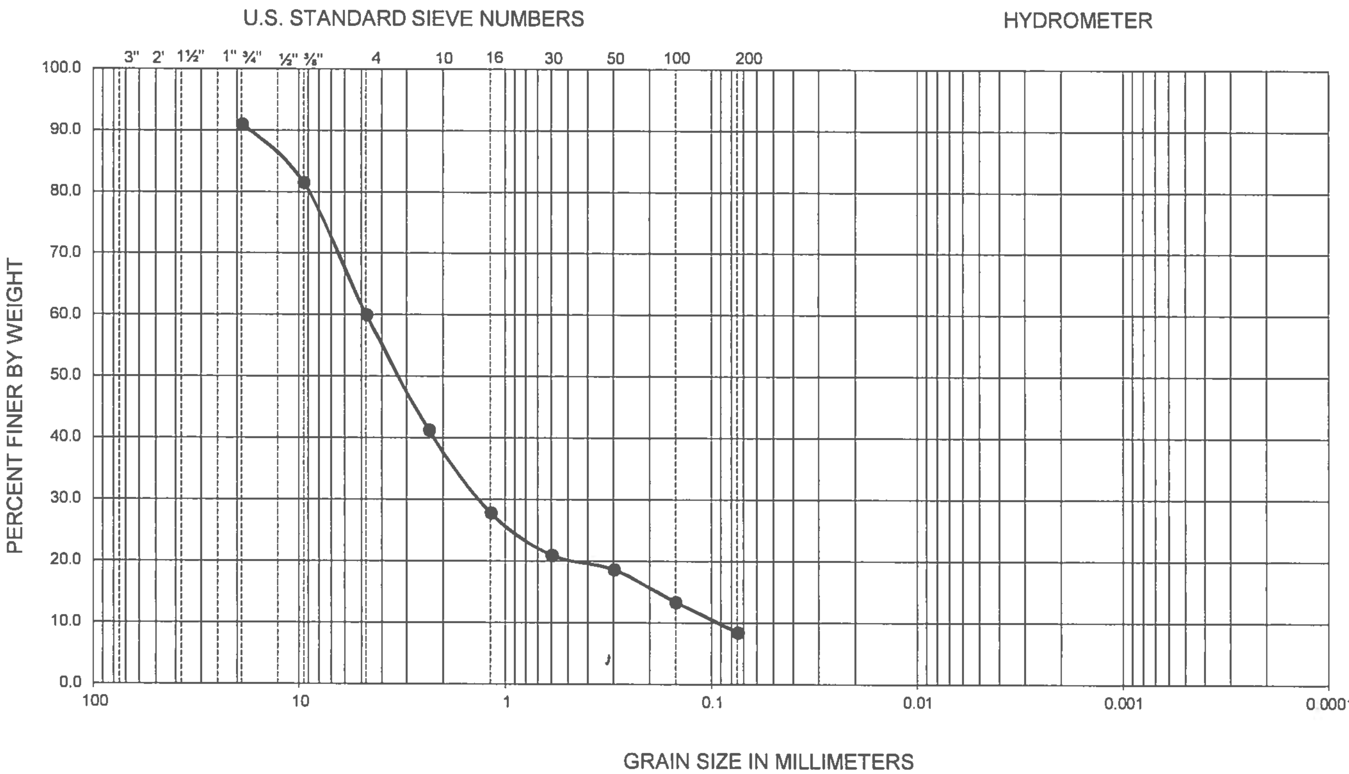
Soil Corrosivity Tests

Soil pH, and resistivity tests were performed on a representative sample in general accordance with California Test (CT) 643. The soluble sulfate and chloride content of selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-5.

