Ninyo & Moore
Geotechnical and Environmental Sciences Consultants

GEOTECHNICAL EVALUATION
BAY ROAD IMPROVEMENTS PHASE 2 &3
EAST PALO ALTO, CALIFORNIA

PREPARED FOR:

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September 11, 2014
Project No. 402371001

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Mr. Brian Krcelic
T.Y. Lin International
2010 Crow Canyon Place, Suite 350
San Ramon, California 94583

Subject: Geotechnical Evaluation
Bay Road Improvements Phase 2 & 3
East Palo Alto, California

Dear Mr. Krcelic:

In accordance with your authorization, Ninyo & Moore is pleased to submit this geotechnical evaluation report for the proposed Bay Road Improvements Phase 2 & 3 project in East Palo Alto, California. This report presents our findings, conclusions, and geotechnical recommendations for the project.

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

REGISTERED PROFESSIONAL ENGINEER
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Sincerely,
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TABLE OF CONTENTS

Page

Tables

Figures

Appendices

1. INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed Bay Road Improvements Phase 2 & 3 project in East Palo Alto, California. The project area consists of Bay Road between Clarke Avenue and Cooley Landing, Pulgas Avenue between Bay Road and Runnymede Street, and Runnymede Street between Pulgas Avenue and the Baylands Nature Preserve (Figure 1). The purpose of this study was to conduct a geotechnical evaluation in the areas of the proposed improvements to evaluate the subsurface soil conditions and to provide geotechnical recommendations pertaining to the design and construction of the planned improvements. This report presents our geotechnical findings, conclusions, and recommendations regarding this project.

2. SCOPE OF SERVICES

Our scope of geotechnical services included:

- Review of readily available background materials, including topographic maps, geologic data and maps, and fault and seismic hazard maps.
- Geotechnical site reconnaissance to observe the surficial geologic conditions and to select and mark the boring locations for utility mark-out services.
- Coordination with Underground Service Alert (USA) to locate the underground utilities in the vicinity of the proposed borings. A private utility survey was also conducted to further evaluate subsurface hazards.
- Obtain a boring permit from the San Mateo County Environmental Health Services Division.
- Performance of a subsurface exploration program consisting of drilling, sampling, and logging of five small-diameter, hollow stem auger borings up to a depth of approximately 20 feet below the ground surface. The borings were logged by a representative of Ninyo & Moore, and bulk and relatively undisturbed soil samples were collected at selected intervals for laboratory testing.
- Soil cuttings from the subsurface exploration were collected in drums and disposed in a landfill accepting non-hazardous waste.

- Performing geotechnical laboratory testing on selected samples including evaluation of in-situ moisture content and dry density, percentage of particles finer than the No. 200 sieve, gradation, Atterberg limits , soil corrosivity, and R-value.

- Data compilation and geotechnical analysis of the field and laboratory data. Our services included analyses to evaluate and provide recommendations pertaining to the following:

  - Subsurface conditions at the site including stratigraphy and depth to groundwater.

  - Evaluation of the seismicity, liquefaction potential, and secondary seismic hazards at the site.

  - Suitability of the proposed construction from a geotechnical standpoint.

  - Excavation characteristics and excavation stability.

  - Earthwork and compaction requirements, including subgrade preparation, and suitability of the on-site soils for use as fill material.

  - Evaluation of the corrosion potential of site soils.

  - Preliminary evaluation of new structural pavement sections.

- Preparation of this report presenting the results of our site reconnaissance, subsurface exploration, laboratory testing, and engineering analyses, as well as our geotechnical recommendations for design and construction of the proposed improvements.

3. SITE DESCRIPTION

The project area consists of Bay Road between Clarke Avenue and Cooley Landing, Pulgas Avenue between Bay Road and Runnymede Street, and Runnymede Street between Pulgas Avenue and the Baylands Nature Preserve in East Palo Alto, California (Figure 1).

The selected streets are asphalt-paved**.** Drainage is controlled by gutters and storm drains**.** The selected portion of Bay Road is a two-way street, and contains two segments with differing widths. The segment of Bay Road between Clarke Avenue and Pulgas Avenue is approximately 80 feet wide, with two lanes for travel in each direction, a shoulder for parking in each direction, and a median for portions of the segment. The segment of Bay Road between Pulgas Avenue and Cooley Landing is approximately 25 feet wide, with one lane for travel and a shoulder for parking in each direction. Ground elevations along Bay Road between Clarke Avenue and Cooley landing range from approximately 9 to 18 feet above MSL (Google Earth, 2014). The selected portion of Pulgas Avenue is a two-way street, approximately 35 feet wide, with one lane for travel and a shoulder for parking in each direction. Ground elevations range from approximately 10 to 11 feet above MSL (Google Earth, 2014). The selected portion of Runnymede is a two-way street, approximately 30 to 40 feet wide, with one lane for travel and a shoulder for parking in each direction. Ground elevations range from approximately 7 to 10 feet above MSL (Google Earth, 2014).

4. PROPOSED IMPROVEMENTS

We understand that the project will consists of the reconstruction of the entire section of the Bay Road from Clark Avenue to Cooley Landing including new sidewalks, bicycle lanes, and parking lanes, and associated streetscape areas. We further understand that the proposed improvements will also include installing and/or enhancing existing storm drainage system, and undergrounding of overhead utilities along the Bay Road, Pulgas Avenue, and Runnymede Street.

5. FIELD EXPLORATION AND LABORATORY TESTING

Our field exploration included a site reconnaissance and a subsurface exploration conducted on July 23, 2014. The subsurface exploration consisted of drilling, logging, and sampling five (5) hollow-stem auger exploratory borings. The hollow-stem auger borings were advanced to depths of approximately 20 feet below ground surface using a truck-mounted drill rig. The approximate locations of the borings are presented on Figure 2. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected representative soil samples for laboratory testing. Descriptions of the subsurface materials encountered are presented in the following sections. Detailed boring logs are presented in Appendix A.

Laboratory testing of representative soil samples was performed to evaluate in-situ moisture content and dry density, percentage of particles finer than the No. 200 sieve, gradation analysis, Atterberg limits, soil corrosivity, and R-value. The results of our in-situ moisture content and dry density evaluation are presented on the boring logs in Appendix A. The remaining laboratory testing results are presented in Appendix B.

6. GEOLOGY AND SUBSURFACE CONDITIONS

The following sections provide information regarding the geologic conditions relative to the project site.

6.1. Regional Geologic Setting

The project site is located on the western side of the San Francisco Bay in the Coast Ranges Geomorphic Province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys stretching approximately 600 miles from the Oregon border to the Santa Ynez River. They are formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, uplifted and metamorphosed, 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 ranges that trend northwest-southeast, due to lateral and vertical movement on the San Andreas fault system and associated splays.

6.2. Site Geology

Regional geologic mapping indicates that the project area is underlain by near-surface artificial fill, Holocene-age basin deposits, and Holocene-age flood plain (Bay Mud) deposits. The artificial fill material consists of variable material and within the project area it is mapped over and/or adjacent to the basin deposits and bay mud. The Holocene-age basin deposits are described as very fine silty clay to clay deposits occupying flat floored basins at the distal edge of alluvial fans adjacent to the bay mud. The Holocene-age flood plain deposits are described as medium to dark gray, dense, sandy to silty clay which usually occurs between levee deposits and basin deposits (Brabb et al., 2000). The findings of our subsurface exploration, described below, indicate that the site is generally underlain by alluvium and includes areas of fill over the alluvium. A Regional Geologic Map is provided as Figure 3.

6.3. Subsurface Conditions

Based on our review of pertinent geologic maps and the results of our subsurface exploration, the site is generally underlain by alluvium and includes areas of fill over the alluvium. Generalized descriptions of the units encountered are provided in the subsequent sections. More detailed descriptions are presented on the boring logs in Appendix A.

6.3.1. Pavement Section

The pavement sections encountered in our borings on Bay Road consisted of asphalt concrete (AC), about 3 to 7½ inches thick, over an aggregate base (AB) section that was about 1 to 4½ inches thick. AB section was not encountered in Boring B-5. The pavement sections encountered in our boring on Pulgas Avenue consisted of about 4 inches of AC over about 3 inches of AB. The pavement section encountered in our boring on Runnymede Street consisted of about 4 inches of AC over about 4 inches of AB.

Variations in thickness of AC and AB beyond the range observed may be encountered due to past maintenance, utility work or other factors.

6.3.2. Fill

Fill soils were encountered in Borings B-1, B-2, and B-5. The fill was approximately 2 to 3 feet deep. No fill was encountered in Borings B-3 and B-4. The encountered fill materials generally consisted of moist, loose to dense clayey sand, and dense silty sand.

6.3.3. Alluvium

Alluvial soils were encountered in each of the borings to the explored depth of up to approximately 20 feet. The alluvium generally consisted of moist to wet, soft to very stiff clay and sandy silty clay, very loose to loose sandy silt, very loose to loose silty sand

and clayey sand, loose poorly graded sand with silt, and very loose to medium dense poorly graded sand with gravel.

6.3.4. Groundwater

Groundwater was encountered in our borings during drilling between depths ranging from about 4 to 13½ feet but predominantly between 4 and 6½ feet below the existing grade. Historic high groundwater is reportedly 5 to 10 feet below the ground surface (California Geological Survey [CGS], 2006a). The depth to groundwater within the limits of the study area is subject to spatial variations in topography and the elevation of the phreatic surface. Groundwater may rise to a higher elevation than was encountered in our exploratory borings due to the short time available for seepage of water into the borings. Furthermore, groundwater levels may fluctuate in response to seasonal variations in precipitation, tidal influences, groundwater pumping/dewatering nearby, changes in irrigation practices adjacent to or within the study area, or other factors.

7. GEOLOGIC HAZARDS AND GEOTECHNICAL CONSIDERATIONS

This study considered a number of potential issues relevant to the proposed improvements, including seismic hazards, landsliding, expansive soils, flood hazards, corrosive soil excavatability, and material suitability. These issues are discussed in the following subsections.

7.1. Seismic Hazards

The site is located in a seismically active region. The seismic hazards considered in this study include the potential for surface ground rupture, ground shaking due to seismic activity, seismically induced liquefaction, dynamic settlement, lateral spreading, seismic slope stability, tsunamis, and seiches. These potential hazards are discussed in the following subsections.

7.1.1. Historical Seismicity and Ground Failures

The site is located in a seismically active region, as is the majority of northern California. Table 1 lists selected principal known active faults that may affect the subject site and the maximum moment magnitude (M_{max}) as published by Cao, et al. (2003) for the California Geological Survey (CGS). The approximate fault-to-site distances were evaluated using the seismic data published by the United States Geological Survey (USGS).

Table 1 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude [1] (M_{max})
Monte Vista- Shannon	6.1 (9.9)	6.7
San Andreas	8.0 (12.9)	7.9
Hayward	11.1 (17.8)	7.3
Calaveras	16.0 (25.8)	6.9
San Gregorio	17.9 (28.8)	7.4

Notes:
[1] Cao, et al., 2003

7.1.2. Faulting and Ground Surface Rupture

The numerous faults in northern California include active, potentially active, and inactive faults. As defined by the CGS, active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during the Quaternary time (approximately the last 1,600,000 years) but for which evidence of Holocene movement has not been established. Inactive faults do not show evidence of movement within the Quaternary time.

The subject site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone) (California Division of Mines and Geology [CDMG], 1974). However, the site is located in a seismically active area and the potential for strong ground motion in the project area is considered significant during the design life of the proposed improvements. Figure 4 shows the approximate site location relative to the major faults in the region. The active Monte Vista – Shannon fault is located approximately 6.1 miles southwest of the site; however, the San Andreas fault is considered to be the predominant fault and is located approximately 7.8 miles west of the site. Major known active faults in the region consist generally of en-echelon, northwest-striking, right-lateral, strike-slip faults. These include the Hayward and Calaveras faults, located east of the site, and the Monte Vista – Shannon, San Andreas, and San Gregorio faults, located west of the site.

