GEOTECHNICAL EVALUATION
ASHLAND FAMILY HOUSING
16309, 16325, 16327, AND 16331 KENT AVENUE
SAN LORENZO, CALIFORNIA

PREPARED FOR:
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May 3, 2013
Project No. 402090001

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Mr. Brian Saliman
Resources for Community Development
2220 Oxford Street
Berkeley, California 94704

Subject: Geotechnical Evaluation
Ashland Family Housing
16309, 16325, 16327, and 16331 Kent Avenue
San Lorenzo, California

Dear Mr. Saliman:

In accordance with your authorization, we have performed a geotechnical evaluation for the new Ashland Family Housing project to be located at 16309, 16325, 16327, and 16331 Kent Avenue in San Lorenzo, California. This report presents our geotechnical findings, conclusions, and recommendations regarding the proposed project.

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

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

Sincerely,
NINYO & MOORE

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1. INTRODUCTION

In accordance with your request, we have performed a geotechnical evaluation for the new Ashland Family Housing project to be located at 16309, 16325, 16327, and 16331 Kent Avenue in San Lorenzo, California (Figure 1). This report presents our findings and conclusions regarding the geotechnical conditions encountered at the Ashland Family Housing project site, and our recommendations for design and construction of the project.

2. SCOPE OF SERVICES

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

- Review of background data listed in the References section of this report. The data reviewed included topographic maps, geologic data and maps, fault and seismic hazard maps, flood hazard maps, a previous geotechnical report for the site prepared by Jensen Van Lienden (JVL), 2011, and a site plan for the project.
- Reviewed of previous geotechnical reports by Ninyo & Moore for the new Holland Park northwest of the project site (Ninyo & Moore, 2009) and for the adjacent new Ashland Youth Center north of the site (Ninyo & Moore, 2011).
- Geologic reconnaissance to observe site conditions including existing usage, topographic features, drainage, and surficial geologic conditions.
- Mark out of the proposed exploratory boring locations prior to contacting Underground Service Alert.
- Procurement of subsurface drilling permits from the Alameda County of Public Works Agency (ACPWA).
- Subsurface exploration consisting of drilling and sampling of two mud rotary wash borings and one hollow-stem auger boring. The borings were advanced to depths of approximately 10 to 51½ feet. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk and relatively undisturbed soil samples for laboratory tests. The borings were backfilled in conformance with the ACPWA drilling permits.

- Laboratory testing of selected soil samples was performed to evaluate the geotechnical properties of the subsurface materials including in-situ moisture content and density, percentage of soil particles finer than the No. 200 sieve, Atterberg limits, expansion index, and unconfined compressive strength.

- Compilation and analysis of the field and laboratory data to evaluate the following:

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

  - Geotechnical issues that may impact the design, construction, and/or performance of the proposed improvements.

  - Design parameters for mat and ribbed (waffle slab) foundations.

  - Lateral earth pressures for retaining wall design.

  - Preliminary pavement design recommendations.

  - Soil type and seismic coefficients for seismic design conforming to the 2010 CBC.

  - Earthwork guidelines for excavation and compaction, subgrade preparation, suitability of using the onsite soil as fill material for the proposed improvements, and trench backfill.

- Preparation of this geotechnical report presenting our findings and conclusions from our evaluation, and our geotechnical recommendations for the design and construction of the proposed Ashland Family Housing project.

3. SITE CONDITIONS

The Ashland Family Housing project site is located at 16309, 16325, 16327, and 16331 Kent Avenue in San Lorenzo, California (Figure 1). The project site is located at approximately 37.694749 degrees north latitude and -122.114758 degrees west longitude. The site is irregular in shape and is bound to the north by the Ashland Youth Center and commercial properties adjacent to East 14th Street, to the west by baseball fields, to the east by Kent Avenue, and to the south by single-family residences (Figure 2). Elevations range from about 36½ feet above Mean Sea Level (MSL) at the northwest corner of the project site to about 40¾ feet at the southeast corner of the site (Kava Massih Architects, 2013). Existing structures on the site are vacant and consist of mobile homes/trailers in the northern and central portions of the site, single-family structures, a two-story multi-unit residential building along the southern edge of the site, and a retail building in the northwest corner of the site.

Our review of topographic maps for the site indicates that drainage for the site consists of sheet flow towards the west (USGS, 2012). The creek and watershed map of Hayward and San Leandro (Sowers, 2000) indicates that the site is located north of San Lorenzo Creek. The map indicates that a drainage culvert is located adjacent to the site along East 14th Street.

4. PROJECT DESCRIPTION

The Ashland Family Housing project consists of Buildings A through E. Buildings A, B, C, and D are multi-story and multi-unit residential structures with footprints totaling approximately 30,000 square feet. The buildings are shown on Figure 2. Building E is a one-story community space building with a footprint of approximately 2,500 square feet. Based upon our review of plans prepared by Kava Massih Architects (2013), ancillary improvements at the project site will consist of trash enclosures, a community garden, raised planter boxes, short site retaining walls up to about 6 feet in height, concrete flatwork, and asphalt concrete perimeter parking and driveways. Up to approximately 5 feet of fill may be placed to divert surface flow towards the east.

5. FIELD EXPLORATION AND LABORATORY TESTING

Our field exploration at the Ashland Family Housing project site included a geologic reconnaissance and subsurface exploration that was conducted on March 8, 2013. The subsurface exploration consisted of drilling, logging, and sampling of three soil borings. The boring locations, as shown on Figure 2, were selected based on the proposed building layout shown on a site plan provided by Kava Massih Architects and on the locations of the exploratory borings of a subsurface exploration performed by Jensen-Van Lienden Associates, Inc. (JVL, 2011). Prior to commencing the subsurface exploration, Underground Service Alert was notified for field marking of the existing utilities, and a drilling permit was obtained from ACPWD.

Two borings (Borings B-1 and B-2) were drilled to an approximate depth of 51½ feet below the existing grade with a truck-mounted mud rotary wash rig. Boring B-3 was drilled to an approximate depth of 10 feet below the existing grade with a truck-mounted drill rig equipped with hollow stem augers. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected drive and bulk soil samples from the borings. The samples were then transported to our in-house geotechnical laboratory for testing. The borings were backfilled with Portland cement grout shortly after drilling in conformance with the ACPWD drilling permit. Descriptions of the subsurface materials encountered are presented in the following sections. Logs of the borings are presented in Appendix A.

Laboratory testing of soil samples recovered from the borings included in-place moisture content and dry density, percentage of particles finer than the No. 200 sieve, Atterberg limits, expansion index, and unconfined compressive strength. The results of the in-place moisture content and dry density tests are shown at the corresponding sample depths on the boring logs in Appendix A. The results of the other laboratory tests performed are presented in Appendix B.

6. GEOLOGY AND SUBSURFACE CONDITIONS

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

6.1. Regional Geologic Setting

The project site is located on the east side of San Francisco Bay in the Coast Ranges geomorphic province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill structural valleys and line continental margins. The San Francisco Bay Area has several ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. Further discussion of faulting relative to the site is provided in Section 7.1.1 of this report.

6.2. Site Geology

The Ashland Family Housing project site is mapped as being underlain by Quaternary alluvium (Dibblee, 2005, CGS, 2003a, Graymer, 2000). Dibblee (2005) indicates that the project site is underlain by Holocene surficial sediments that consist of alluvial gravel, sand, and clay; and CGS (2003a) indicates that the project site is underlain by Holocene alluvial fan deposits. Graymer (2000), shows that the site is generally underlain by Pleistocene alluvial fan and fluvial deposits that consist of dense gravelly and clayey sand or clayey gravel and sandy clay. Our subsurface exploration indicates that the project site is generally underlain by artificial fill overlying alluvium.

6.3. Subsurface Conditions

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

6.3.1. Pavement Section

The pavement section encountered in Borings B-1, B-2, and B-3 consisted of asphalt concrete about 1¾ to 2 inches thick and aggregate base between about 2½ and 3 inches thick.

6.3.2. Fill

Fill was encountered in the borings at the subject site from below the pavement section to depths of about 2 to 3½ feet below the existing grade. The fill material encountered in the borings generally consisted of moist, firm to stiff clay with trace to few sand and trace to few gravel.

6.3.3. Alluvium

Alluvium was encountered below the fill to the depth of exploration in the borings. The alluvium encountered in the borings generally consisted of moist to saturated, soft to very stiff, silty to sandy clay; and moist to wet, very loose to loose, silty to clayey sand with trace to some gravel.

6.4. Groundwater

Groundwater was encountered in Boring B-3 during drilling. The depth to groundwater in our boring was about 7 feet below the existing grade at the time of the subsurface exploration. Additionally, the depth of groundwater was measured to be approximately 6 feet below the existing ground surface in a well located in the southeast portion of the site. These reading correspond with an elevation of the groundwater surface that ranges between about 31¼ and 32¾ feet MSL at the project site.