R-Value

The resistance value, or R-value, for site soil was evaluated in general accordance with California Test (CT) 301. The sample was prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test result is shown on Figure B-6.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

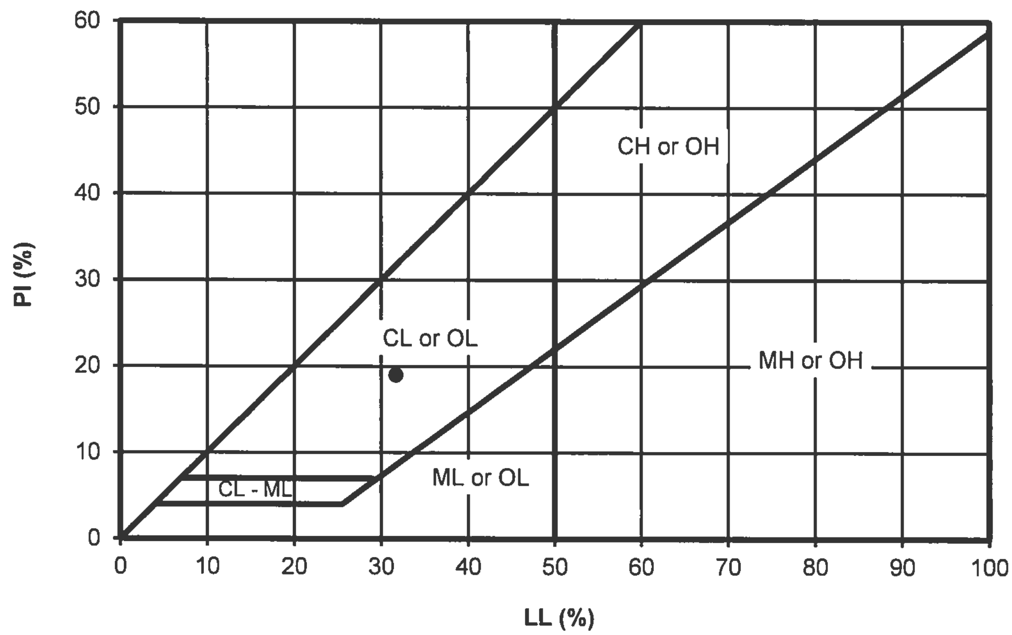


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-4	6.0-6.5	--	--	--	0.10	1.37	4.74	48.0	4.0	8	SP-SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

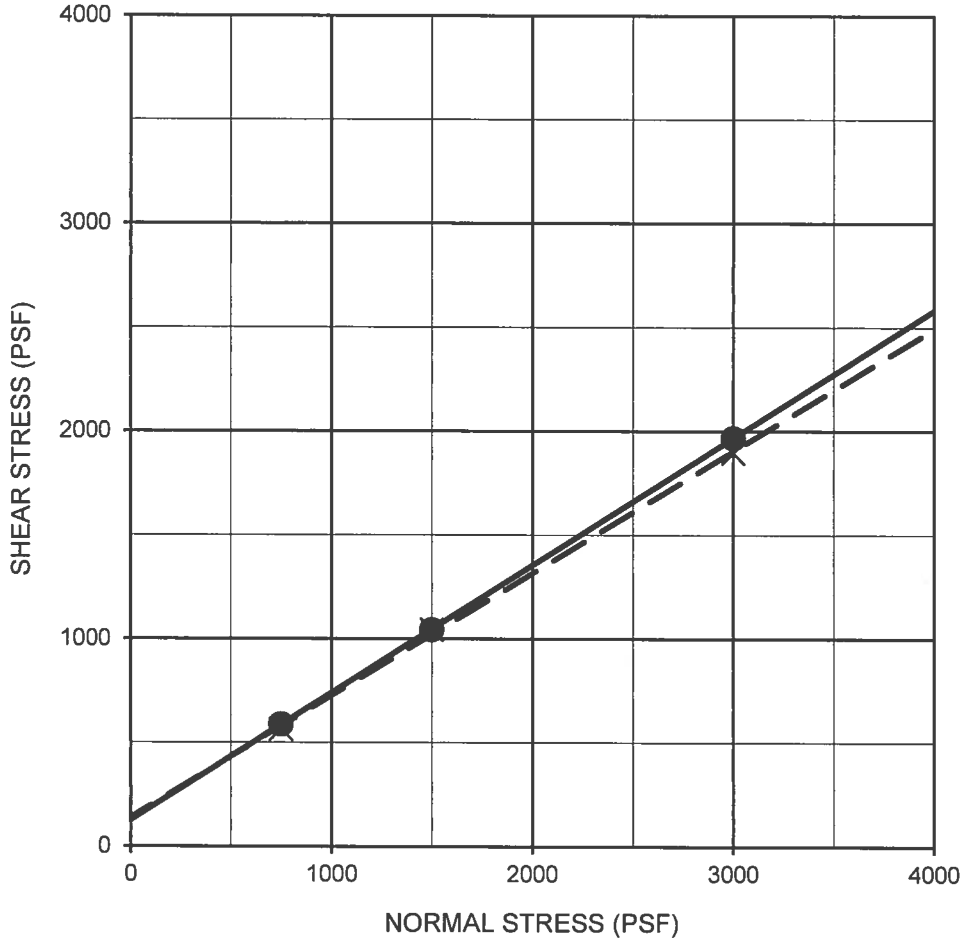
Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	B-1
402679001	5/16		