Based on our review of the referenced geologic maps, it is our opinion that the site is not underlain by known active or potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 11,000 years and 1,600,000 years, respectively). Therefore, the potential for ground surface rupture due to faulting at the site is considered low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

7.1.3. Seismic Ground Motion

The 2013 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the MCE_R for the site was calculated as 0.6g using the United States Geological Survey (USGS, 2013) seismic design tool (web-based).

The 2013 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCE_G peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.52g using the USGS

(USGS, 2013) seismic design tool that yielded a mapped MCE_G peak ground acceleration of 0.52g for the site and a site coefficient (F_{PGA}) of 1.00 for Site Class D.

7.1.4. 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. Based on our review of the State of California Seismic Hazards Zones map (CGS, 2006b), the subject site is located in a mapped liquefaction hazard zone.

We encountered deposits of loose to medium dense granular materials below the high ground water (about 4 feet below the existing grade) during our subsurface exploration. We evaluated the liquefaction susceptibility of these deposits in accordance with the method presented by Youd et al. (2001) using the blow count data collected during our subsurface exploration and considering a seismic event producing a PGA of 0.52g resulting from a Magnitude 7.9 earthquake (based upon our deaggregation analysis of the design PGA). The results of our analysis indicate that the relatively granular materials below the groundwater table may liquefy during a significant seismic event.

Consequently, differential settlement of the pavements, and/or underground utilities from liquefaction, sand-boils, and dynamic settlement should be anticipated following a significant seismic event. Buried vaults or pipelines in this material may be uplifted during liquefaction if the weight of the vault/pipe and its contents combined with the weight of cover and frictional resistance above the groundwater table are not sufficient to balance the buoyant forces. Adequate compaction of pipeline bedding and trench backfill associated with cut and cover installation techniques should reduce the potential for liquefaction around pipes and subsequent uplift. Similarly if the vault is installed in an oversize hole that is filled in with suitably compacted material, uplift concerns related to liquefaction might be reduced. Recommendations for backfill compaction are presented in Section 9.1.8. Pipeline installed by methods other than cut and cover such as jack-and-bore or pipe-bursting, might be more susceptible to uplift due to liquefaction. Dynamic settlement of liquefiable soils is addressed in the following section.

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.

7.1.5. 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 evaluation, we encountered very loose to medium dense granular soil below groundwater. We evaluated the potential for dynamic settlement on site in accordance with the method presented by Tokimatsu and Seed (1987) for saturated sand and the method presented by Pradel (1998) for dry sand using the blowcount data collected during our subsurface exploration and considering a magnitude 7.9 earthquake producing a PGA of 0.52g. The results of our analysis indicate that the proposed improvements may undergo dynamic settlement on the order of 2½ inch (total) with a differential of about 1 inch over approximately 12-foot lateral distance.

Selection of a flexible pipeline system and designing for an increased hydraulic gradient might reduce the potential consequences of liquefaction and differential dynamic settlement. Ground improvement techniques such as vibro-flotation or deep-soil mixing could reduce the potential for liquefaction and the magnitude of the anticipated dynamic settlement. However, we anticipate that post-earthquake repair including remedial grading and paving might be a more cost-effective strategy, and should be anticipated for the project improvements.

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. Consequently, we do not regard lateral spreading as a design consideration.

7.1.7. Tsunamis and Seiches

Tsunamis are long wavelength seismic sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. The project location is not within a tsunami evacuation area as shown on the Tsunami Evacuation Planning Map for San Francisco and San Mateo Counties presented by the Association of Bay Area Governments (ABAG, 2009a). Seiches are waves generated in a large enclosed body of water. Based on the location of the site and its proximity to enclosed bodies of water nearby, the potential for damage due to tsunamis or seiches is a design consideration.

7.1.8. Seismic Slope Stability

Based on our review of State of California Seismic Hazard Zone Map (CGS, 2006b), the site is not located within an earthquake-induced landslide hazard zone (Figure 5). We do not regard seismic slope stability as a design consideration. Landsliding and slope stability of the site are further addressed in Section 7.3.

7.2. Flood Hazards and Dam Inundation

Our review of The Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FEMA, 2012) found that the portions of the Pulgas Avenue and the Runnymede

Street are in the areas having 0.2% annual chance of flood. Remainder of the site is in an area considered to be outside the 0.2% annual chance of flood. ABAG Flood Hazard Area maps (ABAG, 2009) indicate that the portions of the project along Runnymede Street are located in a 100-year flood zone area. Remainder of the project area is considered to be an ururbanized area. Based on review of the Dam Failure Inundation Areas prepared by ABAG (ABAG, 1995), the site is not located within an inundation area following a conjectured catastrophic dam failure.

7.3. Landsliding and Slope Stability

The project site and surrounding area is relatively flat. As such, we do not regard landsliding or slope stability as a design consideration.

7.4. Static Settlement

We anticipate that the loads associated with the new pipelines and related vaults will be balanced by the weight of the displaced soil such that the net load will be negligible. As such, we do not regard post-construction settlement due to static loads as a design consideration.

7.5. Expansive Soils

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soils containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. We do not regard expansive soils as a design consideration given the granular nature of many of the subsurface materials encountered, and the generally low plasticity index of cohesive soils within the study area.

7.6. Corrosion

An evaluation of the corrosivity of the on-site soil was conducted to assess the effect on concrete and metals. The corrosion potential was evaluated using the results of limited laboratory testing on samples obtained during our subsurface evaluation. Laboratory testing to evaluate pH, resistivity, chloride content, and soluble sulfate content was performed on a sample of the near-surface material. The results of the corrosivity tests are presented in Appendix B.

Caltrans defines a corrosive environment as an area where the soil contains more than 500 parts per million (ppm) of chlorides, sulfates of 0.2 percent (2,000 ppm) or more, and pH of 5.5 or less (Caltrans, 2012). The criteria used to evaluate the deleterious nature of soil on concrete are listed in Table 2. Based on these criteria, the sample of material tested does not meet the definition of a corrosive environment; however, the site is located within 1,000 feet of potentially brackish water (Figure 1). The sulfate exposure to concrete is negligible and ferrous metals will still undergo corrosion on site, but special mitigation measures are not needed. Recommended corrosion mitigation measures for reinforced concrete are presented in Section 9.3 .

Table 2 – Criteria for Deleterious Soils

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, 2012)

7.7. Material Suitability

In general, based on the findings from our subsurface exploration, we anticipate that the soil materials excavated for the new underground utility alignments will be suitable, from a geotechnical perspective, for re-use in some capacity as trench backfill, provided that the materials are not too wet to inhibit compaction or get mixed with deleterious or otherwise unsuitable material. Trench spoils, particularly from below the groundwater table, may be too wet to compact when excavated. The spoils may need to be spread out to dry before being reused as backfill.

7.8. Excavatability

We anticipate that the proposed project will involve excavations of up to about 10 feet for installation of new pipelines. The geologic units encountered during our subsurface evaluation within this interval consisted primarily of soft to stiff clays and very loose to medium dense sands. We anticipate that backhoes, excavators, or other trenching equipment in good working condition should be able to make the proposed excavations.

7.9. Excavation Wall Stability

The geologic units encountered during our subsurface evaluation generally consisted of very soft to stiff clays, and very loose to dense sands. Our subsurface evaluation encountered a relatively shallow groundwater table. Cuts in these deposits or excavations below the groundwater table may not remain stable without appropriate inclination of side slopes or shoring. Precipitation on the trench sidewalls or surface runoff over the trench sidewalls may further adversely impact the stability of the excavation walls. Dewatering measures may be needed to provide a dry excavation in which to work. Temporary surface drainage improvements may also be advisable. Recommendations for dewatering and drainage improvements are presented in Sections 9.1.4 and 9.1.5.

Techniques for trench shoring may consist of movable trench boxes or shields, sheeting and hydraulic or mechanical jacks, or driven sheet piling. Appropriately-sized trench boxes or shields should protect workers but may allow movement of the excavation wall which will result in subsidence at the ground surface. Tight sheeting (without gaps) with appropriately-sized hydraulic or mechanical jacks to provide positive pressure against the face of the excavation should reduce the horizontal deflection of the sidewall and resulting subsidence at the ground surface. Driven sheet piles may be needed to support the excavation sidewall if the unsupported wall cannot remain stable long enough to install trench shields or sheeting. Recommendations for excavation stabilization are presented in Section 9.1.3.

7.10. Excavation Bottom Stability

In general, we anticipate that the bottom of the pipeline trenches will remain stable and provide suitable support for the proposed conduits. However, excavations that extend near or below the water table may experience "quick" conditions or bottom instability. Unstable bottom conditions may warrant overexcavation and replacement with crushed, angular rock. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

7.11. Uplift Considerations

The groundwater table is relatively shallow with respect to the proposed depth for some of the new alignments. We anticipate that the overburden pressures at the proposed depths of the new pipeline alignments will balance the buoyancy-related uplift forces due to submergence. Manholes and access vaults below the groundwater table, however, might be impacted by the uplift forces. Recommendations for design parameters to resist buoyancy-related uplift forces are presented in Section 9.2.

8. CONCLUSIONS

Based on the results of our geotechnical evaluation, the proposed project is feasible from a geotechnical perspective provided the recommendations presented in this report are incorporated into the design and construction of the project. In general, the following conclusions were made based on our evaluation:

- Our subsurface exploration encountered a relatively shallow groundwater table. We anticipate that the proposed excavations will encounter groundwater and some of the pipes and vaults will be partially or wholly submerged. Recommendations for dewatering of excavations and parameters for modeling the uplift resistance of submerged vaults are presented in Sections 9.1.4 and 9.2, respectively.
- We anticipate that excavations may need to be shored or sloped appropriately to remain stable. Recommendations for excavation stabilization are presented in Section 9.1.3. Trench bottoms may be unstable due to the shallow groundwater table.
- The site could experience a relatively large degree of ground shaking due to a significant earthquake event on the nearby San Andreas fault.

- Our subsurface exploration encountered deposits of very loose to medium dense granular materials that may liquefy following a significant earthquake. We estimate that dynamic settlement of up to about 2½ inches may occur along the proposed alignments.

9. RECOMMENDATIONS

The following guidelines should be used in the preparation of the construction plans. The geotechnical consultant should review the proposed plans prior to construction.

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 operations may result in new recommendations, which could supersede the recommendations in this section.

9.1.1. Abandonment of Existing Utilities

We anticipate that the existing underground pipelines will largely be removed and disposed of during construction phase of the project. If underground pipeline mains and laterals are to be abandoned in place, the pipes should be crushed, or plugged with pumpable grout or concrete mix. If plugged in place, the grout or concrete should be pumped from the high end of the pipe, with the low end plugged, until concrete completely fills the pipe. Excavations made to remove existing utilities should be backfilled in accordance with our recommendations in Section 9.1.8. Pavement and aggregate base debris generated during the abandonment should be removed from the project site and disposed of at a legal dumpsite. Excavated soil materials may be re-used as engineered fill provided that the materials comply with, or are processed to comply with, the recommendations in Section 9.1.6.

9.1.2. Observation and Removals

Prior to placement of bedding material, the client should request an evaluation of the exposed subgrade by the geotechnical consultant. Materials that are considered unsuitable shall be excavated under the observation of the geotechnical consultant in accordance with the recommendations in this section or the field recommendations of the geotechnical consultant.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soils, fractured, weathered, or soft bedrock, and undocumented or otherwise deleterious fill materials.

Unsuitable materials should be removed from the bottom of the trench to the depth of suitable material as evaluated by the geotechnical consultant in the field. Foundation material should be placed and compacted to get back to the design grade.

The subgrade of trenches excavated near or below the groundwater table may become unstable. Recommendations for stabilizing excavation bottoms should be based on an evaluation by the geotechnical consultant during construction. Stabilization recommendations might include installing additional sumps or well points, or overexcavating to install a drainage blanket or place suitable foundation material.