However, fluctuations in the groundwater level may occur because of variations in ground surface factors. In addition, groundwater levels in fine-grained soil (e.g. those at this site) are known to take significant time to stabilize. Because of time constraints, the borings were required to be backfilled on the day of drilling. The Seismic Hazard Zone Report for the Hayward Quadrangle (CGS, 2003a) indicates that the historic high groundwater level in the site vicinity is between 5 and 10 feet below the ground surface.

7. DISCUSSION

The impact of a number of geotechnical issues and geologic hazards on the proposed improvements were evaluated as part of this study. The geotechnical issues and geologic hazards considered included seismic hazards, settlement of compressible soil layers from static loading, unsuitable materials, excavation characteristics, and corrosive and expansive soil. These issues and hazards are discussed in the following subsections.

7.1. Seismic Hazards

The project site is located in an area considered to be seismically active. The seismic hazards considered in this study include the potential for ground rupture, ground shaking due to seismic activity, seismically induced liquefaction, dynamic settlement, and lateral spreading. These potential hazards are discussed in the following subsections.

7.1.1. Faulting and Ground Surface Rupture

There are numerous recognized faults in northern California. As defined by the Alquist-Priolo Earthquake Fault Zoning Act (Bryant and Hart, 2007), 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 Quaternary time (approximately the last 1.6 million years) but for which evidence of Holocene movement has not been established.

The site is not located within an Earthquake Zones of Required Investigation, based on Alquist-Priolo Earthquake Fault Zones, established by the state geologist (California Geological Survey, 2012) to delineate regions of potential ground surface rupture adjacent to active faults. However, the site is located in a seismically active area, as is the majority of northern California, and the potential for strong ground motion in the project area is considered significant during the design life of the proposed structure

The closest known active faults are the Ashland and Hayward faults located approximately 1,700 and 1,900 feet northeast of the project site, respectively (CGS, 2012). The moment magnitude associated with a rupture of the southern segment of the Hayward Fault is 6.7 (Cao et al., 2003). The approximate locations of major faults and their geographical relationship to the project vicinity are shown on Figure 3.

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. Therefore, the probability of damage from surface fault rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

7.1.2. Ground Motion

The 2010 CBC recommends that the design of structures be based on the horizontal peak ground acceleration (PGA) having a 2 percent probability of exceedance in 50 years which is defined as the Maximum Considered Earthquake (MCE). The statistical return period for PGA_{MCE} is approximately 2,475 years. The probabilistic PGA_{MCE} for the site was calculated as 0.71g using the United States Geological Survey (USGS, 2011) ground motion calculator (web-based). The design PGA was estimated to be 0.47g using the USGS ground motion calculator.

7.1.3. Liquefaction and Strain Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soil of low plasticity (liquefaction) or in wet, sensitive, cohesive soil (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.

During our subsurface exploration, we encountered layers of relatively loose granular soil in Borings B-1 and B-2 that could be susceptible to liquefaction. We evaluated the liquefaction susceptibility of these deposits in accordance with the method presented by Youd et al. (2001) using the blowcount data collected during our subsurface exploration and considering a peak ground acceleration (PGA) of 0.47g with a corresponding earthquake magnitude of 6.7 based on our deaggregation analysis of the design ground motion. For the liquefaction analysis, we assumed a groundwater depth of 5 feet consistent with the mapped range of the historic high groundwater level. The results of our analysis (presented in Appendix C) indicate that the loose granular soil present in Borings B-1 and B-2 below the assumed groundwater table are susceptible to liquefaction under the considered ground motion. The loose granular soil susceptible to liquefaction was encountered in Boring B-1 from approximately 5½ to 6 feet below the existing ground surface and in Boring B-2 from approximately 5 to 9 feet and 31 to

33 feet below the ground surface. The loose granular soil susceptible to liquefaction encountered between depths of 5 to 9 feet below the ground surface will impact the bearing capacity of shallow foundations. Recommendations for mat foundations and remedial grading to create a pad of cement-treated soil are provided to mitigate the impact of a bearing capacity reduction due to liquefaction. Other liquefaction-related impacts including dynamic settlement, ground subsidence, and lateral spread are discussed in the following paragraphs. Index testing performed on samples of fine-grained soil collected during our subsurface exploration indicate that the fine-grained soil is not susceptible to liquefaction based on the criteria reported by Bray & Sancio (2006).

The clay and silty clay encountered during our subsurface exploration are not known to be particularly sensitive. Furthermore, index testing on samples of fine-grained soil collected during our subsurface exploration indicate that the liquidity index of the near surface cohesive soil is less than 0.5 which is consistent, over the stress range of interest, with relatively insensitive soil. Therefore, we do not regard seismic strain-softening behavior as a design consideration.

7.1.4. Dynamic Settlement

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

During our subsurface exploration, we encountered deposits of loose granular soil in Borings B-1 and B-2 that could dynamically compact following an earthquake. We evaluated the potentials for dynamic settlement of these deposits in accordance with the method presented by Tokimatsu and Seed (1987) for saturated sand and the equations published by Pradel (1998) for dry sand using the blowcount data collected during our subsurface exploration and considering a peak ground acceleration (PGA) of 0.47g with a corresponding earthquake magnitude of 6.7 based on our deaggregation analysis of the design ground motion. The results of our analyses indicate that a total dynamic settlement of approximately 3⅓ inches, including approximately 1⅓ inches of dry sand settlement, may occur following the considered seismic event. Remedial grading to create a pad of cement-treated soil, as described in Section 9.1.2.1 of this report, the total dynamic settlement to approximately 2¾ inches, including about ¾ inches of dry sand settlement, following the considered seismic event. Differential dynamic settlement after remedial grading is estimated to be on the order of about 1¼ inches over a horizontal distance of 90 feet. We anticipate that the new buildings can be designed to accommodate this degree of dynamic settlement.

7.1.5. Ground Subsidence

Sand boils that occur when liquefied, near-surface soil escapes to the ground surface, can result in ground subsidence due to loss of material that is in addition to dynamic settlement. Recommendations for remedial grading to create a pad of cement-treated soil below the proposed buildings are provided to mitigate the impact of ground subsidence due to sand boils.

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.

7.2. Static Settlement

We understand that the existing grade at the site will be increased up to 5 feet to divert surface runoff to the east. The placement of fill to raise the grade will increase the effective stress in the soil resulting in settlement. Our analysis indicates that raising the site grade by 5 feet will result in approximately 1¾ inches of total static settlement with a differential settlement of approximately 1 inch within the building pad area over a horizontal distance of 100 feet for a fill embankment that is approximately 5 feet high at the western edge of the site and gradually slopes to the existing grade at the eastern edge of the site.

Additionally, the column loads for the proposed Ashland Family Housing project are expected to be typical. We anticipate, therefore, that the static settlement of mat foundations due to structural loads will be tolerable provided the recommendations presented in this report are followed.

7.3. Unsuitable Materials

Fill materials that were not placed and compacted under the observation of a geotechnical engineer, or fill materials lacking documentation of such observation, are considered undocumented fill. Undocumented fill is considered unsuitable as a bearing material below structures due to the potential for differential settlement resulting from variable support characteristics or the potential inclusion of deleterious materials. Undocumented fill was found in our exploratory borings to depths of up to 3½ feet below the existing grade. Recommendations for remedial grading to mitigate the unsuitable support characteristics of undocumented fill are presented in Section 9.1.2.1.

Soil containing roots or other organic matter are not suitable as fill or subgrade material below structures, walls, pavements, flatwork, or engineered fill. Surficial soil containing roots or other organic matter should be removed as part of the clearing and grubbing operations.

7.4. Excavation Characteristics

We anticipate that grading for the new Ashland Family Housing site will be relatively minor given the relatively flat topography of the site. However, we anticipate that excavation will extend up to about 4 feet below the existing grade for remedial grading associated with the cement treatment of the near surface soil to mitigate the potential impacts of liquefaction. Other grading may include excavations on the order of 2 to 5 feet for utility trenches and landscape plantings. The surficial materials encountered during our subsurface exploration over this interval consisted of firm to stiff clay and loose silty to clayey sand. We anticipate that heavy earthmoving equipment in good working condition should be able to make the proposed excavations. Near vertical cuts in the fill and alluvium may not be stable, particularly if the excavation is exposed to rainfall runoff, encounters seepage or extends into soil that could be classified as sand or gravel. Appropriate temporary slopes or shoring may be needed stabilize excavation sidewalls. Recommendations for excavation stabilization are presented in Section 9.1.3. The bottom of excavations extending below or near historic groundwater levels may unstable under equipment loading due to wet conditions. Recommendations for construction dewatering are provided.