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL (%)	PLASTIC LIMIT, PL (%)	PLASTICITY INDEX, PI (%)	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-2	6.0-6.5	32	13	19	CL	CL



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Ninyo & Moore		ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO.	DATE	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	B-2
402679001	5/16		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Lean CLAY	——●——	B-3	6.0-6.5	Peak	130	32	CL
Lean CLAY	- - X - -	B-3	6.0-6.5	Ultimate	140	30	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	B-3
402679001	5/16		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-3	1.0-5.0	13.0	96.5	26.4	0.033	32	Low

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4829

Ninyo & Moore

EXPANSION INDEX TEST RESULTS

PROJECT NO.	DATE
402679001	5/16

OAKMONT OF EVERGREEN
3550 SAN FELIPE ROAD
SAN JOSE, CALIFORNIA

FIGURE
B-4

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2] (ppm)	SULFATE CONTENT [2] (%)	CHLORIDE CONTENT [3] (ppm)
B-3	1.0-5.0	7.4	1,700	10	0.001	205

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore	CORROSIVITY TEST RESULTS	FIGURE
PROJECT NO. 402679001 / DATE 5/16	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	B-5

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
B-5	1.0-5.0	CL	6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

Ninyo & Moore		R-VALUE TEST RESULTS	FIGURE
PROJECT NO.	DATE	OAKMONT OF EVERGREEN 3550 SAN FELIPE ROAD SAN JOSE, CALIFORNIA	B-6
402679001	5/16		

RVTABLE1

APPENDIX C

PERCOLATION TESTING

Field Procedure for Percolation Testing by Double Ring Infiltrometer

To evaluate the permeability of near surface soils at the site near the proposed storm water management areas, Ninyo & Moore conducted percolation testing in general accordance with locally accepted practices for soil percolation testing. Percolation testing was performed at approximately 3 feet below the ground surface near each boring location as shown on Figure 2. The subsurface materials encountered at the test locations are described on the boring logs. A 6-inch diameter outer PVC pipe and a 4-inch diameter inner PVC pipe were placed into the test hole. Observations of the materials encountered during excavation of the hole indicated that a pre-soak period was not necessary. The test hole was filled with water and the water level in the percolation test hole was monitored for approximately 6 to 7 hours. The water surface elevation was recorded at approximately 15 to 60-minute intervals. For each interval, the volume of water that infiltrated the ground and the percolation rate was calculated for the inner ring and the annulus between the two rings. The average infiltration for the inner ring over consecutive intervals with consistent rate of drop is reported.

Ninyo & Moore

SUMMARY OF DOUBLE-RING INFILTROMETER RESULTS

PROJECT: Oakmont of Evergreen | PROJECT NO. 402679001 | LOCATION: P-1 | DATE: 4/14/2016

TESTED BY: RH | Ring Penetration: 2 inches | Ring Area (in^2)

Liquid Used: Tap Water | Inner (in): 4 | Outer (in): 6 | Inner: 12.6 | Annular: 15.7