9.1.3. Excavation Stabilization and Temporary Slopes

Excavations, including trench excavations, shall be stabilized in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations, Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA, 1989). Stabilization shall consist of shoring sidewalls or laying slopes back. Table 3 lists the OSHA material type classifications and corresponding allowable temporary slope layback inclinations for soil deposits that may be encountered on site. Alternatively, a shoring system may be used to stabilize excavation sidewalls during construction. Potential shoring systems include trench boxes/shields, sheeting with hydraulic or mechanical jacks, cantilever sheet piles, or sheet piles with internal braces. Trench boxes/shields and sheeting may be used to stabilized excavations above groundwater. The lateral earth pressures listed in Table 3 may be used to design or select the trench box/shield or sheeting system in accordance with the criteria listed in the OSHA Excavation Rules and Regulations (29 CFR Part 1926).

The recommendations presented in this table are based on 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.

Table 3 – Recommended OSHA Material Classifications and Allowable Slopes

Formation	OSHA Classification	Allowable Temporary Slope[1,2,3]	Lateral Earth Pressure on Shoring[4], (psf)
Fill and Alluvium	Type C	1½h:1v (34°)	$80 \cdot D + 72$

1 Excavation sidewalls in cohesive soils 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 soils, no layer shall 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.

Sheet piles that extend below the mudline may be needed for excavations below the groundwater table to reduce the potential for “quick” conditions or bottom instability. We anticipate that an embedment depth equivalent to 125 percent of the head differential after dewatering may be needed to provide a suitable factor of safety against piping. The earth pressure diagrams presented in Figure 7 may be used to design a cantilevered sheet pile shoring system. The earth pressures listed in Figure 7 do not include a factor of safety. Once the depth of embedment and point of rotation are selected to meet shear and moment equilibrium at the tip of the sheet pile, the depth of embedment should be increased by 20 to 40 percent for an approximate factor of safety of 1.5 to 2.0 and checked against the embedment depth needed to resist piping. Alternatively, the sheet pile shoring may be supported by internal braces. The earth pressure diagrams presented in Figure 8 may be used to design an internally-braced, sheet pile shoring system. The designer should select an appropriate factor of safety to use with the earth pressure diagrams presented in Figure 8.

The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor with consideration for the tolerable settlement of ground adjacent to the excavation. Potential causes of settlement that should be addressed include loss of lateral support following excavation, vibration during the installation of sheet piles, other construction induced vibrations, dewatering, and removal of the support system. Shoring should be sufficiently tight to reduce washout from behind the shoring. 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.

We understand that the proposed excavations will not be in close proximity to existing structures. 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 adjacent structures at an angle of 1:1 (horizontal to vertical).

9.1.4. Construction Dewatering

Groundwater was encountered in our exploratory borings at a depth of about 4 to 13½ feet but predominantly between 4 and 6½ feet below the existing grade. However, significant fluctuations in the groundwater level may occur as a result of variations in seasonal precipitation and other factors. Water intrusion into the excavations may occur as a result of groundwater intrusion or surface runoff. The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations. 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.

When excavating near or below the groundwater table, the dewatering system should depress the water level below the bottom of the cut to reduce the potential for subgrade instability and washout from behind sheeting or sloughing of exposed trench walls. The dewatering system should maintain the water level about 2 feet below the pipe bedding and foundation material to provide a stable trench bottom. Sump pumps, well points, deep wells, geotextile-geonet composites, perforated underdrains, or stone blankets should be used, as appropriate, to drain water from below the bedding and foundation material. Perforated underdrains and open-graded stone blankets should be wrapped in a suitable geotextile filter to reduce the potential for the removal of fines and subsequent creation of voids in the overlying and adjacent materials. The operation of the dewatering system should continue during and after the installation of the pipe and embedment until sufficient backfill has been placed to balance the uplift forces.

9.1.5. Drainage

Temporary swales or cutoff barriers should be provided to divert surface water away from the excavation. Dams, cutoffs, or other barriers should be constructed on the bottom of the trench to reduce the velocity of subdrain discharge or runoff along the trench bottom thereby reducing the potential for erosion or undermining of trench walls, subgrade, bedding, or foundation materials.

9.1.6. Material Requirements

Materials used during earthwork, grading, and paving operations should comply with the requirements listed in Table 4. Materials should be evaluated by the geotechnical consultant 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.

Table 4 – Recommended Material Requirements

Material and Use	Source	Requirements[1,2]
Foundation below bedding material and pipe zone fill	import	90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve
	on-site borrow	No additional requirements[1]
Pipe Bedding Material and Pipe Zone Fill	import	As per manufacturer's recommendations; or well-graded sand or sand gravel mixture with 5 percent fines or less and nominal size 3/4" or less
	on-site borrow	No additional requirements[1]
Trench Backfill above pipe zone fill	import	Expansion Index less than 50; Free from rocks/lumps in excess of 2" diameter
	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

[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.

[4] Non-corrosive as defined by the Corrosion Guidelines version 1.0 (Caltrans, 2012).

[4] CSS is California Standard Specifications (Caltrans, 2010).

9.1.7. Subgrade Preparation

Subgrade should be prepared as per the recommendations in Table 5. 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 pavements, footings and slabs. Subgrade that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture-conditioned, and recompacted.

Table 5 – Subgrade Preparation Recommendations

Subgrade Location	Preparation Recommendations
Utility Trenches	o Check for and remove unsuitable materials as per Section 9.1.2. o Do not scarify. Compact as per Section 9.1.8 if disturbed. Remove or compact loose/soft material.
Below Fill and Pavements	o Check for unsuitable materials as per Section 9.1.2. o Scarify top 8 inches then moisture-condition and compact as per Section 9.1.8. Remove or compact loose/soft material.

9.1.8. Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts to meet the criteria listed in Table 6. 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. The thickness of the bedding material placed below the pipe should be 4 inches or more. Where unyielding materials are exposed in the trench bottom, or the pipe crosses other utilities or buried structures, the thickness of the bedding material should be increased to 6 inches. The pipe barrel should rest on the bedding material. Small holes should be excavated in the bedding material to provide room for the bells. The bedding and initial backfill should be carefully placed by hand under the pipe haunches and in the bell holes to avoid creating voids below the pipe. Trenches should be wide enough to provide room to operate compaction equipment and wide enough to provide lateral clearance between the pipe and trench wall equivalent to half the pipe diameter. If the native soils cannot sustain a vertical cut or if the trench walls are sloped back for stability, the lateral clearance should be increased to a distance equivalent to the pipe diameter.

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, bedding and pipe zone fill by flooding or jetting is not recommended. Loose material that has sloughed off of trench sidewalls during pipeline installation, backfill placement, or compaction should be removed before placing and compacting additional fill. Special care should be exercised to avoid damaging the pipe during compaction of the backfill. Before allowing vehicles or typical construction equipment to cross over pipe, 36 inches of embedment cover should be placed and compacted over the pipe. Hydro-hammers should not be used for compaction.

Table 6 – Recommended Compaction Requirements

Fill Type	Location	Recommended Compacted Density[1]	Recommended Compacted Moisture[2]
Bedding	Material below conduit invert	90 percent	At or near optimum
Pipe Zone Fill	Material above bedding to 12 inches above pipe	90 percent	At or near optimum
Trench Backfill	Within 3 feet from top of pavement	95 percent	At or near optimum
	In locations not already specified	90 percent	At or near optimum
Aggregate Base	Pavement sections	95 percent	At or near optimum
Asphalt Concrete	Pavement Section	95 percent	Not Applicable
General Fill	In locations not already specified	90 percent	At or above optimum

[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 California Test Method 304.

[2] Optimum moisture should be 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 the pavement section. 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.

9.1.9. Rainy Weather Considerations

We recommend that 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 through 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 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. The geotechnical engineer should be consulted for recommendations to stabilize the site as needed.

9.2. Uplift Resistance

Underground structures, including vaults or pipelines, that extend below the groundwater table will experience buoyancy-related uplift forces that might lead to upward movement. Groundwater was encountered in our exploratory borings predominantly at depths between approximately 4 and 6½ feet below the existing grade. However, groundwater levels may fluctuate with seasonal precipitation, tidal effects, or other factors. Underground structures below the groundwater table should be designed to resist uplift forces related to the buoyancy effect. Uplift forces may be resisted by the weight of the vault plus contents, the weight of soil above the vault, and friction along the sides of the vault. The unit weight of the soil may be considered to be 120 pounds per cubic foot (pcf) above the groundwater table and 62 pcf below the groundwater table. Frictional uplift resistance is the product of the friction coefficient and the effective contact pressure. A friction coefficient of 0.30 may be assumed for uplift resistance for mass or formed concrete against sand or clays. The effective contact pressure may be calculated using an equivalent fluid pressure of 60 pcf above the groundwater table and 32 pcf below the groundwater table. We do not anticipate that static uplift will be a design consideration for pipelines with embedment equivalent to twice the pipe diameter due to the magnitude of the overburden pressures.

Uplift resistance may be reduced during a seismic event if the material around the pipe or underground vault liquefies. Frictional uplift resistance should be neglected below the groundwater table to model the reduction in uplift resistance due to liquefaction. Pipelines installed by trenchless methods may be impacted by liquefaction and associated uplift. Liquefaction related uplift should not be a design consideration for pipelines installed in open trenches and backfilled with appropriately compacted material due to the relative density of the backfill material. Similarly, liquefaction related uplift should not be a design consideration for underground structures or vaults installed in an oversize excavation backfilled with appropriately compacted material.

9.3. 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 soils and the proximity of brackish water to 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 Concrete Institute (ACI) Committee 318 (ACI, 2012).

9.4. Preliminary Asphalt Concrete Pavement Design

Recommendations for pavement reconstruction are presented in the following sections.

9.4.1. Asphalt Concrete Pavement Reconstruction

Appropriate alternative pavement sections were evaluated for new construction or reconstruction of the existing pavements. Laboratory testing performed during our study on a sample of representative near-surface soil yielded an R-value of 6. The traffic index (TI) values for the paved areas have not been selected. For preliminary design purpurposes, structural pavement sections were evaluated using TI values of 7 through 10 based on our experience with similar pavements on other projects.

Our preliminary analysis was conducted to evaluate the asphalt pavement structural section following the methodology presented in Section 600 of the Highway Design Manual (Caltrans, 2012). The asphalt pavements were designed assuming a 20-year design life. It is assumed that periodic maintenance, including crack sealing and resurfacing, will be performed during the design life of the pavement. Premature deterioration may occur without periodic maintenance. Our preliminary recommendations for the pavement sections are presented in Table 7.

Table 7 – Preliminary Asphalt Concrete Pavement Structural Sections

Traffic Index	Alternative 1	Alternative 2
	HMA/AB (inches)	**Full Depth HMA (inches)**
7	6.0/12.0	10.5
8	8.0/12.0	12.0
9	9.0/12.0	14.0
10	11.0/12.0	15.5

Notes:
HMA – Hot Mix Asphalt
AB – Class II Aggregate Base

Subgrade soil in the areas to be paved should be prepared as recommended in Section 9.1.7 of this report. Concentrated runoff should not be allowed to 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.

9.4.2. Pavement Drainage

To improve drainage and reduce the potential for premature deterioration of pavement due to poor drainage, we recommend that edge subdrains be constructed where feasible. Edge subdrains should consist of slotted, stiff, plastic pipe encapsulated by ¾-inch, open-graded, crushed rock wrapped with filter fabric (Mirafi 140N, or equivalent) in a 12- inch wide trench. The edge subdrains should be constructed against the AB section of the pavement. The wrapped crushed rock should be capped by a relatively impervious layer such as a paved shoulder, concrete gutter, or 6 inches of compacted clay. The collector pipe should be 12 inches or more below the bottom of the pavement section. Alternatively, geocomposite drainage panels (Contech Stripdrain 100, or equivalent) may be placed vertically in a narrow trench (against the outside wall) backfilled with AB. Unslotted plastic outlet pipes, suitably sloped, should be provided at appropriate intervals to drain and appropriately dispose of accumulated water. Outlet pipe trenches should be backfilled with material of low permeability or include cut-off walls/diaphragms to reduce potential for piping. Vents and cleanouts should be provided at suitable intervals to promote free drainage and maintenance.

9.5. Pavement Restoration

We assume that the existing pavement on either side of the proposed underground utility pipeline will remain in place. For pavement patching over the proposed pipelines, we recommend that the new pavement section match the existing section on either side.