7.5. Corrosive Soil

An evaluation of the corrosion potential of on-site soil was conducted during our study to assess the impact on concrete and metals for the New Holland Park and Youth Center located northwest of the site (Ninyo & Moore, 2009). The corrosion potential was evaluated using the results of limited laboratory testing on samples obtained during our subsurface study performed for the New Holland Park and Youth Center. Laboratory testing to quantify pH, resistivity, chloride, and soluble sulfate contents was performed on samples of the fill and alluvium and the results of the tests are presented in Table 1.

Table 1 – Corrosivity Test Results

Sample Location[1]	Sample Depth (FT)	pH[2]	Resistivity[2] (Ohm-cm)	Sulfate Content[3]		Chloride Content[4] (ppm)
				(ppm)	(%)	
B-5	0 – 2.75	7.3	2,100	50	0.005	215
B-7	2.5 – 5.0	7.6	1,206	100	0.010	150

[1] Borings excavated during our subsurface exploration for the New Holland Park and Youth Center project (Ninyo and Moore, 2009).
[2] Performed in general accordance with California Test Method 643.
[3] Performed in general accordance with California Test Method 417.
[4] Performed in general accordance with California Test Method 422.

Caltrans defines a corrosive environment as an area within 1,000 feet of brackish water or where the soil contains more than 500 parts per million of chlorides, sulfates of 0.2 percent or more, or 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 samples of material tested do not meet the definition of a corrosive environment and the sulfate exposure to concrete is negligible. Ferrous metals will still undergo corrosion on site, but special mitigation measures are not needed.

Table 2 – Criteria for Deleterious Soils on Concrete

Sulfate Content Percent by Weight	Sulfate Exposure
0.0 to 0.1	Negligible
0.1 to 0.2	Moderate
0.2 to 2.0	Severe
> 2.0	Very Severe

Reference: American Concrete Institute (ACI) Committee 318 Table 4.3.1 (ACI, 2012)

7.6. Expansive Soil

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soil containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. Laboratory testing was performed on a selected sample of the near-surface soil to evaluate the expansion index. The test was performed in general accordance with the American Society of Testing and Materials (ASTM) Standard D 4829 (Expansion Index). The results of our laboratory testing indicate that the expansion index of the near-surface soil sample is 44. This result is indicative of a low expansion characteristic. Based upon these results, it is our opinion that special mitigation measures for expansive soil should not be needed.

8. CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint provided that the recommendations presented in this report are incorporated into the design and construction of the subject project. Key findings from the geotechnical evaluation and subsurface exploration include the following:

- Fill soil was encountered in our boring to depths of up to approximately 3½ feet below the ground surface at the site. These materials are considered unsuitable beneath the proposed buildings. Recommendations for remedial grading are provided.
- Excavations may be unstable due to a shallow groundwater table. Recommendations for dewatering of excavations are presented in Section 9.1.4.
- The site will experience a relatively large degree of ground shaking due to a significant earthquake event on a nearby fault.
- Raising the site grade to improve drainage will increase the effective stress in the soil resulting in settlement. Our analysis indicates that raising the site grade by 5 feet will result in approximately 1¾ inches of total static settlement due to fill placement with a differential settlement of approximately 1 inch within the building pad area over a horizontal distance of 100 feet. Optional recommendations are provided to reduce the anticipated settlement due to fill placement by surcharging the building pad areas.
- Our subsurface exploration encountered layers of shallow granular soil that may liquefy following a significant earthquake. Recommendations for remedial grading are provided to create a pad of cement-treated soil. The pad of cement-treated soil will mitigate the impact

of a bearing capacity reduction due to liquefaction, impede manifestation of sand boils and resulting subsidence below the buildings, and reduce the potential dynamic settlement. We estimate that the total dynamic settlement due to the design ground motion will be on the order of 2¾ inches after remedial grading with a differential dynamic settlement of approximately 1¼ inches over a horizontal distance of 90 feet. Recommendations for mat or waffle slab foundations are provided.

9. RECOMMENDATIONS

The following guidelines should be used in the preparation of the construction plans and specifications. Ninyo & Moore should review the plans and specifications to check that these recommendations are appropriately interpreted and incorporated.

9.1. Earthwork

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

9.1.1. Site Preparation

Site preparation should begin with the removal of vegetation, utility lines, asphalt, concrete, debris and other deleterious materials from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dumpsite away from the project area. Existing utilities within the project limits that are to remain in service should be re-routed, or protected from damage. Abandoned utilities should be removed. Excavations resulting from removal of buried utilities or obstructions should be backfilled with compacted fill.

9.1.2. Observation, Removals, and Remedial Grading

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

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil; and undocumented or otherwise deleterious fill materials. Unsuitable materials should be removed from trench bottoms and below bearing surfaces to a depth at which suitable foundation subgrade is exposed. Removals should extend a distance beyond the perimeter of the bearing surface approximately equivalent to the depth of removal below the bearing surface.

Undocumented fill was encountered during our subsurface exploration at the site to depths ranging from approximately 2 to 3½ feet below the existing grade. The undocumented fill material is an unsuitable bearing material below the proposed buildings. Recommendations for remedial grading are presented in the following section of this report.

9.1.2.1. Remedial Grading

We understand that the proposed improvements at the site will include the construction of multi-unit residential buildings that will be supported on mat or waffle slab foundations. Remedial grading will be needed to reduce the potential for reduction of bearing capacity due to liquefaction of shallow soil layers, impede manifestation of sand boils below the buildings, and to mitigate the impact of undocumented fill. The remedial grading should consist of constructing a 3-foot thick pad of cement-treated soil below the building slabs extending to 3½ feet or more below the existing grade and 5 feet beyond the footprint of the building foundations. Recommendations for cement treatment of soil are presented in Section 9.1.7 of this report. Untreated fill conforming to the criteria listed in Section 9.1.6 may be placed and compacted over the soil-cement pad in accordance with the recommendations in Section 9.1.9 to achieve finish pad elevation.

The location and extent of the remedial grading should be illustrated on the grading plans and applicable details to reduce the potential that these remedial grading recommendations are overlooked or misinterpreted during the bidding process.

9.1.3. Excavation Stabilization and Temporary Slopes

Excavations, including footing and 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). 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 conforming to the OSHA Excavation Rules and Regulations (29 CFR Part 1926) may be used to stabilize excavation sidewalls during construction. Shoring system criteria for excavations up to 20 feet in depth are listed in the OSHA Excavation Rules and Regulations (29 CFR Part 1926). The lateral earth pressures listed in Table 3 may be used to design or select the shoring system. The recommendations listed in this table are based upon the limited subsurface data provided by our exploratory borings and excavations 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 (above groundwater)	Type C	1½h:1v (34°)	80·D + 72

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

2 In layered soil, 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.

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

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 the adjacent structure at an angle of 1:1 (horizontal to vertical).

9.1.4. Construction Dewatering

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

9.1.5. Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with our recommendations in Section 9.1.3. Utility trenches should be backfilled with materials that conform to our recommendations in Section 9.1.6. Bedding materials should conform to the specifications provided in Table 4. Trench backfill should be compacted in accordance with Section 9.1.9 of this report. Trench backfill should be compacted by mechanical means. Densification of trench backfill by flooding or jetting should not be permitted.

To reduce potential for moisture intrusion into the building envelope, we recommend plugging utility trenches at locations where the trench excavations cross under the building perimeter. The trench plug should be constructed of a compacted, fine-grained, cohesive soil that fills the cross-sectional area of the trench for a distance equivalent to the depth of the excavation. Alternatively, the plug may be constructed of Controlled Low Strength Material (CLSM) conforming with the recommendations of American Concrete Institute (ACI) Committee 229 (ACI, 2012). Utility trenches cut through the soil-cement pad should be backfilled with CLSM having an unconfined compressive strength of approximately 50 to 150 pounds per square inch at 28 days when evaluated by ASTM D 4832.

9.1.6. Material Recommendations

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.

Table 4 – Recommended Material Requirements

Material and Use	Source	Requirements[1,2]
General Fill	Import	Expansion Index of 50 or less
	On-site borrow	No additional requirements[1]
Aggregate Base for pavements	Import	Class II; CSS[4] Section 26-1.02
Asphalt Concrete for pavements	Import	Type A; CSS[4] Section 39-2
Permeable Aggregate - retaining wall backdrain - capillary break gravel	Import	Open-graded, clean, compactable crushed rock or angular gravel; nominal size 3/4" or less
GeoFabric -retaining wall backdrain	Import	Non-woven filter fabric, Mirafi 140N or equivalent
Vapor Retarding Membrane	Import	10 mil, Class A plastic membrane as per ASTM E 1745
Pipe/Conduit Bedding and Pipe Zone Material - material below conduit invert to 12" above conduit	Import	90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve
Trench Backfill - above bedding material	Import or on-site borrow	Free from rock/lumps in excess of 4" diameter or 2" diameter in top 12"

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

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

3 Non-corrosive as defined by the Corrosion Guidelines version 2.0 (Caltrans, 2012).

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

9.1.7. Cement Soil Treatment

A pad of cement-treated soil should be constructed as described in Section 9.1.2.1 to improve support characteristics, impede the manifestation of sand boils below the buildings, and to mitigate the potential for a bearing capacity reduction due to liquefaction. The chemical treatment should consist of mixing cement conforming to ASTM standard C150 Type II/V or Type V with on-site soil at a rate of 5 percent by dry weight of soil.