Trial No.	Time (hr:mm)	Δ Time (min)	Total Elapsed Time (minutes)	Water Level (in) Inner	Water Level (in) Annular	Liquid Temp (°F)	Increment Infiltration Rate Inner (in/hr)	Increment Infiltration Rate Annular Space (in/hr)	Notes:
1	9:01	16.00	16.00	8	8	70	0.94	0.94	
	9:17			7.75	7.75	70			
2	9:17	14.00	30.00	7.75	7.75	70	1.07	1.07	
	9:31			7.5	7.5	70			
3	9:31	64.00	94.00	7.5	7.5	70	0.82	0.82	
	10:35			6.625	6.625	70			
4	10:35	55.00	149.00	6.625	6.625	70	0.82	0.82	
	11:30			5.875	5.875	70			
5	11:30	64.00	213.00	5.875	5.875	70	0.70	0.70	
	12:34			5.125	5.125	70			
6	12:34	47.00	260.00	8	8	70	0.80	0.80	
	13:21			7.375	7.375	70			
7	13:21	71.00	331.00	7.25	7.25	70	0.74	0.74	
	14:32			6.375	6.375	70			
8	14:32	63.00	394.00	6.375	6.375	70	0.71	0.71	
	15:35			5.625	5.625	70			
9	15:35	32.00	426.00	5.625	5.625	70	0.70	0.70	
	16:07			5.25	5.25	70			

FINAL READING 85 MINUTES/INCH

Ninyo & Moore

SUMMARY OF DOUBLE-RING INFILTROMETER RESULTS

PROJECT: Oakmont of Evergreen **PROJECT NO.** 402679001 **LOCATION:** P-2 **DATE:** 4/14/2016

TESTED BY: RH **Ring Penetration: 2 inches** **Ring Area (in^2)**

Liquid Used: Tap Water **Inner (in):** 4 **Outer (in):** 6 **Inner:** 12.6 **Annular:** 15.7

Trial No.	Time (hr:mm)	Δ Time (min)	Total Elapsed Time (minutes)	Water Level (in) Inner	Water Level (in) Annular	Liquid Temp (°F)	Increment Infiltration Rate Inner (in/hr)	Increment Infiltration Rate Annular Space (in/hr)	Notes:
1	10:19	66.00	66.00	8	8	70	0.45	0.45	
	11:25			7.5	7.5	70			
2	11:25	55.00	121.00	7.5	7.5	70	0.41	0.41	
	12:20			7.125	7.125	70			
3	12:20	65.00	186.00	7.125	7.125	70	0.35	0.35	
	13:25			6.75	6.75	70			
4	13:25	58.00	244.00	6.75	6.75	70	0.26	0.26	
	14:23			6.5	6.5	70			
5	14:23	32.00	276.00	6.5	6.5	70	0.23	0.23	
	14:55			6.375	6.375	70			
6	14:55	70.00	346.00	6.375	6.375	70	0.21	0.21	
	16:05			6.125	6.125	70			

FINAL READING 280 MINUTES/INCH

APPENDIX D

CALCULATIONS

LIQUEFACTION AND DYNAMIC SETTLEMENT WORKSHEET

JOB NO.: 402679001 **JOB NAME:** Oakmont/Evergreen
CALCULATION BY: PCC **DATE:** 5/13/2016
CHECKED BY: **DATE:**
BORING/PROFILE: B-1