9.6. Instrumentation and Documentation

Consideration should be given to implementing documentation and instrumentation programs to evaluate design assumptions, existing conditions, and to monitor movements, levels and deformations during construction. The monitoring programs may include the use of seismographs, groundwater monitoring wells, inclinometers, convergence points and an array of surface control points. The resulting data should be reviewed and evaluated by the geotechnical consultant. These programs should be in-place or conducted prior to the start of construction.

9.6.1. Documentation of Existing Conditions

A pre-construction survey may be performed on residences and structures within approximately 50 feet of the proposed trench excavations. The surveys may include photo-documenting existing cracks, and measuring crack widths and vertical separations, if applicable. Consideration may be given to videotaping the survey. In addition, interviews with property owners may be conducted to provide knowledge of the age and type of the buildings as well as maintenance history and utility problems.

9.6.2. Lateral Movement of Shoring Support System

Inclinometers or survey points may be established behind excavations located in areas where structures are located above a 1:1 (horizontal to vertical) plane projected from the bottom of the proposed excavations. An evaluation of the final project plans may be performed to see if other structures or sensitive site improvements are located within close proximity of the proposed excavations. The inclinometers or survey points should be monitored and evaluated daily during excavation activities to provide an advanced warning system of potential problems.

9.7. Review of Construction Plans

The recommendations provided in this report are based on preliminary design information for the proposed construction. We recommend that the geotechnical consultant review project plans. 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.

9.8. Pre-Construction Conference

We recommend that a pre-construction conference be held. City representatives, the civil engineer, the geotechnical consultant, and the contractor should be in attendance to discuss the plans, the project, and the proposed construction schedule.

9.9. Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions disclosed by widely spaced exploratory borings. The geotechnical consultant in the field during construction should check the interpolated subsurface conditions. During construction, the geotechnical consultant should:

- Observe subgrade for stability and removal of unsuitable materials.
- Observe excavation shoring.
- Check and test imported materials prior to their use as fill.
- Observe trench backfill and compaction.
- Perform field density tests to evaluate trench backfill and aggregate base compaction.

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 due to 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, 2012, ACI Manual of Concrete Practice.

American Society for Testing and Materials (ASTM), 2013, Annual Book of ASTM Standards, West Conshohocken, Pennsylvania.

Association of Bay Area Governments (ABAG), 1995, Hazard Map, Dam Failure Inundation Areas; available online at http://www.abag.ca.gov.

Association of Bay Area Governments (ABAG), 2009a, Tsunami Evacuation Planning Map for San Francisco and San Mateo Counties; available online at http://quake.abag.ca.gov.

Association of Bay Area Governments (ABAG), 2009b, FEMA Flood Hazard Area Map; available online at http://quake.abag.ca.gov.

Brabb, E.E., Graymer, R.W., and Jones, D.J., 1998, Geology of the Onshore Part of San Mateo County, California, Derived from the Digital Database Open-File 98-137, U.S. Geological Survey.

Brabb, E.E., Graymer, R.W., and Jones, D.J., 2000, Geologic Map and Map Database of the Palo Alto 30' x 60' Quadrangle, California, U.S. Geological Survey, MF-2332, Scale 1:100,000.

Bray, J.D., and Sancio, R.B., 2006, Assessment of the Liquefaction Susceptibility of Fine-Grained Soils, Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers (ASCE), Vol. 132, No. 9, pp. 1165-1177.

California Building Standards Commission (CBSC), 2013, California Building Code (CBC): California Code of Regulations, Title 24, Part 2, Volumes 1 and 2.

California Division of Mines and Geology, 1974, State of California Special Studies Zones, Palo Alto Quadrangle: dated July 1: scale 1:24,000.

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

California Department of Transportation (Caltrans), 2006, California Test Methods (CTM), http://www.dot.ca.gov/hq/esc/ctms/index.html.

California Department of Transportation (Caltrans), 2012, Section 600 – Pavement Engineering, Division of Design, Manuals & Guidance: dated November.

California Department of Transportation (Caltrans), 2010, Standard Plans and Specifications.

California Geological Survey, 2006a, Seismic Hazard Zone Report for the Palo Alto 7.5-Minute Quadrangle, San Mateo and Santa Clara Counties, California, Seismic Hazard Zone Report 111.

California Geological Survey, 2006b, Seismic Hazard Zones Map for the Palo Alto Quadrangle: Scale 1:24,000.

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

Federal Emergency Management Agency (FEMA), 2012, Flood Insurance Rate Map, San Mateo County, California and Incorporated Areas, Panel 309 of 510, Map Number 06081C10309E, dated October 16.

Google Earth, 2014, Version 6.2

Jennings, C.W., 2014, and Bryant, W.A., 2010, Fault Activity Map: California Geological Survey, California Geologic Data Map Series, Map No. 6, Scale 1:750,000.

Kramer, S.L., 1996, Geotechnical Earthquake Engineering, Prentice Hall.

Ninyo & Moore, In-house Proprietary Information.

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.

Pradel, D., 1998, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, Volume 124, No. 4, dated April.

Tokimatsu, K., and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, 113(8), 861-878.

United States Geological Survey, 2013, United States Seismic Design Maps, World Wide Web, http://geohazards.usgs.gov/designmaps/us/application.php.

Witter, R.C., Knudsen, K.L, Sowers, J.M., Wentworth, C.M., Koehler, R.D., and Randolph, C.E., 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California: U.S. Geological Survey Open-File Report 2006-1037, Scale 1:24,000 (http://pubs.usgs.gov/of/2006/1037/)

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H., II., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering: American Society of Civil Engineering 124(10), p. 817-833.

REFERENCE: THOMAS GUIDE, 2008, METRO AREAS OF ALAMEDA, CONTRA COSTA, MARIN, SAN FRANCISCO, SAN MATEO, AND SANTA CLARA COUNTIES.

N

SCALE IN FEET

0 1,500 3,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

SITE BOUNDARY

Ninyo & Moore

PROJECT NO.	DATE	
402371001	9/14	

SITE LOCATION

BAY ROAD IMPROVEMENTS PHASE 2 AND 3
EAST PALO ALTO, CALIFORNIA

FIGURE

1

402371001-SL.dwg, Sep 09, 2014, 1:46pm, SN

REFERENCE: GOOGLE EARTH IMAGERY, 2014.

N

SCALE IN FEET

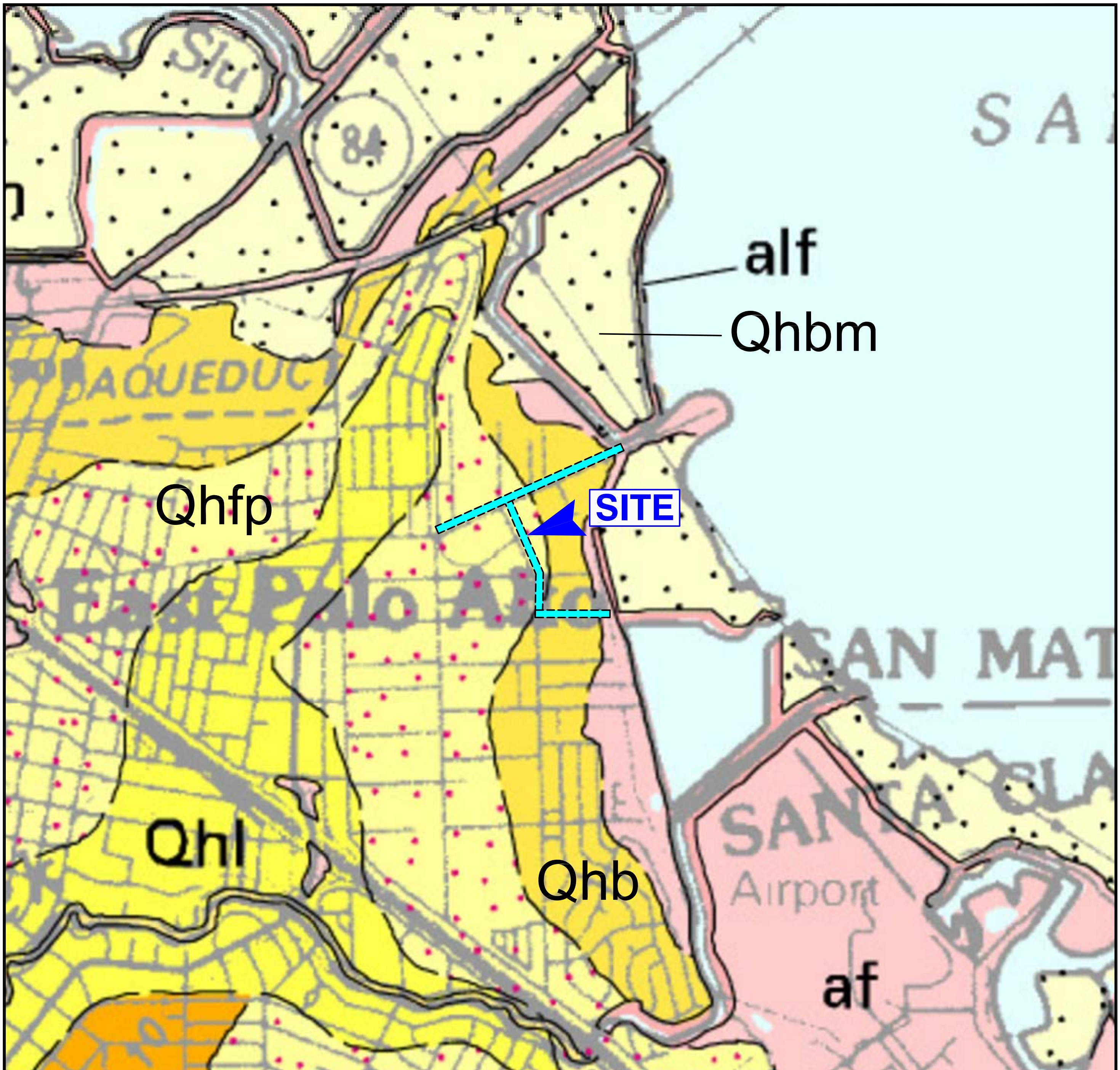
0 400 800

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND	
B-5	BORING
TD=20'	TD=TOTAL DEPTH IN FEET

Ninyo & Moore		BORING LOCATIONS	FIGURE
PROJECT NO.	DATE	BAY ROAD IMPROVEMENTS PHASE 2 AND 3 EAST PALO ALTO, CALIFORNIA	2
402371001	9/14		

402371001-BL.dwg, Sep 09, 2014, 1:30pm, SN



REFERENCE: E.E. BRABB, R.W. GRAYMER, D.L. JONES, 2000, GEOLOGIC MAP AND MAP DATABASE OF THE PALO ALTO 30'X60' QUADRANGLE, CALIFORNIA, MF-2332, SCALE 1:100,000.