The chemical treatment should be performed by an experienced specialty contractor. The cement for should be proportioned and spread in dry form with a mechanical spreader and mixed into the soil on a mixing table or in place to produce consistent distribution of the cement within the treated layer. The depth of mixing should not exceed 18-inches per lift or the capacity of the mixer if less. Precautions to reduce the potential for dusting of cement, such as scheduling or suspending operations to avoid windy weather, should be taken. Casting or tailgating of cement should not be permitted. The mixer should be equipped with a rotary cutting/mixing assembly, grade checker, and an automatic water distribution system.

The moisture content of the soil should not exceed the optimum moisture content of the material, as evaluated by ASTM D558, when the cement is spread and initially mixed. The subgrade should be mixed and aerated as needed to reduce the moisture content. If additional water is needed to achieve the optimum moisture content as evaluated by ASTM D558, the water should be added during a re-mixing operation after the cement has been initially mixed into the subgrade so as to reduce the potential for the formation of cement balls when water is applied. Mixing or spreading operations should not be performed during inclement weather or when the ambient temperature is less than 35 degrees Fahrenheit or during foggy or rainy weather. Adjacent passes of the mixer should overlap by 4 inches or more. The treated soil-cement subgrade should be compacted within 2 hours of initial mixing to achieve 98 percent of the referenced density as evaluated by ASTM D558.

Vehicular traffic and heavy construction equipment should not be allowed on the soil-cement pad for a 1 hour period after compaction. The soil-cement material should be maintained in a moist condition for a 7-day curing period by sprinkling water and covering the treated material with additional fill or moist straw. The finished soil cement pad should be protected from damage by construction equipment. Utility trenches cut through the soil-cement pad should be backfilled with CLSM.

9.1.8. Subgrade Preparation

Subgrade below footings, slabs, pavement, walkways or fill, should be prepared as per the recommendations in Table 5.

Table 5 – Subgrade Preparation Recommendations

Subgrade Location	Preparation Recommendations
Retaining Wall Footings and Utility Trenches	o Check for unsuitable materials as per Section 9.1.2. o Remove or compact loose/soft material. o Keep in moist condition by sprinkling water.
Below Sidewalks and Exterior Flatwork	o Check for unsuitable materials as per Section 9.1.2. o Scarify top 8” then moisture condition and compact as per Section 9.1.9. o Keep in moist condition by sprinkling water.
Below Building Slabs	o Create soil-cement pad as per Section 9.1.2.1. o Keep in moist condition by sprinkling water.
Below Fill and Pavements	o Check for unsuitable materials as per Section 9.1.2. o Scarify top 8” then moisture condition and compact as per Section 9.1.9. Omit scarification where fill is placed over cement-treated soil. o Keep in moist condition by sprinkling water.

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

9.1.9. Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 6. The allowable thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness. Heavy compaction equipment should not be used in the zone of influence behind retaining walls. The zone of influence is the region above a plane extending up and away from the heel of the wall at a slope of about 1:1 (horizontal to vertical).

Table 6 – Recommended Compaction Requirements

Fill Type	Location	Recommended Compacted Density[1]	Recommended Compacted Moisture[2]
Subgrade	Below fill, flatwork, or footings	90 percent	At or above optimum
	Below building slabs or pavement	95 percent	At or above optimum
Bedding and Pipe Zone	Material below conduit invert to 12 inches above conduit	90 percent	At or above optimum
Cement Treated Soil	Below building slabs	98 percent	At or above optimum
Trench Backfill	Below building slabs or pavement	95 percent	At or above optimum
	In locations not already specified	90 percent	At or above optimum
General Fill	Retaining wall backfill and below flatwork	90 percent	At or above optimum
	Up to 2 feet below asphalt concrete pavement section	95 percent	At or above optimum
	Below building slabs[3]	95 percent	At or above optimum
	In locations not already specified	90 percent	At or above optimum
Aggregate Base	Pavement Section	95 percent	At or above optimum
Asphalt Concrete	Pavement Section	95 percent	Not Applicable

1 Expressed as percent relative compaction or ratio of field density to reference density (typically on a dry density basis for soil and aggregate and on a wet density basis for asphalt concrete). The reference density of untreated soil and aggregate should be evaluated by ASTM D 1557. The reference density of cement treated soil should be evaluated by ASTM D 558. The reference density of asphalt concrete should be evaluated by California Test Method 304.

2 Optimum moisture should be evaluated by ASTM D 1557.

3 Placed below a plane extending down and away from the outer edges of building slabs at 1:1 angle.

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

9.1.10. Surcharge Program

Building pad areas may be surcharged to reduce the potential for future settlement due to static loading. We anticipate that a temporary embankment built to 3 feet above finish floor elevation and extending to 5 feet beyond the footprint of the building pads should significantly reduce the potential future static settlement with a 3 month surcharge duration.

9.1.11. Rainy Weather Considerations

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

In addition, construction activities performed during rainy weather may impact the stability of excavation subgrade and exposed ground. Temporary swales should be constructed to divert surface runoff away from excavations and slopes. Steep temporary slopes should be covered with plastic sheeting during significant rains. The geotechnical consultant should be consulted for recommendations to stabilize the site as needed.

9.2. Foundations

The new buildings may be supported on mat foundations. Alternatively, the buildings may be supported on ribbed foundations or waffle slabs. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in design of the structures.

9.2.1. Mat Foundations

The proposed buildings may be supported on mat foundations bearing on a 3-foot thick pad of cement-treated soil as described in Section 9.1.2.1 or on engineered fill over the 3-foot thick pad of cement treated soil. The structural engineer should design the mat slab based on the anticipated loading and intended usage using an allowable bearing capacity of 6,000 pounds per square foot (psf). This allowable bearing capacity includes a factor of safety of 3 and may be increased by one-third when considering wind or seismic loading combinations. The deflection of the mat due to applied loads may be modeled using a modulus of subgrade reaction of 35 pounds per cubic inch for an 18-inch thick mat. The buildings should be designed to accommodate a total settlement of approximately 3½ inches due to sustained building loads and dynamic settlement with a differential settlement of approximately 1¾ inches over a horizontal distance of 60 feet. The buildings should be designed to accommodate an additional 1½ inches of total settlement for 5 feet of fill to achieve the new site grades unless the building pads are surcharged as described in Section 9.1.10 with an additional differential settlement of approximately ¾ inches over a horizontal distance of 60 feet. Mat foundations should be constructed with a thickened edge extending 24 inches below the grade around the building perimeter to reduce potential for moisture changes below the foundation.

To evaluate resistance to lateral loads for mat foundations, we recommend a coefficient of friction of 0.35 and an allowable lateral bearing pressure of 300 psf per foot of depth up to 3,000 psf. Lateral bearing pressure should be neglected to a depth of 1 foot when the ground surface is not covered by pavement or slabs. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces.

The slab should be reinforced with No. 4 deformed steel bars or larger. Joints and reinforcement should be designed and detailed by the structural engineer. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of slab reinforcement. Refer to Section 9.5 for the recommended concrete cover over reinforcing steel. Slabs underlying enclosed spaces with humidity controlled environments or slabs covered by moisture sensitive floor coverings should incorporate a moisture vapor retarding system into the design. Recommendations for moisture vapor retarding systems are presented in Section 9.7.

9.2.2. Waffle Slabs

The proposed buildings may be supported on waffle slabs or ribbed foundations bearing on a 3-foot thick pad of cement-treated soil as described in Section 9.1.2.1 or on engineered fill over the 3-foot thick pad of cement treated soil. The structural engineer may use the parameters listed in Table 7 to design the waffle slab based on the anticipated loading. The buildings should be designed to accommodate a total settlement of approximately 3½ inches due to sustained building loads and dynamic settlement with a differential settlement of approximately 1¾ inches over a horizontal distance of 80 feet. The buildings should be designed to accommodate an additional 1½ inches of total settlement for 5 feet of fill to achieve the new site grades unless the building pads are surcharged as described in Section 9.1.10 with an additional differential settlement of approximately ¾ inches over a horizontal distance of 80 feet. The perimeter rib of waffle slabs should extend 24 inches below the grade around the building perimeter to reduce potential for moisture changes below the foundation.