Depth (z) of Layer Top	Bottom	Formation	Soil Type	Thick. t (ft.)	Midpoint of Layer (ft.)	Layer γ (pcf)	Total Stress σ_v (ksf)	Effect. Stress σ_v' (ksf)	$(N_1)_{60}$	Fines Content FC (%)	$(N_1)_{60cs}$ Note 1	r_d Note 2	CSR_M Note 3	Ks Note 5	$CRR_{7.5}$ σ_v'=1atm Note 6	CRR Note 7	FOS_{liq} Note 8	$CSR_{7.5}$ Note 9	$(N_1)_{60}$+(N)corr Note 10	ε_v (%) Fig. 4-3	Sat. Sand Settlemnt ΔH_{sat} (in) Note 11	τ_{avg} (tsf)	mean effective stress σ_m' (tsf)	max Shear Modulus G_{max} (tsf)	a	b	γ (%)	ε_{15} (%)	ε_{Nc} (%)	dry sand settlement ρ_e (in.)
0	6	Alluvium	SC	6	3	120	0.36	0.36	24	30	29	0.998	0.363	1.100	0.448	0.577		0.310	26	1.0		0.07	0.12	482	0.128528	23260	0.046246	0.035215	0.029446	0.04
6	12	Alluvium	CL	6	9	120	1.08	1.08	50	70	56	0.976	0.355	1.100	2.000	2.000		0.303	54	0.0		0.19	0.35	1065						
12	17	Alluvium	CL	5	14.5	120	1.74	1.74	37	70	43	0.953	0.347	1.058	2.000	2.000		0.296	41	0.0		0.30	0.56	1234						
17	20	Alluvium	SW	3	18.5	120	2.22	2.22	33	5	33	0.934	0.340	0.989	0.759	0.880		0.290	33	0.0		0.38	0.72	1296	0.151922	7809	0.06255	0.034296	0.028677	0.02
20	23	Alluvium	SW	3	21.5	120	2.58	2.49	33	5	33	0.919	0.347	0.962	0.759	0.856	2.47	0.296	33	0.0	0.00	0.43	0.80	1372						
23	28	Alluvium	SW	5	25.5	120	3.06	2.72	46	5	46	0.898	0.368	0.927	2.000	2.000	5.43	0.314	46	0.0	0.00	0.50	0.88	1602						
28	32	Alluvium	SW	4	30	120	3.60	2.98	32	5	32	0.873	0.384	0.924	0.644	0.698	1.81	0.328	32	0.0	0.00	0.57	0.96	1486						
32	35.5	Alluvium	SW	3.5	33.75	120	4.05	3.19	37	5	37	0.852	0.393	0.879	1.750	1.803	4.58	0.336	37	0.0	0.00	0.63	1.03	1615						

Total Settlement = 0.00 0.06

20 = d_w, depth to groundwater table (ft)
6.9 = M, moment magnitude of design earthquake
0.56 = a_{max}, peak horizontal ground acceleration for design earthquake (g)

NOTES:

1 clean sand blowcounts @ 1 tsf overburden @ 60% energy ratio, $(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$, where $\Delta(N_1)_{60}$ = exp[1.63+9.7/(FC+0.01)-(15.7/(FC+0.01))^2]
2 stress reduction factor, r_d = exp[-1.012-1.126*sin(z/3.3/11.73+5.133)+M*(0.106+0.118*sin(z/3.3/11.28+5.142))]
3 cyclic stress ratio @ M, $CSR_M = \tau_{avg}/\sigma_v' = 0.65(a_{max}/g)(\sigma_v)(r_d)/(\sigma_v')$
4 magnitude scaling factor, MSF = min[1.8,6.9*exp(-M/4)-0.058] ==> MSF = 1.17
5 overburden correction factor, Ks = min[1.1,1-(1/(18.9-2.55*sqrt(min[(N1)60cs,37]))*ln[sv'/2.12]]
6 cyclic resistance ratio @ M=7.5 sv'=1atm, $CRR_{7.5,1}$ = 2 if (N1)60cs < 37.5, else exp[(N1)60cs/14.1 + ((N1)60cs/126)^2 - ((N1)60cs/23.6)^3 + ((N1)60cs/25.4)^4 - 2.8]
6 cyclic resistance ratio @ M,sv', CRR =min[2,CRR7.5,1atm*Ks*MSF]
8 factor of safety against liquefaction, $FOS_{liq} = (CRR/CSR_M)$
9 cyclic stress ratio @ M=7.5, $CSR_{7.5} = CSR_M/MSF$
10 N-value correction for Fines Content for Settlement Analysis (Table 7.2, SCEC, 1999)
11 settlement of saturated sand, $\Delta H_{sat} = \varepsilon_v{}^*t$
12 Coefficient of Lateral Earth Pressure at Rest, Ko: Ko = 0.47
13 Number of Strain Cycles, Nc: Nc = 10.0787