N

SCALE IN FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

af	Artificial fill
alf	Artificial levee fill
Qhb	Basin deposits
Qhbm	Bay mud
Qhfp	Flood-plain deposits
Qhl	Natural levee deposits
- - - -	Geologic contact

402371001-RG.dwg, Sep 09, 2014, 1:47pm, SN

Ninyo & Moore

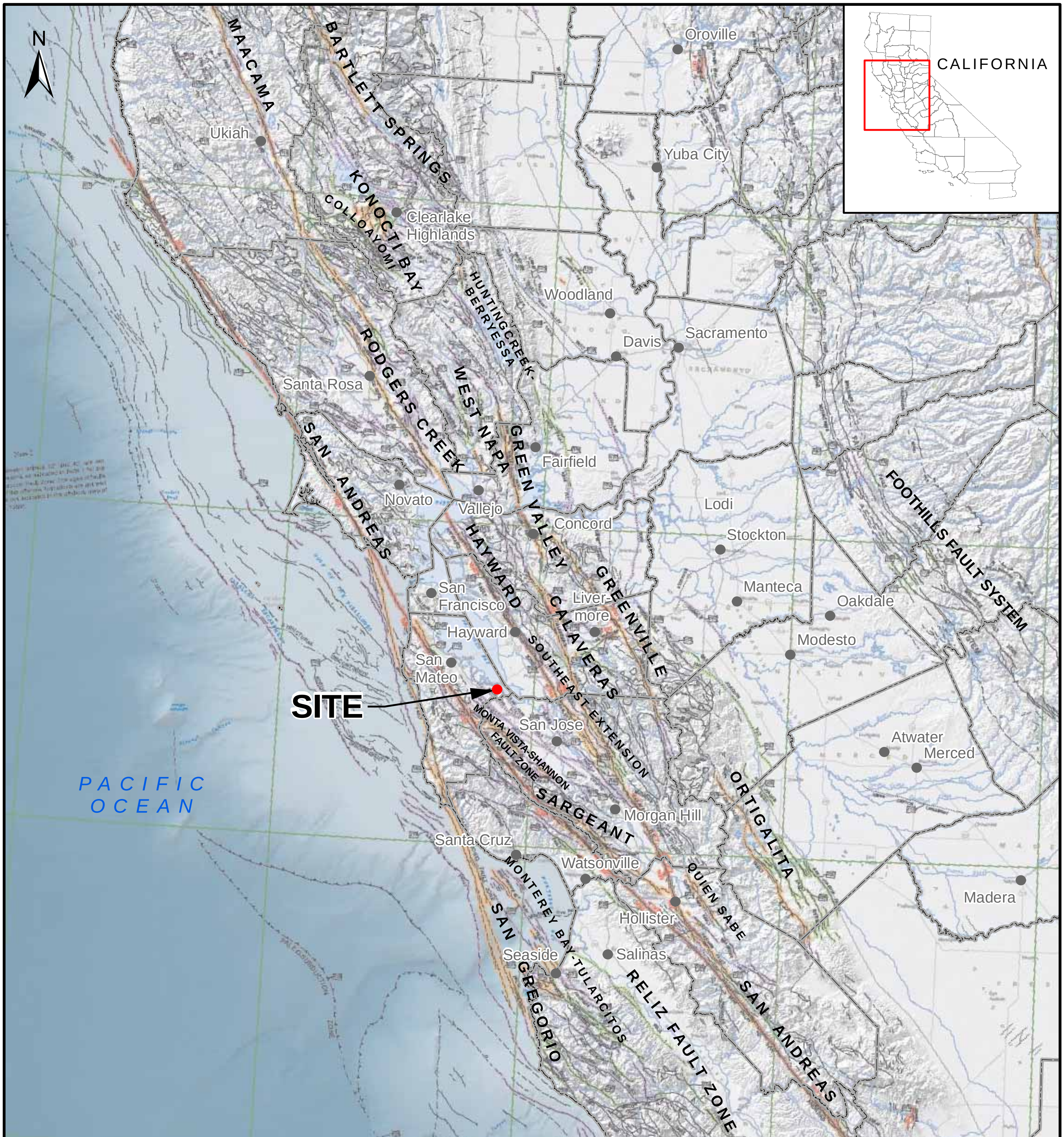
REGIONAL GEOLOGY

PROJECT NO.	DATE
402371001	9/14

BAY ROAD IMPROVEMENTS PHASE 2 AND 3
EAST PALO ALTO, CALIFORNIA

FIGURE

3



SOURCE: Jennings, C.W., and Bryant, W.A., 2010, Fault Activity Map of California, California Geological Survey, California Geologic Data Series, Map No.6, Scale 1:750,000.

APPROXIMATE SCALE

0 30 60 MILES

LEGEND

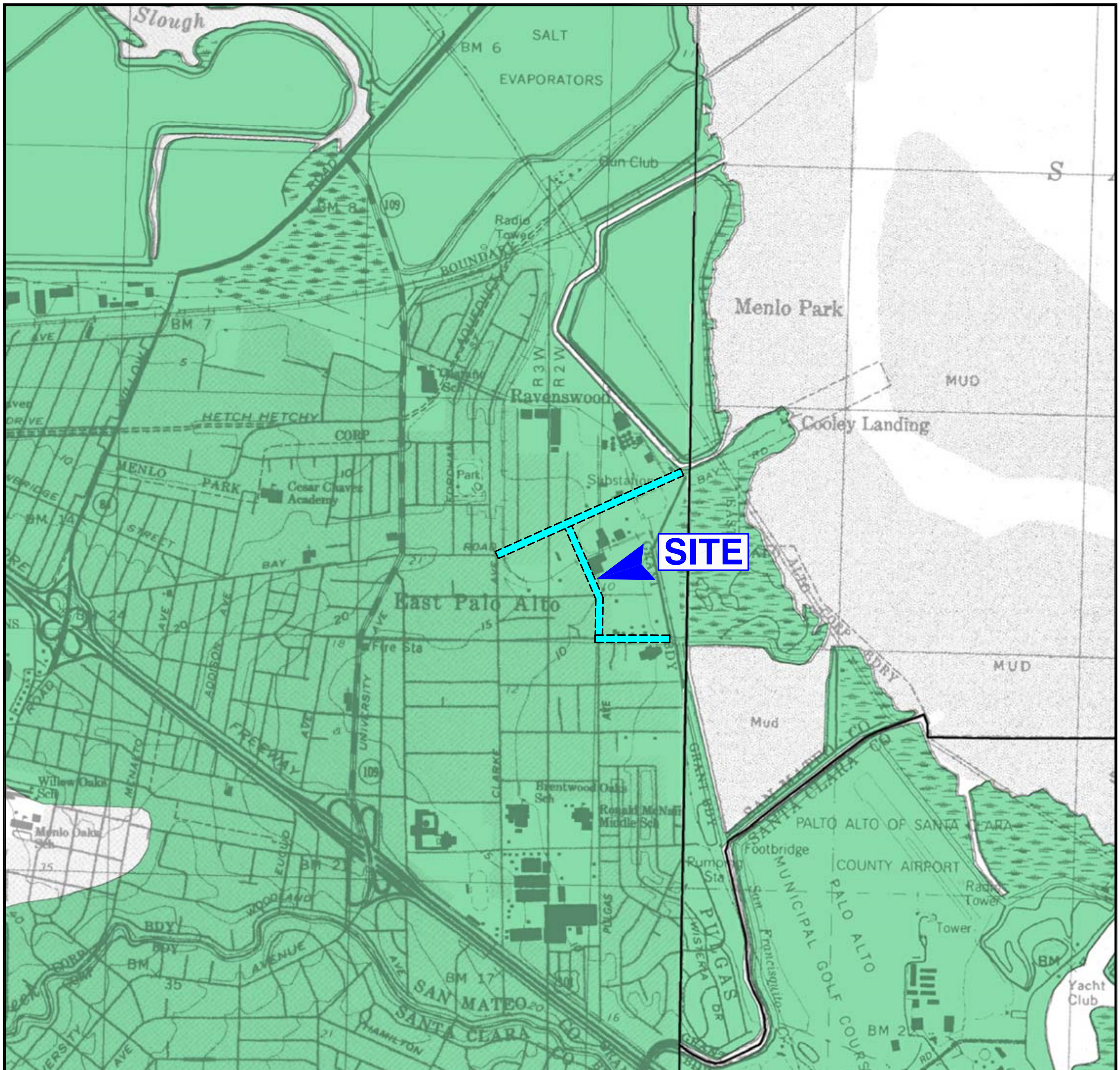
CALIFORNIA FAULT ACTIVITY

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

NOTES: ALL DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE

Ninyo & Moore	FAULT LOCATIONS	FIGURE
PROJECT NO. / 402371001	DATE / 9/14	BAY ROAD IMPROVEMENTS PHASE 2 AND 3 EAST PALO ALTO, CALIFORNIA	4

C:\GIS\DATA\fault_loc_2010_OAK\fault_loc_2010_OAK_92.mxd



REFERENCE: CALIFORNIA GEOLOGICAL SURVEY, 2006, SEISMIC HAZARD ZONES, MOUNTAIN VIEW AND PALO ALTO QUADRANGLES, SCALE 1:24,000.

N

SCALE IN FEET

0 2,000 4,000

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.

Ninyo & Moore

PROJECT NO.	DATE
402371001	9/14

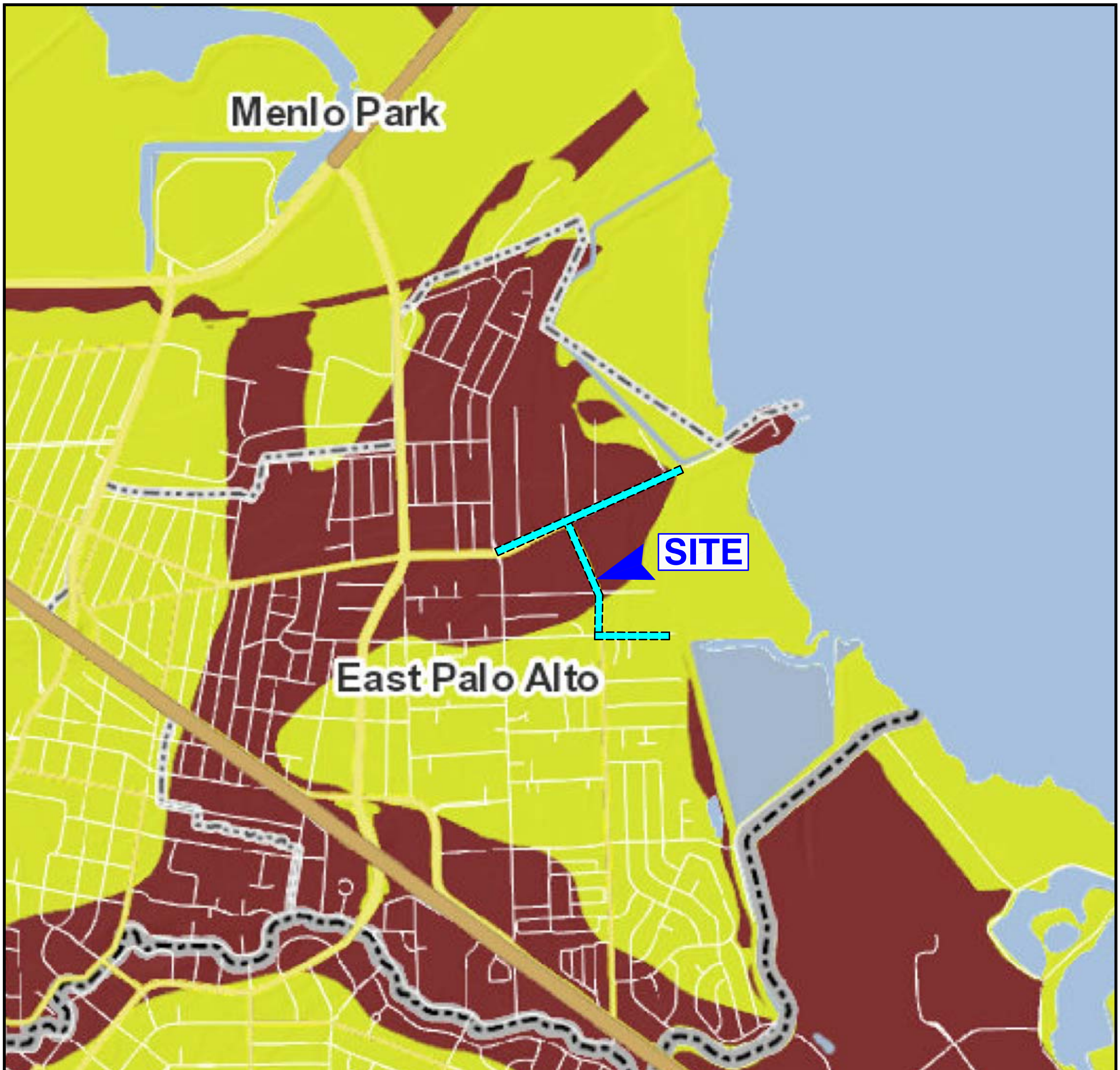
SEISMIC HAZARD ZONES

BAY ROAD IMPROVEMENTS PHASE 2 AND 3
EAST PALO ALTO, CALIFORNIA

FIGURE

5

402371001-SHZ.dwg, Sep 09, 2014, 1:54pm, SN



REFERENCE: USGS OPEN-FILE REPORT 2006-1037; SCALE 1:24,000 (http://pubs.usgs.gov/of/2006/1037/
USGS OPEN-FILE REPORT 00-444; (http://pubs.usgs.gov/of/2000/of00-444/