Table 7 – Recommended Design Parameters for Waffle Slabs

Perimeter Rib Depth[1]	Allowable Bearing Capacity[2]	Modulus Subgrade Reaction	Edge Lift		Center Lift	
			Edge Moisture Variation e_m	Differential Movement y_m	Edge Moisture Variation e_m	Differential Movement y_m
24 inches	6,000 psf	65 pci	3.5 feet	0.30 inch	4.2 feet	0.13 inch

[1] Below the lowest adjacent external grade.
[2] Allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind of seismic load. Listed value includes a factor of safety of 3.

To evaluate resistance to lateral loads for mat foundations, we recommend a coefficient of friction of 0.35 and an allowable lateral bearing pressure of 300 psf per foot of depth up to 3,000 psf. Lateral bearing pressure should be neglected to a depth of 1 foot when the ground surface is not covered by pavement or slabs. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces.

The slab should be reinforced with deformed No. 4 steel bars or larger as designed by the structural engineer. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of slab reinforcement. Refer to Section 9.5 for the recommended concrete cover over reinforcing steel. Slabs underlying enclosed spaces with humidity controlled environments or slabs covered by moisture sensitive floor coverings should incorporate a moisture vapor retarding system into the design. Recommendations for moisture vapor retarding systems are presented in Section 9.7.

9.3. Retaining Walls

Minor retaining walls (wall height above footing of 6 feet or less) may be designed for active, at-rest, and passive equivalent fluid earth pressures of 40, 65, and 500 psf per foot depth for level backfill conditions. Lateral forces may be resisted by friction at the base of the wall footing and passive earth pressure acting on the embedded wall, wall footing, or wall key, if present. Passive earth pressure should be neglected to a depth of 1 foot below the ground surface when evaluating lateral load resistance where the ground surface is not covered by pavement or flatwork. Gravity and semi-gravity cantilever walls may be designed for a coefficient of friction of 0.35 to resist lateral loads and an allowable bearing capacity of 2,000 psf for a 12-inch footing width and 12 inches of embedment below the adjacent grade plus 250 psf per additional foot of width and 800 psf per additional foot of embedment up to 5,000 psf.

Walls should be designed to withstand a total settlement of ½-inch and a differential settlement of ¼-inch over a 20-foot span. We recommend that the wall and the wall footing be reinforced. Footings should be designed by the structural engineer based on the anticipated loading and usage. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of footing reinforcement. Refer to Section 9.5 for the recommended concrete cover over reinforcing steel.

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

Hydrostatic pressures may be neglected, provided that suitable drainage of the retained soil is provided. The retained soil should be drained by a weep holes or subdrain at the base of the wall stem consistent with the detail in Figure 4. Alternatively, geocomposite drain panels (Miradrain 6000XL, or similar) placed against the back of the wall may be used to supplement a smaller subdrain located near the base of the wall. Measures to reduce the rate of moisture or vapor intrusion through the wall may be advisable for walls where the discoloration resulting from moisture intrusion would be undesirable. Such measures might include use of concrete with a low water-to-cementitious-materials ratio, and/or the placement of an asphalt emulsion or 15-mil thick plastic membrane to the back surface of the wall.

9.4. Exterior Flatwork

Pedestrian sidewalks (adjacent to pavements) and walkways (removed from pavements carrying vehicular traffic) constructed of Portland cement concrete should consist of 4 inches of concrete over 6 inches of aggregate base. The concrete thickness should be increased to 6 inches at driveways. These sections presume that the subgrade is prepared in accordance with our recommendations in Section 9.1.8. Aggregate base sections for walkways and sidewalks should conform to and be compacted in accordance with our recommendations in Sections 9.1.6 and 9.1.9, respectively.

Portland cement concrete sidewalks and walkways should be appropriately jointed to reduce the random occurrence of cracks. Joints should be laid out in a square pattern at consistent intervals. Contraction, construction, and isolation joints should be detailed and constructed in accordance with the guidelines of American Concrete Institute (ACI) Committee 302 (MCP, 2012). We recommend spacing contraction joints at 8 feet, or less.

9.5. Concrete

Laboratory testing indicated that the concentration of sulfate and corresponding potential for sulfate attack on concrete is negligible for the soil tested. However, due to the variability in the on-site soil and the potential future use of reclaimed water at the site, we recommend that Type II/V or Type V cement be used for concrete structures in contact with soil. In addition, we recommend a water-to-cement ratio of not more than 0.45. A 3-inch thick or thicker concrete cover should be maintained over reinforcing steel where concrete is in contact with soil in accordance with Section 7.7 of ACI Committee 318 (ACI, 2012).

9.6. Preliminary Asphalt Concrete Pavement Design

We understand that asphalt concrete pavement sections are being considered for the proposed parking area and driveways. Laboratory testing performed during our study for the New Holland Park and Youth Center on a sample of representative near-surface soil yielded an R-value of approximately 5. The traffic index (TI) values for the paved areas have not been selected. For preliminary design purposes, we have evaluated structural pavement sections using TI values of 5 and 6 based on our experience with similar pavements on other projects. We did not evaluate the TI for the proposed pavements.

Ninyo & Moore conducted our preliminary analysis to evaluate the asphalt pavement structural section following the methodology presented in Section 600 of the Highway Design Manual (Caltrans, 2008). 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 8. Recommendations for subgrade preparation are presented in Sections 9.1.8.

Table 8 – Preliminary Asphalt Concrete Pavement Structural Sections

Traffic Index	Alternative 1	Alternative 2
	AC/AB (inches)	**Full Depth AC (inches)**
5	3.0 /10.0	7.5
6	3.5/13.0	9.0

Notes:
AC – Asphalt Concrete
AB –Aggregate Base

Subgrade soil in areas to be paved should be prepared as recommended in Sections 9.1.8 and 9.1.9 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.

9.7. Moisture Vapor Retarder

The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. We recommend that the moisture vapor retarding system consist of a 4-inch-thick capillary break, overlain by a plastic membrane 15-mil-thick. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of ¾-inch nominal size. An optional 2-inch thick blotter sand layer may be placed over the plastic membrane. The blotter sand should be in a moist but not saturated condition prior to concrete placement. If the blotter sand layer is omitted; to reduce the potential for slab curling and cracking, an appropriate concrete mix with low shrinkage characteristics and a low water-to-cementitious-materials ratio should be specified. In addition, the concrete should be delivered and placed in accordance with ASTM C94 with attention to concrete temperature and elapsed time from batching to placement, and the slab should be cured in accordance with Section 302.1, 305, or 306 of the Manual of Concrete Practice (ACI, 2012), as appropriate. The plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. The bottom of the moisture barrier system should be higher in elevation than the exterior grade, if possible. Positive drainage should be established and maintained adjacent to foundations and flatwork.

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

9.8. Seismic Design Considerations

Design of the proposed structures should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 9 presents the seismic design parameters for the site in accordance with CBC (2010) guidelines and mapped spectral acceleration parameters (USGS, 2011). The seismic design criteria provided presume that the fundamental period of the structure does not exceed ½ second.

Table 9 – 2010 California Building Code Seismic Design Criteria

Seismic Design Factors	Value
Site Class	E
Site Coefficient, F_a	0.9
Site Coefficient, F_v	2.4
Mapped Spectral Acceleration at 0.2-second Period, S_s	1.959 g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.758 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.763 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.819 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.175 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.213 g

9.9. Drainage and Site Maintenance

Positive surface drainage should be provided to divert surface water and roof runoff away from foundations or retaining walls and off site. Downspouts should be connected to a closed drainage system to discharge at a suitable location 10 feet or more away from the foundations. Runoff should be diverted by the use of swales or pipes into a collective drainage system. Surface water should not be allowed to pond adjacent to footings or retaining walls, and drainage on the site should be provided so that water is not permitted to pond.

9.10. Review of Construction Plans and Specifications

We recommend that the geotechnical consultant review the project plans and specifications to check for conformance with the intent of the recommendations in this report. The recommendations provided in this report are based on preliminary design information for the proposed construction. It should be noted that, on review of these documents, some of the recommendations presented in this report might be revised or modified to meet the project requirements.