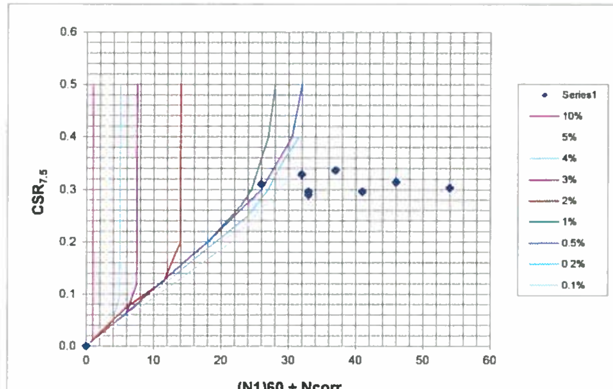


Fig 4-3: Tokimatsu & Seed, 1987

REFERENCES:
Idriss, I.M. & Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, Monograph MNO-12
Tokimatsu, K. & Seed, H.B., 1987, Evaluation of Settlements in Sand Due to Earthquake Shaking, Journal of Geotechnical Engineering Division, ASCE, Vol 113, No 8.
Pradel, D.J., 1998, Procedure to Evaluate Earthquake Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol 124, No. 4.
Southern California Earthquake Center (SCEC), 1999, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, dated March.

LIQUEFACTION AND DYNAMIC SETTLEMENT WORKSHEET

JOB NO.: 402679001 **JOB NAME:** Oakmont/Evergreen
CALCULATION BY: PCC **DATE:** 5/13/2016
CHECKED BY: **DATE:**
BORING/PROFILE: B-2

Depth (z) of Layer Top	Bottom	Formation	Soil Type	Thick. t (ft.)	Midpoint of Layer (ft.)	Layer γ (pcf)	Total Stress σ_v (ksf)	Effect. Stress σ_v' (ksf)	$(N_1)_{60}$	Fines Content FC (%)	$(N_1)_{60cs}$ Note 1	r_d Note 2	CSR_M Note 3	Ks Note 5	$CRR_{7.5}$ σ_v'=1atm Note 6	CRR Note 7	FOS_{liq} Note 8	$CSR_{7.5}$ Note 9	$(N_1)_{60}$ + (N)corr Note 10	ε_v (%) Fig. 4-3	Sat. Sand Settlemnt ΔH_{sat} (in) Note 11	τ_{avg} (tsf)	mean effective stress σ_m' (tsf)	max Shear Modulus G_{max} (tsf)	a	b	γ (%)	ε_{15} (%)	ε_{Nc} (%)	dry sand settlement ρ_e (in.)
0	8	Alluvium	CL	8	4	120	0.48	0.48	9	70	15	0.995	0.362	1.100	0.153	0.197		0.309	13	2.2		0.09	0.16	442						
8	16	Alluvium	CL	8	12	120	1.44	1.44	23	70	29	0.964	0.351	1.073	0.408	0.514		0.300	27	0.0		0.25	0.47	976						
16	18	Alluvium	CL	2	17	120	2.04	2.04	20	70	26	0.941	0.343	1.006	0.304	0.359		0.292	24	1.2		0.35	0.66	1117						
18	20	Alluvium	SC	2	19	120	2.28	2.28	16	30	21	0.931	0.339	0.990	0.224	0.259		0.289	18	1.7		0.39	0.74	1073	0.152677	7685	0.107161	0.121603	0.10168	0.05
20	23	Alluvium	SC	3	21.5	120	2.58	2.49	16	30	21	0.919	0.347	0.978	0.224	0.256	0.74	0.296	18	1.7	0.60	0.43	0.80	1121						
23	26.5	Alluvium	CL	3.5	24.75	120	2.97	2.67	12	70	18	0.902	0.365	0.972	0.179	0.204		0.311	16	1.8		0.49	0.86	1118						

Total Settlement = 0.60 0.05

20 = d_w, depth to groundwater table (ft)
6.9 = M, moment magnitude of design earthquake
0.56 = a_{max}, peak horizontal ground acceleration for design earthquake (g)

NOTES:

1 clean sand blowcounts @ 1 tsf overburden @ 60% energy ratio, $(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$, where $\Delta(N_1)_{60}$ = exp[1.63+9.7/(FC+0.01)-(15.7/(FC+0.01))^2]
2 stress reduction factor, r_d = exp[-1.012-1.126*sin(z/3.3/11.73+5.133)+M*(0.106+0.118*sin(z/3.3/11.28+5.142))]
3 cyclic stress ratio @ M, $CSR_M = \tau_{avg}/\sigma_v' = 0.65(a_{max}/g)(\sigma_v)(r_d)/(\sigma_v')$
4 magnitude scaling factor, MSF = min[1.8,6.9*exp(-M/4)-0.058] ==> MSF = 1.17
5 overburden correction factor, Ks = min[1.1,1-(1/(18.9-2.55*sqrt(min[(N1)60cs,37])))*ln[sv'/2.12]]
6 cyclic resistance ratio @ M=7.5 sv'=1atm, $CRR_{7.5,1}$ = 2 if (N1)60cs < 37.5, else exp[(N1)60cs/14.1 + ((N1)60cs/126)^2 - ((N1)60cs/23.6)^3 + ((N1)60cs/25.4)^4 - 2.8]
6 cyclic resistance ratio @ M,sv', CRR =min[2,CRR7.5,1atm*Ks*MSF]
8 factor of safety against liquefaction, FOS_{liq} = (CRR/CSR_M)
9 cyclic stress ratio @ M=7.5, $CSR_{7.5} = CSR_M$/MSF
10 N-value correction for Fines Content for Settlement Analysis (Table 7.2, SCEC, 1999)
11 settlement of saturated sand, $\Delta H_{sat} = \varepsilon_v * t$
12 Coefficient of Lateral Earth Pressure at Rest, Ko: Ko = 0.47
13 Number of Strain Cycles, Nc: Nc = 10.0787

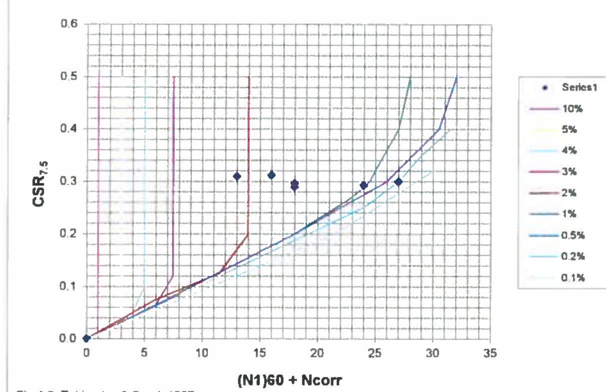


Fig 4-3: Tokimatsu & Seed, 1987

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Tokimatsu, K. & Seed, H.B., 1987, Evaluation of Settlements in Sand Due to Earthquake Shaking, Journal of Geotechnical Engineering Division, ASCE, Vol 113, No 8.
Pradel, D.J., 1998, Procedure to Evaluate Earthquake Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol 124, No. 4.
Southern California Earthquake Center (SCEC), 1999, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, dated March.

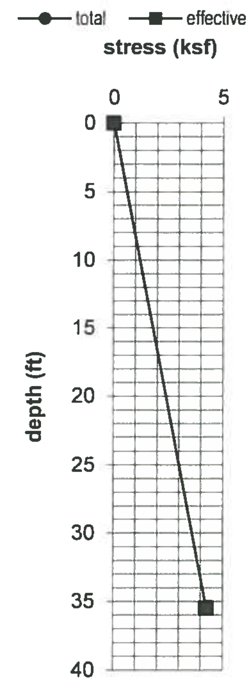
Project Name: Oakmont/Evergreen
Calculation By: PCC
Checked By:

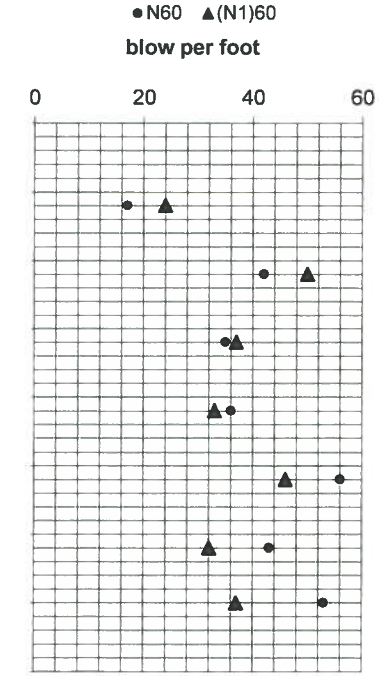
Project No.: 402679001
Date: 5/13/16
Date:

Profile/Boring: B-1

Depth to Groundwater (ft): 50

Formation	USCS Class.	depth to top of layer (ft)	depth to base of layer (ft)	layer thickness (ft)	Wet Unit Weight (pcf)	tot. vert. stress at top (psf)	tot. vert. stress at base (psf)
		0	35.5	35.5	120	0	4260





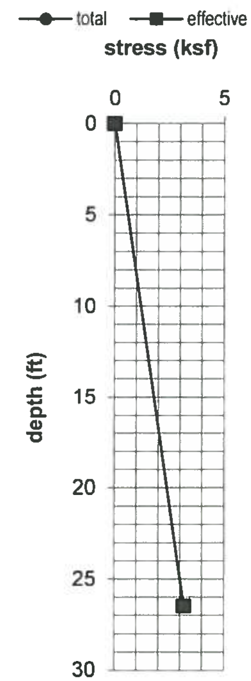
hammer type/method: wireline hammer at surface
borehole diameter (in): 4

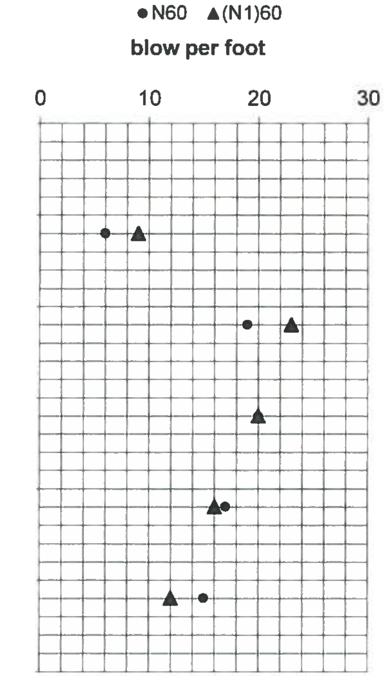
energy ratio correction, Cer: 1.00
borehole diameter correction, Cb: 1.00

sample depth (ft)	tot. vert. stress at depth (psf)	eff. vert. stress at depth (psf)	blowcnt N, (bpf)	cohesive? yes=1	sampler type?	equiv. SPT blowcnt Nspt, (bpf)	drill rod length	drill rod length correction Cr	N60 (bpf)	overbrdn correction Cn, (bpf)	N1 (bpf)	(N1)60
6	720	720	42	0	1	22	10	0.75	17	1.43	32	24
11	1320	1320	69	1	1	49	15	0.85	42	1.21	59	50
16	1920	1920	52	1	1	37	20	0.95	35	1.04	38	37
21	2520	2520	72	0	1	38	25	0.95	36	0.92	35	33
26	3120	3120	59	0	0	59	30	0.95	56	0.82	49	46
31	3720	3720	43	0	0	43	35	1.00	43	0.74	32	32
35	4200	4200	53	0	0	53	35	1.00	53	0.69	37	37

Project Name: Oakmont/Evergreen
Calculation By: PCC
Checked By:

Project No.: 402679001
Date: 5/13/16
Date:





Profile/Boring: B-2

Depth to Groundwater (ft): 50

Formation	USCS Class.	depth to top of layer (ft)	depth to base of layer (ft)	layer thickness (ft)	Wet Unit Weight (pcf)	tot. vert. stress at top (psf)	tot. vert. stress at base (psf)
		0	26.5	26.5	120	0	3180

hammer type/method: wireline hammer at surface
borehole diameter (in): 4

energy ratio correction, Cer: 1.00
borehole diameter correction, Cb: 1.00

sample depth (ft)	tot. vert. stress at depth (psf)	eff. vert. stress at depth (psf)	blowcnt N, (bpf)	cohesive? yes=1	sampler type?	equiv. SPT blowcnt Nspt, (bpf)	drill rod length	drill rod length correction Cr	N60 (bpf)	overbrdn correction Cn, (bpf)	N1 (bpf)	(N1)60
6	720	720	12	1	1	8	10	0.75	6	1.43	12	9
11	1320	1320	31	1	1	22	15	0.85	19	1.21	26	23
16	1920	1920	29	1	1	21	20	0.95	20	1.04	21	20
21	2520	2520	34	0	1	18	25	0.95	17	0.92	16	16
26	3120	3120	22	1	1	16	30	0.95	15	0.82	13	12

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