N

SCALE IN FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

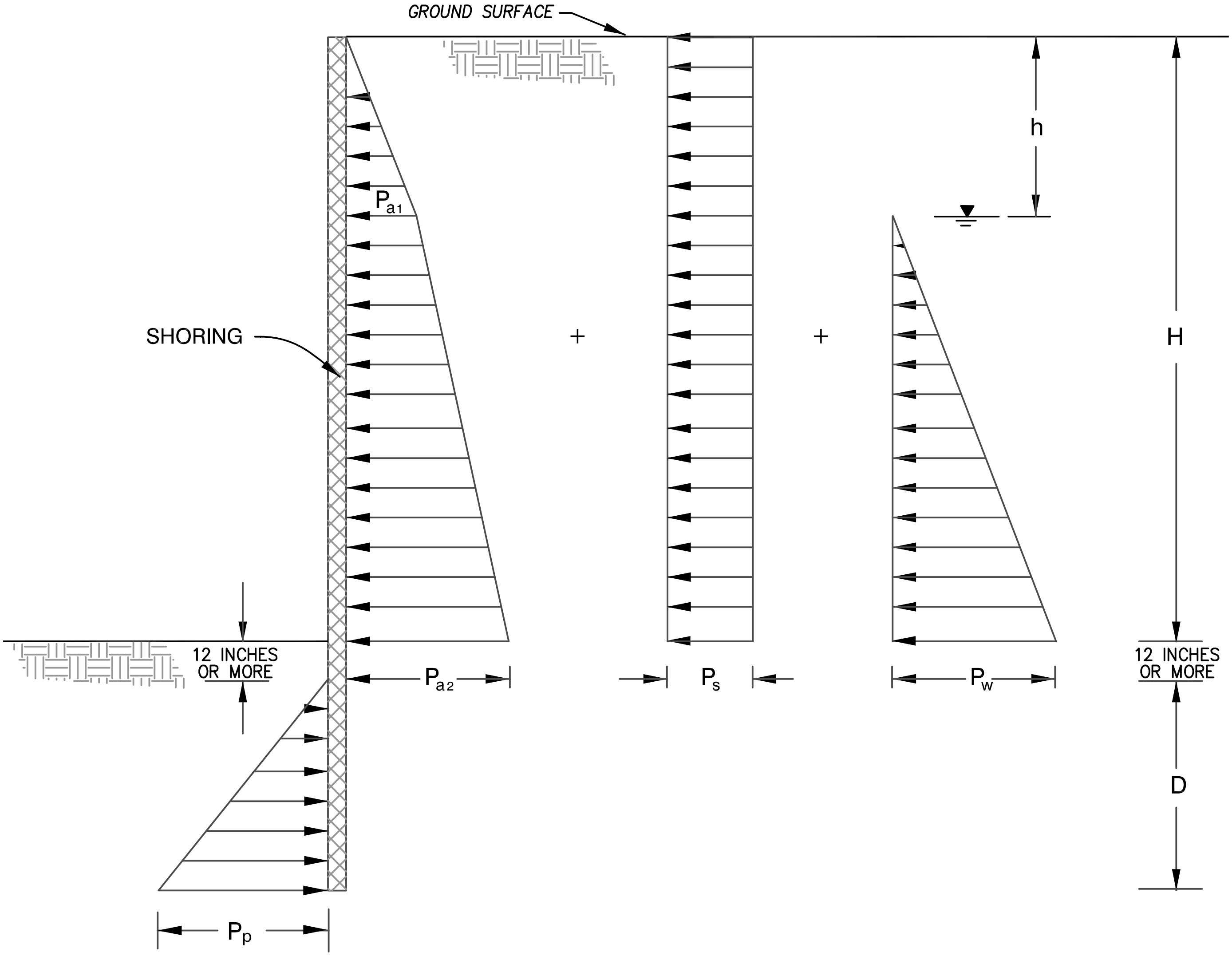
SUSCEPTIBILITY LEVEL

- VERY HIGH
- HIGH
- MODERATE
- LOW
- VERY LOW
- WATER BODIES

- MAJOR ROADS
- LOCAL ROADS

402371001-LS.dwg, Sep 09, 2014, 1:44pm, SN

Ninyo & Moore		LIQUEFACTION SUSCEPTIBILITY	FIGURE
PROJECT NO.	DATE	BAY ROAD IMPROVEMENTS PHASE 2 AND 3 EAST PALO ALTO, CALIFORNIA	6
402371001	9/14		

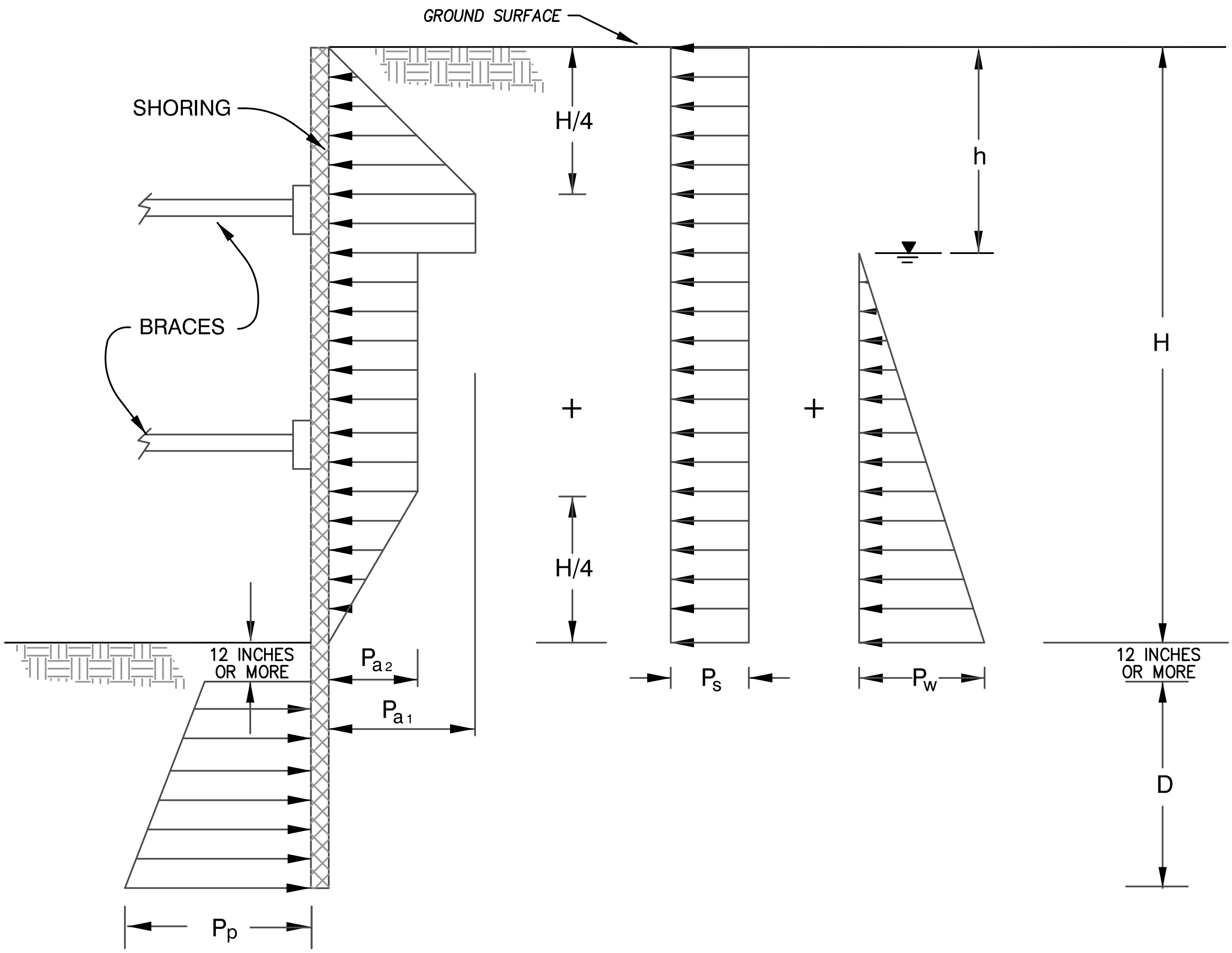


NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   P_{a1} = 43 H psf; $P_{a2} = P_{a1}$ + 24 (H - h) psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_S
   P_S = 120 psf
3. HYDROSTATIC PRESSURE, P_w
   P_w =62.4 (H - h) psf
4. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 180 D psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H, h AND D ARE IN FEET
7. GROUNDWATER TABLE

NOT TO SCALE

Ninyo & Moore		LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING BELOW GROUNDWATER	FIGURE
PROJECT NUMBER	DATE	BAY ROAD IMPROVEMENTS PHASE 2 AND 3 EAST PALO ALTO, CALIFORNIA	7
402371001	9/14		



NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   $P_{a1} = 48$ H psf; $P_{a2} = 26$ H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   $P_s = 120$ psf
3. HYDROSTATIC PRESSURE, P_w
   $P_w = 62.4$ (H - h) psf
4. PASSIVE LATERAL EARTH PRESSURE, P_p
   $P_p = 65$ D + 1000 psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H, h AND D ARE IN FEET
7. GROUNDWATER TABLE

NOT TO SCALE

Ninyo & Moore		LATERAL EARTH PRESSURES FOR BRACED EXCAVATION BELOW GROUNDWATER (STIFF CLAY)	FIGURE
PROJECT NUMBER	DATE	BAY ROAD IMPROVEMENTS PHASE 2 AND 3 EAST PALO ALTO, CALIFORNIA	8
402371001	9/14		

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 logs 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.

Shelby Tube

The Shelby tube is a seamless, thin-walled, steel tube having an external diameter of 2.4 or 3.0 inches and a length of 8 to 30 inches. The tube was connected to the drill rod or a hand tool and pushed into an undisturbed soil mass to obtain a relatively undisturbed sample of soft, cohesive soil in general accordance with ASTM D 1587. When the tube was almost full (to avoid overpenetration), it was withdrawn from the boring, removed from the drill rod or hand tool, sealed at both ends, and transported to the laboratory for testing.