9.11. Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions disclosed by three exploratory borings at the Ashland Family Housing site. The geotechnical consultant in the field during construction should check the interpolated subsurface conditions. During construction, the geotechnical consultant should:

- Observe removal of unsuitable materials and remedial grading.
- Observe preparation and compaction of subgrade.
- Check and test imported materials prior to use as fill.
- Observe placement and compaction of fill.
- Observe preparation and placement of cement treated soil.
- Perform field density tests to evaluate fill and subgrade compaction.
- Observe excavations for bearing materials and cleaning prior to placement of reinforcing steel and concrete.
- Observe condition of water vapor retarding system prior to concrete placement.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, we request that the selected consultant provide a letter to the city (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

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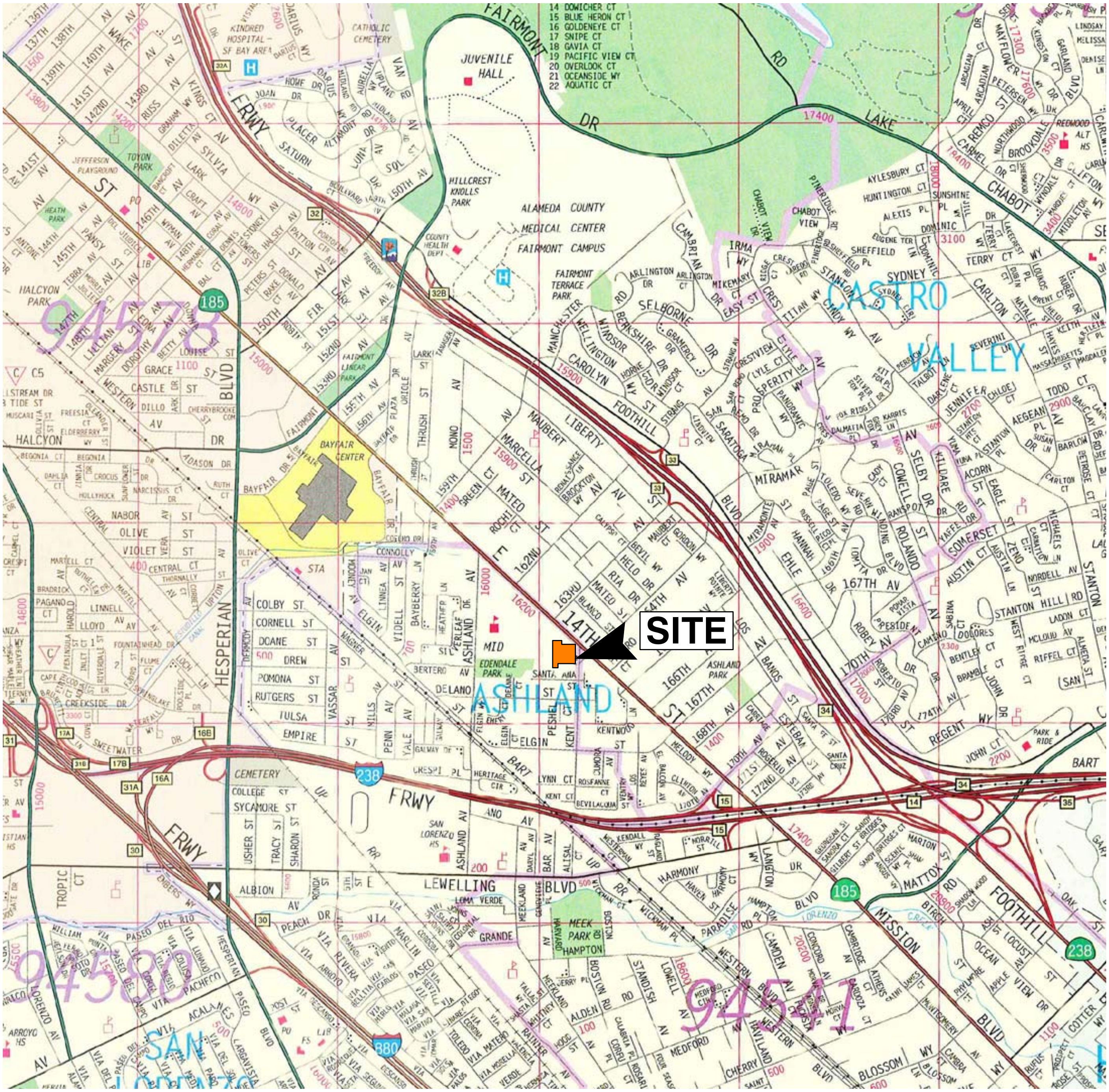

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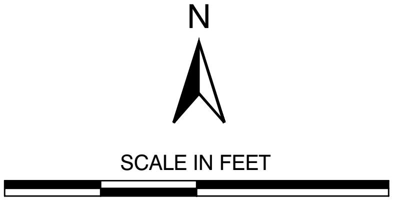


NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore		SITE LOCATION	FIGURE
PROJECT NO.	DATE	ASHLAND FAMILY HOUSING 16309, 16325, 16327, AND 16331 KENT AVENUE SAN LORENZO, CALIFORNIA	1
402090001	5/13		

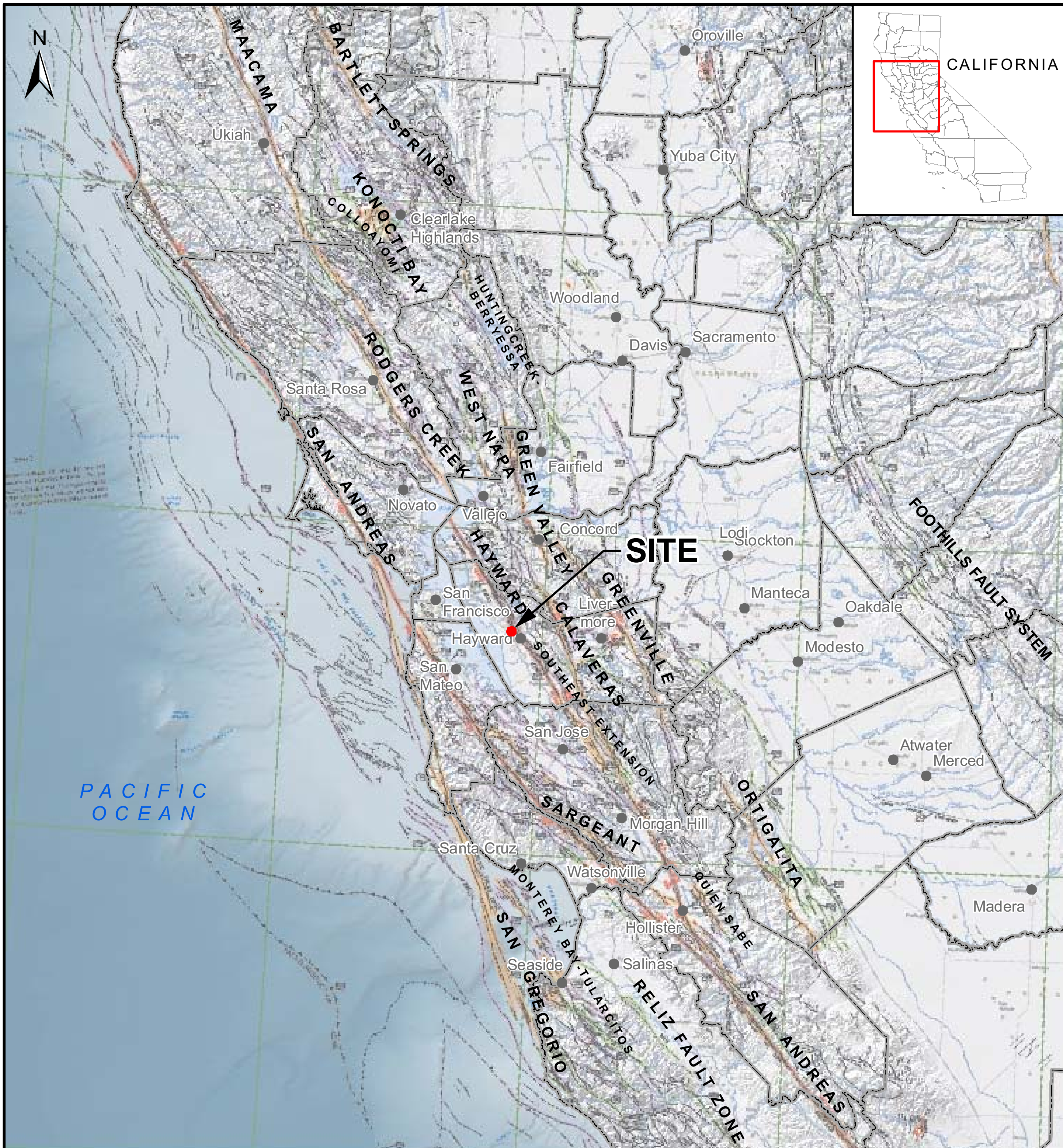
402090001-SL.dwg, May 03, 2013, 11:34am, SN

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

REFERENCE: LUK AND ASSOCIATES, SHEET C-2, JANUARY 2013.

Ninyo & Moore		BORING LOCATIONS	FIGURE
PROJECT NO.	DATE	ASHLAND FAMILY HOUSING 16309, 16325, 16327, AND 16331 KENT AVENUE SAN LORENZO, CALIFORNIA	2
402090001	5/13		

402090001-BL.dwg, May 03, 2013, 11:53am, SN



SOURCE: Fault Activity Map of California, 2010, Jennings, C.W., and Bryant, W.A., California Geological Survey.

APPROXIMATE SCALE

0 30 60 MILES

LEGEND

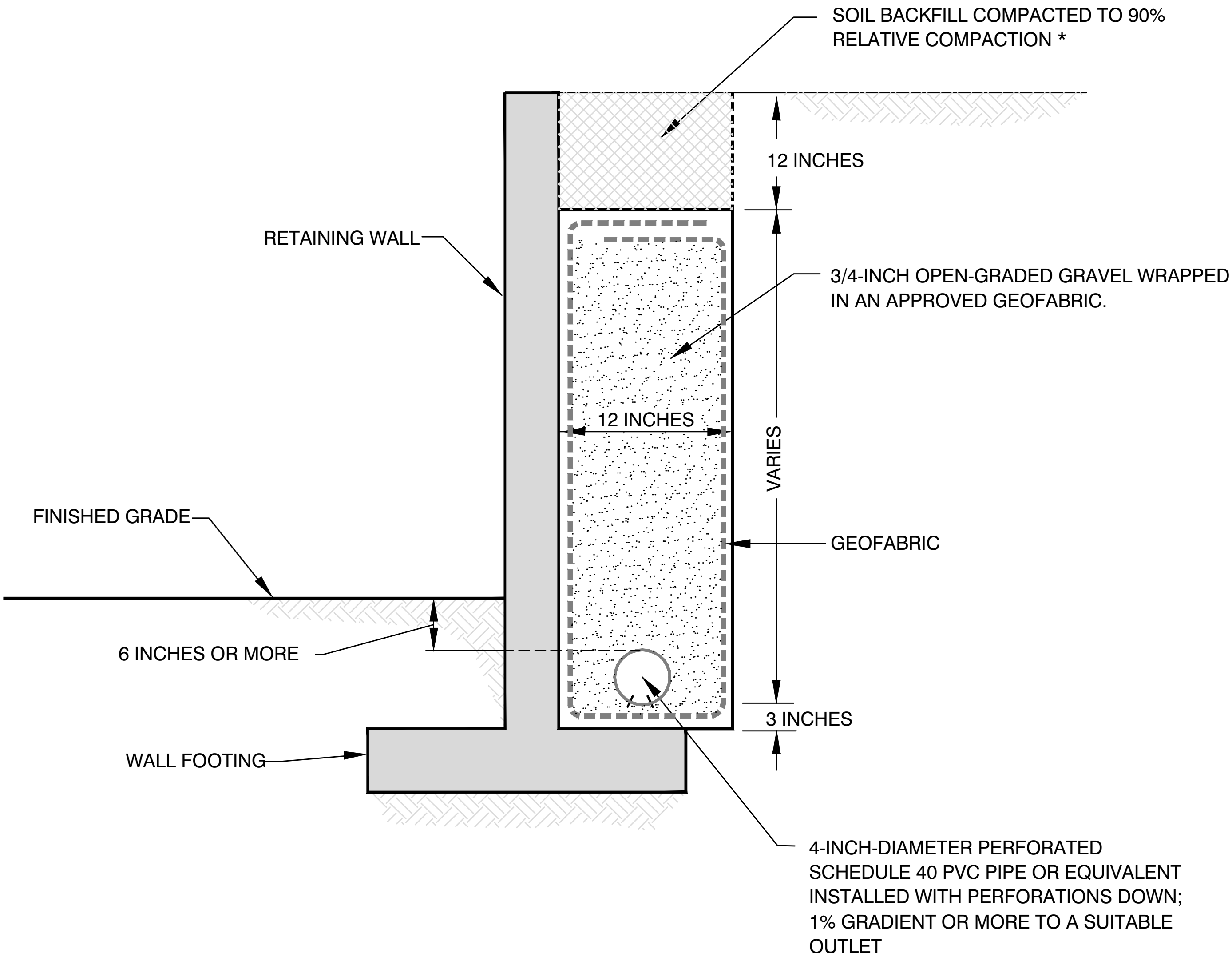
CALIFORNIA FAULT ACTIVITY

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

NOTES: ALL DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE

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

Ninyo & Moore		FAULT LOCATIONS	FIGURE
PROJECT NO.	DATE	ASHLAND FAMILY HOUSING 16309, 16325, 16327, AND 16331 KENT AVENUE SAN LORENZO, CALIFORNIA	3
402090001	5/13		



*BASED ON ASTM D1557

NOT TO SCALE

Ninyo & Moore		RETAINING WALL DRAINAGE DETAIL	FIGURE
PROJECT NUMBER	DATE	ASHLAND FAMILY HOUSING 16309, 16325, 16327, AND 16331 KENT AVENUE SAN LORENZO, CALIFORNIA	4
402090001	5/13		

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

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

Bulk Samples

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

The Standard Penetration Test (SPT) Sampler

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

Field Procedure for the Collection of Relatively Undisturbed Samples

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

The Modified Split-Barrel Drive Sampler

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

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
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.
								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
15								
20								The total depth line is a solid line that is drawn at the bottom of the boring.

Ninyo & Moore

BORING LOG

EXPLANATION OF BORING LOG SYMBOLS

PROJECT NO.	DATE	FIGURE
	Rev. 01/03	

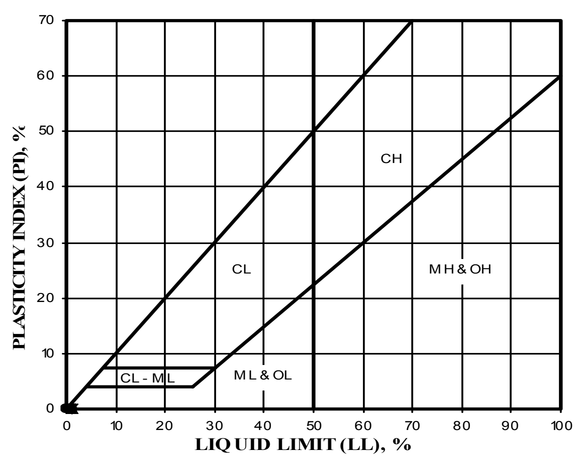
U.S.C.S. METHOD OF SOIL CLASSIFICATION

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



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

DATE DRILLED 3-8-13 BORING NO. B-1

GROUND ELEVATION 37.5' ± MSL SHEET 1 OF 3

METHOD OF DRILLING 4" Mud Rotary - Mobile B-53

DRIVE WEIGHT 140 lbs (Auto Hammer) DROP 30"

SAMPLED BY LLB LOGGED BY LLB REVIEWED BY NSD/PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
0							SP-SM	ASPHALT CONCRETE: Approximately 2 inches thick.
							CL	AGGREGATE BASE: Approximately 3 inches thick. Brown to gray, damp, medium dense, gravelly SAND with silt.
								FILL: Dark to medium brown, moist, stiff, CLAY; few gravel.
			6				CL	ALLUVIUM: Medium brown, moist, stiff, silty CLAY; trace gravel; little sand.
5			3					Black to medium brown; soft to firm.
							SP-SC	Black to medium brown, moist, very loose, SAND with clay.
			1				CL	Black to medium brown, moist, soft, silty CLAY.
			2					Medium brown.
			1					
10								
15			9					Dark gray, stiff.
								PP=2.5 tsf
20								

Ninyo & Moore

BORING LOG

ASHLAND FAMILY HOUSING - 16309, 16325, 16327 & 16331 KENT AVENUE
SAN LORENZO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402090001	5/13	A-1

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DATE DRILLED 3-8-13 BORING NO. B-1 GROUND ELEVATION 37.5' ± MSL SHEET 2 OF 3 METHOD OF DRILLING 4" Mud Rotary - Mobile B-53 DRIVE WEIGHT 140 lbs (Auto Hammer) DROP 30" SAMPLED BY LLB LOGGED BY LLB REVIEWED BY NSD/PCC **DESCRIPTION/INTERPRETATION**
20			3				CL	ALLUVIUM: (continued) Yellowish brown, moist, soft to firm, silty CLAY; few sand; trace gravel.
25			7	31.4	91.5			Stiff.
30			3					Light brown, soft to firm; little to some sand.
35			10	27.6	94.5			Olive brown, stiff; few sand. PP=1.25 tsf
40								

Ninyo & Moore

BORING LOG

ASHLAND FAMILY HOUSING - 16309, 16325, 16327 & 16331 KENT AVENUE
SAN LORENZO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402090001	5/13	A-2

DATE DRILLED 3-8-13 BORING NO. B-1

GROUND ELEVATION 37.5' ± MSL SHEET 3 OF 3

METHOD OF DRILLING 4" Mud Rotary - Mobile B-53

DRIVE WEIGHT 140 lbs (Auto Hammer) DROP 30"

SAMPLED BY LLB LOGGED BY LLB REVIEWED BY NSD/PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
40			2				CL	ALLUVIUM: (continued) Olive brown, moist, soft, silty CLAY; little sand.
45			13	18.8	101.4			Greenish gray, stiff to very stiff. PP=2.0 tsf
50			7					Medium brown, stiff; few sand.
55								Total Depth = 51.5 feet. The depth to groundwater was not evaluated due to the use of drilling fluid for the mud rotary borings. Backfilled with Portland cement grout on 3-8-13. PP=Unconfined compressive strength in tons per square foot (tsf) as evaluated by pocket penetrometer.
60								