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.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								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.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	ALLUVIUM: Solid line denotes unit change.
								Dashed line denotes material change.
15								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: Sheared Bedding Surface
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

EXPLANATION OF BORING LOG SYMBOLS

PROJECT NO.	DATE	FIGURE
	Rev. 01/03	

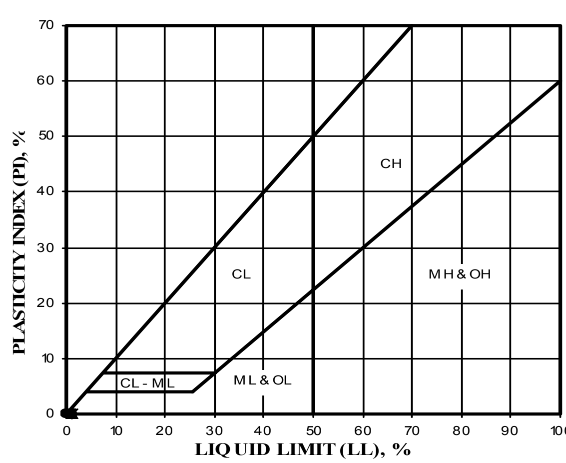
U.S.C.S. METHOD OF SOIL CLASSIFICATION

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 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

PLASTICITY CHART



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

DATE DRILLED	7/23/14	BORING NO.	B-1
GROUND ELEVATION	14'± (MSL)	SHEET	1 OF 2
METHOD OF DRILLING	8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill		
DRIVE WEIGHT	140 LBS Wireline Hammer	DROP	30"
SAMPLED BY	DWM	LOGGED BY	DWM
REVIEWED BY	KG/SG		

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 7.5 inches thick.
							AGGREGATE BASE: Approximately 4.5 inches thick.
	Bulk					SM	FILL: Brown, moist, dense, silty SAND; little gravel.
						CL	ALLUVIUM: Brown, moist, stiff, CLAY; little sand; trace gravel.
	Driven	14					
							Firm.
		5				SM	Brown, moist, loose, silty SAND; trace gravel.
5						CL	Brown, moist, firm, CLAY; little sand.
	Driven	10					
						SC	Brown, moist, loose, clayey SAND with gravel.
		4					Very loose.
	Driven	9					Loose.
10		3					Very loose.
						SM	Brown, moist, very loose, silty SAND.
						CL	Brown, moist, stiff, CLAY.
			▼				Wet.
15			▽				
	Driven	18	28.6	92.7			
							Firm.
	Driven	8					
20							

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
20								Total depth = 20 feet. Groundwater was encountered at 15 feet at 9:35 AM and was measured at a depth of approximately 13.5 feet at 9:40 AM. Groundwater may rise to a level higher that that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps andother documents reveiwed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
25								
30								
35								
40								

DATE DRILLED 7/23/14 BORING NO. B-1

GROUND ELEVATION 14'± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill

DRIVE WEIGHT 140 LBS Wireline Hammer DROP 30"

SAMPLED BY DWM LOGGED BY DWM REVIEWED BY KG/SG

Ninyo & Moore

BORING LOG

BAY ROAD IMPROVEMENT PHASE 2 & 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402371001	9/14	A-2

DATE DRILLED	7/23/14	BORING NO.	B-2
GROUND ELEVATION	11'± (MSL)	SHEET 1 OF 2	
METHOD OF DRILLING	8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill		
DRIVE WEIGHT	140 LBS Wireline Hammer	DROP	30"
SAMPLED BY	DWM	LOGGED BY DWM	REVIEWED BY KG/SG

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 4 inches thick.
							AGGREGATE BASE: Approximately 3 inches thick.
	Bulk					SC	FILL: Dark brown, moist, loose, clayey SAND; little gravel.
	Driven	12	7.9	105.4		SM	ALLUVIUM: Brown, moist, loose, silty SAND; few gravel.
	Driven	12				SP	Light brown, moist, medium dense, poorly-graded SAND with gravel.
5	Driven	24					Some rounded gravel.
	Driven	11	▼ ▽				Wet. Very loose.
	Driven	4				CL	Brown, wet, soft, CLAY; trace rounded gravel.
10		25"/30"					10 PSI.
	Driven	15					Stiff.
15							
	Driven	8					Firm; few rounded gravel.
20							

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BORING LOG

BAY ROAD IMPROVEMENT PHASE 2 & 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402371001	9/14	A-3

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

DATE DRILLED 7/23/14 BORING NO. B-2

GROUND ELEVATION 11'± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill

DRIVE WEIGHT 140 LBS Wireline Hammer DROP 30"

SAMPLED BY DWM LOGGED BY DWM REVIEWED BY KG/SG

DESCRIPTION/INTERPRETATION

20

Total depth = 20 feet.

Groundwater was encountered at 8.5 feet at 10:50 AM and was measured at a depth of approximately 8 feet at 11:00 AM.

Groundwater may rise to a level higher that that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps andother documents reveiwed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

25

The boring was backfilled with cement grout on 7/23/14.

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BORING LOG

BAY ROAD IMPROVEMENT PHASE 2 & 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402371001	9/14	A-4

DATE DRILLED	7/23/14
BORING NO.	B-3
GROUND ELEVATION	10'± (MSL)
SHEET	1 OF 2
METHOD OF DRILLING	8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill
DRIVE WEIGHT	140 LBS Wireline Hammer
DROP	30"
SAMPLED BY	DWM
LOGGED BY	DWM
REVIEWED BY	KG/SG

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 3 inches thick.
						CL	AGGREGATE BASE: Approximately 1 inch thick.
							ALLUVIUM: Black, moist, stiff, CLAY; few sand.
							Dark brown.
		14	18.3	105.6			
5		6					Brown, firm; little sand.
		9					Wet; little sand.
			25.4			ML	Brown, wet, loose, sandy SILT.
		5					
		5					Very loose.
10							
						CL	Brown, wet, soft, CLAY; few sand.
		14"/30"					10 PSI at top; 30 PSI at bottom.
15							
							Dark brown, stiff; few gravel.
		9					
20							

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BORING LOG

BAY ROAD IMPROVEMENT PHASE 2 & 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402371001	9/14	A-5

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
20								Total depth = 20 feet. Groundwater was encountered at 8.5 feet at 12:10 PM and was measured at a depth of approximately 6.5 feet at 12:45 PM. Groundwater may rise to a level higher that that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. The boring was backfilled with cement grout on 7/23/14.
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DATE DRILLED 7/23/14 BORING NO. B-3

GROUND ELEVATION 10'± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill

DRIVE WEIGHT 140 LBS Wireline Hammer DROP 30"

SAMPLED BY DWM LOGGED BY DWM REVIEWED BY KG/SG

Ninyo & Moore

BORING LOG

BAY ROAD IMPROVEMENT PHASE 2 & 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402371001	9/14	A-6

DATE DRILLED	7/23/14	BORING NO.	B-4
GROUND ELEVATION	7'± (MSL)	SHEET 1 OF 2	
METHOD OF DRILLING	8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill		
DRIVE WEIGHT	140 LBS Wireline Hammer	DROP	30"
SAMPLED BY	DWM	LOGGED BY DWM	REVIEWED BY KG/SG

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 4 inches thick.
							AGGREGATE BASE: Approximately 4 inches thick.
						CL	ALLUVIUM: Gray to black, moist, stiff, sandy CLAY; few gravel.
		18	19.8	105.5			
		7					Black, wet, firm.
5		13	24.8	97.4			Gray to black, stiff.
		3					Soft.
						SM	Brown, wet, loose, silty SAND.
		15				CL	White, wet, stiff, CLAY.
10							
		23"/30"					15 PSI.
15							
		19					Olive brown; very stiff.
20							

BORING LOG

BAY ROAD IMPROVEMENT PHASE 2 & 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402371001	9/14	A-7

Ninyo & Moore

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
20								Total depth = 20 feet. Groundwater was encountered at 5 feet at 3:07 PM and was measured at a depth of approximately 4 feet at 3:12 PM. Groundwater may rise to a level higher that that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. The boring was backfilled with cement grout on 7/23/14.
25								
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DATE DRILLED 7/23/14 BORING NO. B-4

GROUND ELEVATION 7'± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill

DRIVE WEIGHT 140 LBS Wireline Hammer DROP 30"

SAMPLED BY DWM LOGGED BY DWM REVIEWED BY KG/SG

Ninyo & Moore

BORING LOG

BAY ROAD IMPROVEMENT PHASE 2 & 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402371001	9/14	A-8

DATE DRILLED	7/23/14
BORING NO.	B-5
GROUND ELEVATION	9'± (MSL)
SHEET	1 OF 2
METHOD OF DRILLING	8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill
DRIVE WEIGHT	140 LBS Wireline Hammer
DROP	30"
SAMPLED BY	DWM
LOGGED BY	DWM
REVIEWED BY	KG/SG

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 6 inches thick.
	Bulk					SC	FILL: Dark brown, moist, dense, clayey SAND; little gravel.
							Light brown.
	Driven	12				CL	ALLUVIUM: Black, moist, stiff, CLAY.
5	Driven	5	▼				No recovery; wet, firm.
	Driven	8	24.2 ▽	96.8			Gray.
	Driven	6					Olive gray.
						SP-SM	Olive, wet, loose, poorly graded SAND with silt.
10	Driven	4					Very loose.
						CL-ML	Brown to dark brown, wet, soft, sandy silty CLAY; few gravel.
15	Driven	5	31.0	89.4			
20	Driven	10					Dark gray, stiff; few sand.

Ninyo & Moore

BORING LOG

BAY ROAD IMPROVEMENT PHASE 2 & 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402371001	9/14	A-9

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DATE DRILLED 7/23/14 BORING NO. B-5 GROUND ELEVATION 9'± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 8" Hollow Stem Auger (Exploration Geoservices) Mobile B-53 Drill DRIVE WEIGHT 140 LBS Wireline Hammer DROP 30" SAMPLED BY DWM LOGGED BY DWM REVIEWED BY KG/SG **DESCRIPTION/INTERPRETATION**
20								Total depth = 20 feet. Groundwater was encountered at 7 feet at 1:20 PM and was measured at a depth of approximately 5 feet at 1:25 PM. Groundwater may rise to a level higher that that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. The boring was backfilled with cement grout on 7/23/14.
25								
30								
35								
40								

Ninyo & Moore

BORING LOG

BAY ROAD IMPROVEMENT PHASE 2 & 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402371001	9/14	A-10

APPENDIX B

LABORATORY TESTING

Classification

Soils were 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 and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-1.

Gradation Analysis

Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures B-2 through B-4. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples 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 classifications are shown on Figure B-5.

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 the selected sample was evaluated in general accordance with CT 417, and CT 422, respectively. The test results are presented on Figure B-6.

R-Value

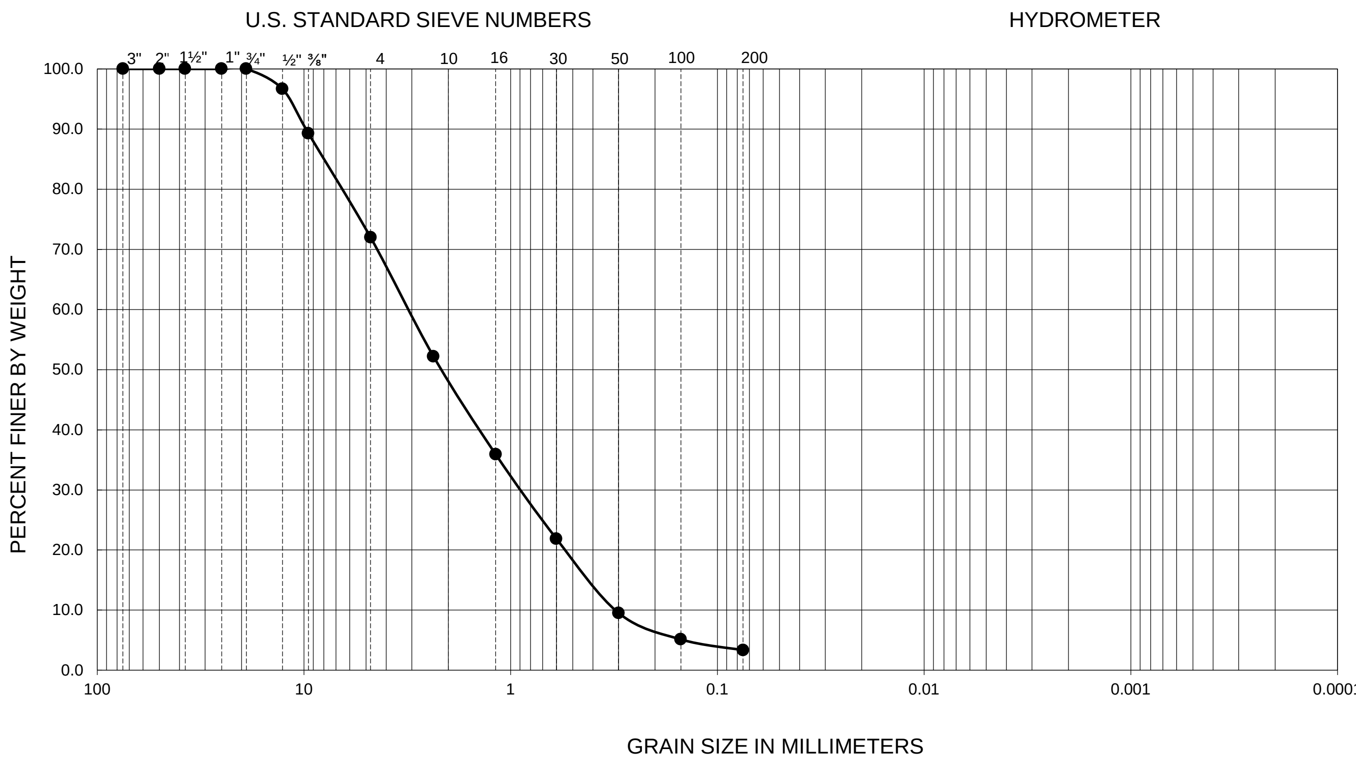
The resistance value, or R-value, for site soils was evaluated in general accordance with CT 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test result is shown on Figure B-7.

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	7.0 - 8.5	Brown, loose, clayey SAND with gravel.	77	40	SC
B-4	6.0 - 6.5	Gray to black, stiff, CLAY; few gravel.	95	64	CL
B-5	14.5 -15.0	Brown to dark brown, soft, silty CLAY	100	54	CL-ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT NO.	DATE	BAY ROAD IMPROVEMENTS PHASE 2 AND 3 EAST PALO ALTO, CALIFORNIA	B-1
402371001	9/14		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

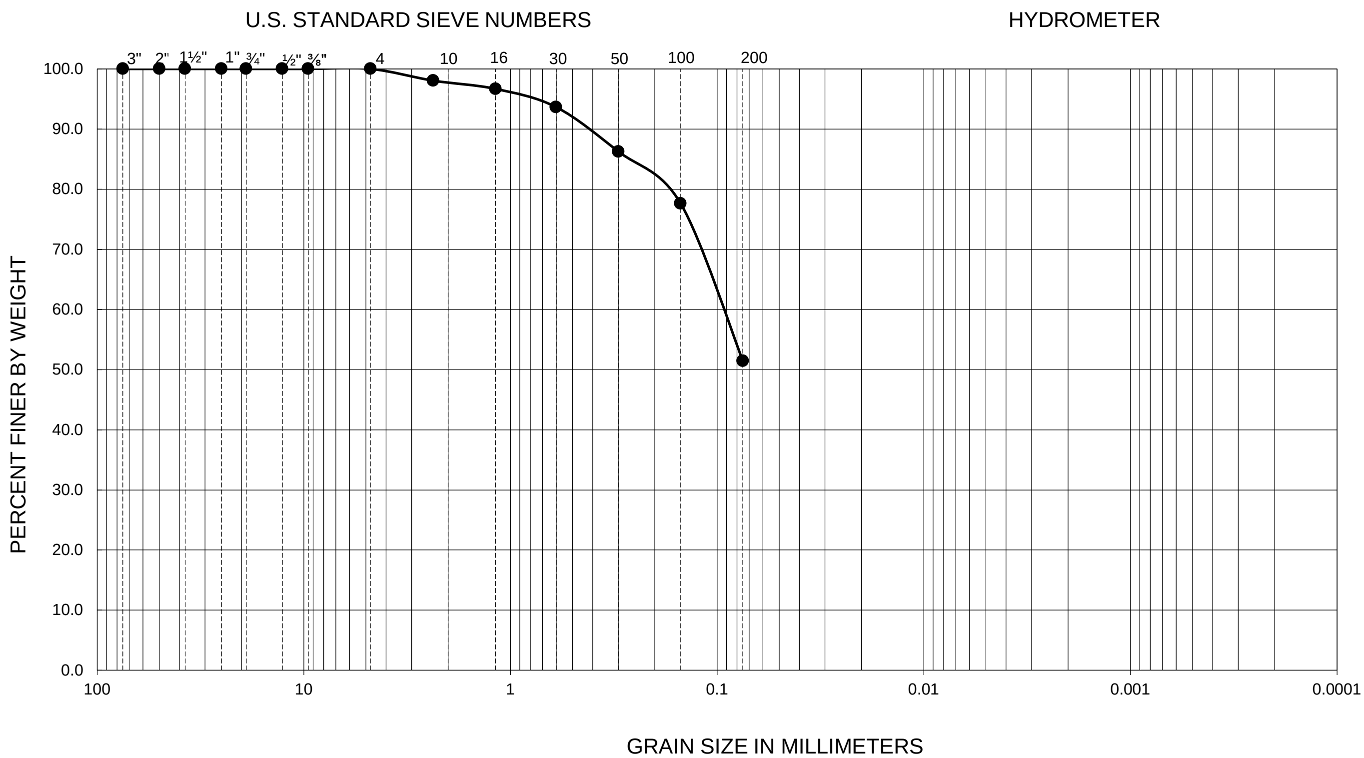


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-2	7.0 - 8.5	--	--	--	0.31	0.9	3.1	10.0	0.8	3	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	BAY ROAD IMPROVEMENTS PHASE 2 AND 3 EAST PALO ALTO, CALIFORNIA	B-2
402371001	9/14		

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-3	7.0 - 8.5	23	20	3	--	--	0.10	--	--	51	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore

GRADATION TEST RESULTS

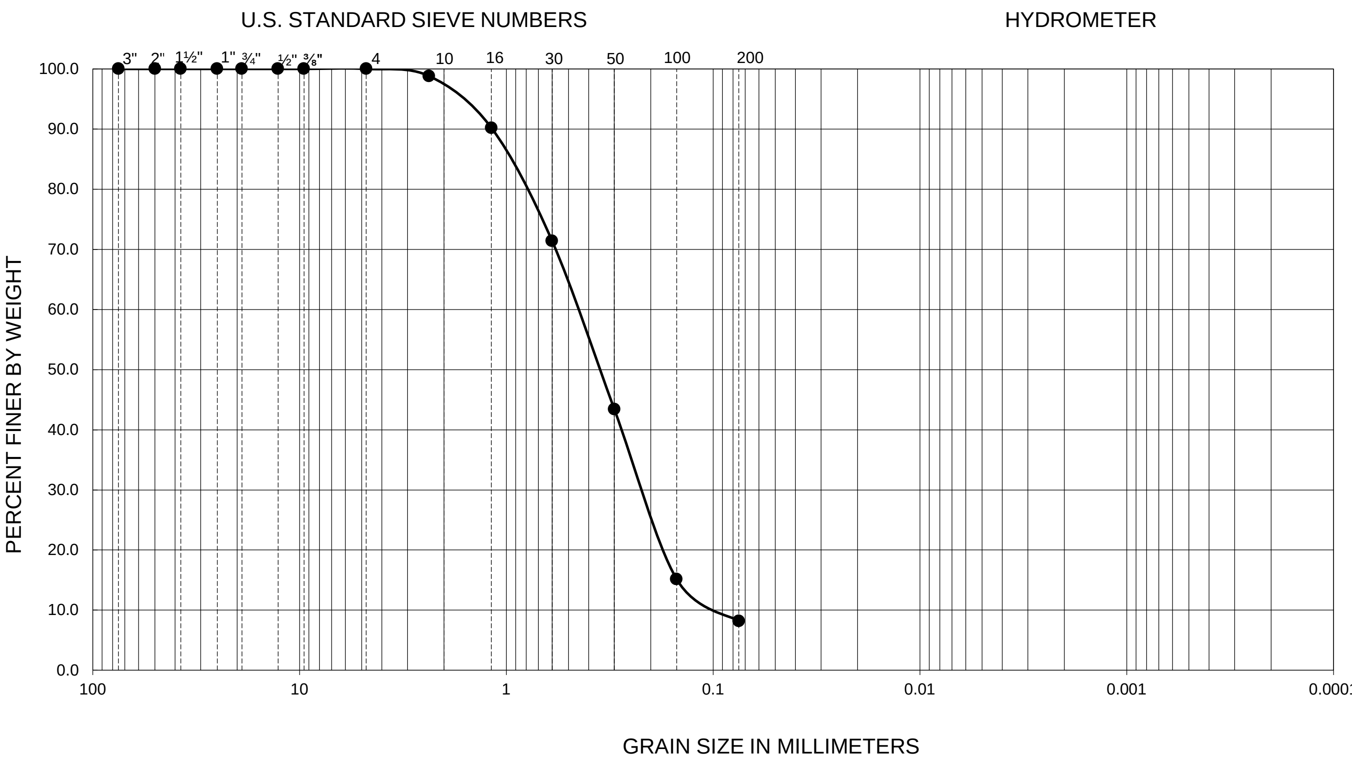
BAY ROAD IMPROVEMENTS PHASE 2 AND 3
EAST PALO ALTO, CALIFORNIA

PROJECT NO.	DATE
402371001	9/14

FIGURE
B-3

Fig 3.xls

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



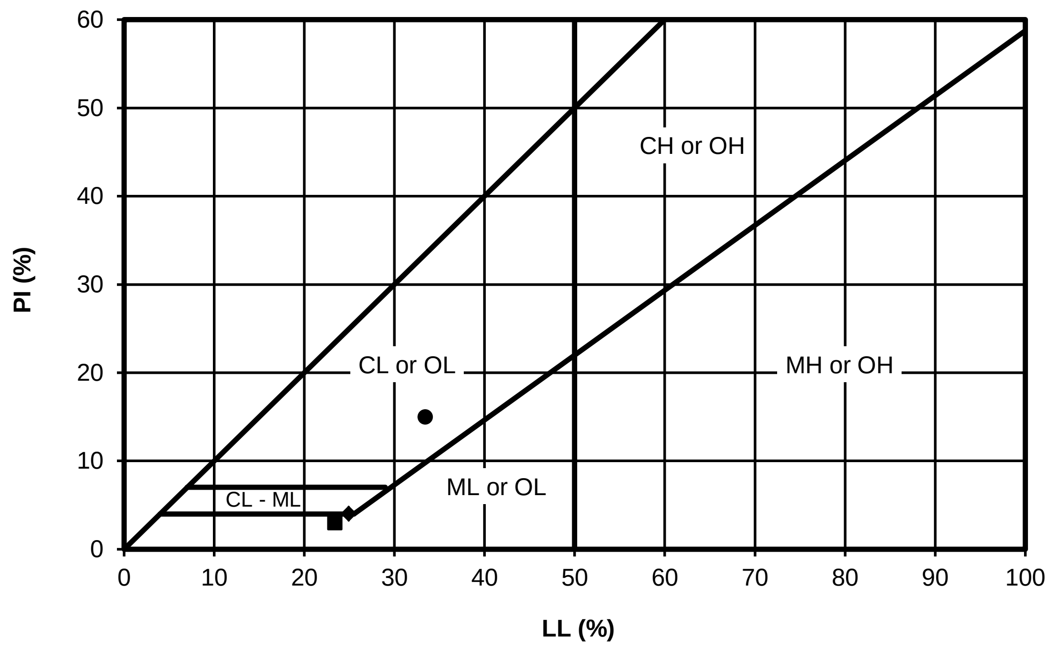
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-5	8.5-10.0	--	--	--	0.10	0.22	0.44	4.4	1.1	8	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	BAY ROAD IMPROVEMENTS PHASE 2 AND 3 EAST PALO ALTO, CALIFORNIA	B-4
402371001	9/14		

Fig 4.xls

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-1	7.0 - 8.5	33	18	15	CL	SC
■	B-3	7.0 - 8.5	23	20	3	ML	ML
◆	B-5	14.5 - 15.0	25	21	4	CL-ML	CL-ML



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Ninyo & Moore

ATTERBERG LIMITS TEST RESULTS

FIGURE

B-5

PROJECT NO.	DATE
#REF!	9/14

BAY ROAD IMPROVEMENTS PHASE 2 AND 3
EAST PALO ALTO, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-4	0.5-5.0	8.2	764	230	0.023	340

[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

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PROJECT NO.	DATE
402371001	9/14

CORROSIVITY TEST RESULTS

BAY ROAD IMPROVEMENTS PHASE 2 AND 3
EAST PALO ALTO, CALIFORNIA

FIGURE
B-6

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
B-4	0.5 - 5.0	CL	6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

Ninyo & Moore

R-VALUE TEST RESULTS

FIGURE

B-7

PROJECT NO.	DATE
402371001	9/14

BAY ROAD IMPROVEMENTS PHASE 2 AND 3
EAST PALO ALTO, CALIFORNIA