Ninyo & Moore

BORING LOG

ASHLAND FAMILY HOUSING - 16309, 16325, 16327 & 16331 KENT AVENUE
SAN LORENZO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402090001	5/13	A-3

DATE DRILLED	3-8-13	BORING NO.	B-2
GROUND ELEVATION	39.75' ± MSL	SHEET	1 OF 3
METHOD OF DRILLING	4" Mud Rotary - Mobile B-53		
DRIVE WEIGHT	140 lbs (Auto Hammer)	DROP	30"
SAMPLED BY	LLB	LOGGED BY	LLB
REVIEWED BY	NSD/PCC		

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 1.75 inches thick.
						SP-SM	AGGREGATE BASE: Approximately 2.5 inches thick. Brown to gray, damp, medium dense, gravelly SAND with silt.
						CL	FILL: Dark brown, moist, stiff, CLAY; trace gravel.
						SC-SM	ALLUVIUM: Dark to medium brown, moist, loose, silty clayey SAND; trace gravel.
		8	18.6	104.3			PP=1.25 tsf
5		2					Very loose.
		4					Loose.
						SM	Brown, moist to wet, very loose, silty SAND; some gravel.
		1					
		6				CL	Dark brown, moist, stiff, silty CLAY; few sand; scattered organics.
10							
							Medium brown.
15		10					
20							

Ninyo & Moore

BORING LOG

ASHLAND FAMILY HOUSING - 16309, 16325, 16327 & 16331 KENT AVENUE
SAN LORENZO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402090001	5/13	A-4

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
20		■	19				CL	ALLUVIUM:(continued) Yellowish brown, moist, very stiff, silty CLAY; trace sand. PP=2.75 tsf
25		◣	3					Light brown, soft to firm.
30		■	12					Medium brown, stiff.
							SC	Medium brown, moist to wet, loose, clayey SAND.
							CL	Medium brown, moist, soft to firm, silty CLAY.
35		◣	3					
40								

DATE DRILLED 3-8-13 BORING NO. B-2

GROUND ELEVATION 39.75' ± MSL SHEET 2 OF 3

METHOD OF DRILLING 4" Mud Rotary - Mobile B-53

DRIVE WEIGHT 140 lbs (Auto Hammer) DROP 30"

SAMPLED BY LLB LOGGED BY LLB REVIEWED BY NSD/PCC

Ninyo & Moore

BORING LOG

ASHLAND FAMILY HOUSING - 16309, 16325, 16327 & 16331 KENT AVENUE
SAN LORENZO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402090001	5/13	A-5

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
								DATE DRILLED 3-8-13 BORING NO. B-2 GROUND ELEVATION 39.75' ± MSL SHEET 3 OF 3 METHOD OF DRILLING 4" Mud Rotary - Mobile B-53 DRIVE WEIGHT 140 lbs (Auto Hammer) DROP 30" SAMPLED BY LLB LOGGED BY LLB REVIEWED BY NSD/PCC
40			10				CL	ALLUVIUM:(continued) Light yellowish brown, moist, stiff, silty CLAY; little sand. PP=1.0 tsf
45			6					Light olive brown.
50			32					Olive brown, hard; little sand.
								Total Depth = 51.5 feet. The depth to groundwater was not evaluated due to the use of drilling fluid for the mud rotary borings. Backfilled with Portland cement concrete on 3-8-13. PP=Unconfined compressive strength in tons per square foot (tsf) as evaluated by pocket penetrometer.
55								
60								

- DATE DRILLED: 3-8-13
- BORING NO.: B-3
- GROUND ELEVATION: 38.25' ± MSL
- SHEET 1 OF 1
- METHOD OF DRILLING: 8" Hollow Stem Mobile B-53
- DRIVE WEIGHT: 140 lbs (Auto Hammer)
- DROP: 30"
- SAMPLED BY: LLB
- LOGGED BY: LLB
- REVIEWED BY: NSD/PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 2 inches thick.
						SP-SM	AGGREGATE BASE: Approximately 3 inches thick. Brown to gray, damp, medium dense, gravelly SAND with silt.
	Bulk, Driven	5				CL	FILL: Dark brown, moist, firm to stiff, CLAY; few sand; trace gravel.
	Driven	9					
						CL	ALLUVIUM: Medium brown, moist, stiff, sandy CLAY; some sand.
5	Driven	1					Soft, little sand.
			▼				Saturated.
	Driven	6					Stiff.
10							Total Depth = 10 feet.
							Groundwater was measured at a depth of approximately 7 feet in borehole at about 1 hour of completion of drilling. Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report.
							Backfilled with Portland cement concrete on 3-8-13.
15							
20							

Ninyo & Moore

BORING LOG

ASHLAND FAMILY HOUSING - 16309, 16325, 16327 & 16331 KENT AVENUE
SAN LORENZO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
402090001	5/13	A-7

APPENDIX B

LABORATORY TESTING

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

In-Place Moisture 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.

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-2.

Expansion Index Test
The expansion index of a selected material was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of the test are presented on Figure B-3.

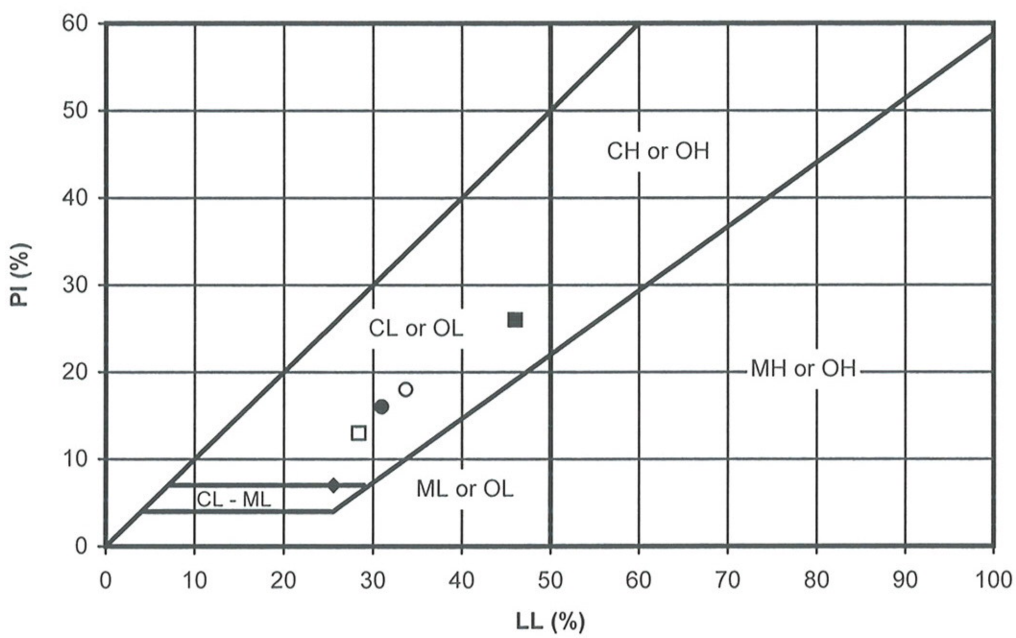
Unconfined Compression Tests
Unconfined compression tests were performed on relatively undisturbed samples in general accordance with ASTM D 2166. The test results are shown on Figure B-4.

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	5.5 - 6.0	Black to medium brown SAND with clay	100	12	SC
B-1	8.5 - 10.0	Black to medium brown silty CLAY	100	75	CL
B-2	4.0 - 5.5	Dark to medium brown silty clayey SAND	97	41	SC-SM
B-2	7.0 - 8.5	Brown silty SAND; some gravel	67	16	SM
B-3	4.0 - 5.5	Medium brown sandy CLAY	100	54	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT NO.	DATE	ASHLAND FAMILY HOUSING 16309, 16325, 16327, AND 16331 KENT AVENUE SAN LORENZO, CALIFORNIA	B-1
402090001	5/13		

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	5.5 - 7.0	31	15	16	CL	CL
■	B-1	8.5 - 10.0	46	20	26	CL	CL
◆	B-2	4.0 - 5.5	26	19	7	CL-ML	SC-SM
○	B-2	8.5 - 10.0	34	16	18	CL	CL
□	B-3	4.0 - 5.5	28	15	13	CL	CL



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Ninyo & Moore		ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO.	DATE	ASHLAND FAMILY HOUSING 16309, 16325, 16327, AND 16331 KENT AVENUE SAN LORENZO, CALIFORNIA	B-2
402090001	5/13		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-3	0.5 - 5.0	13.8	94.5	23.4	0.045	44	Low

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

Ninyo & Moore

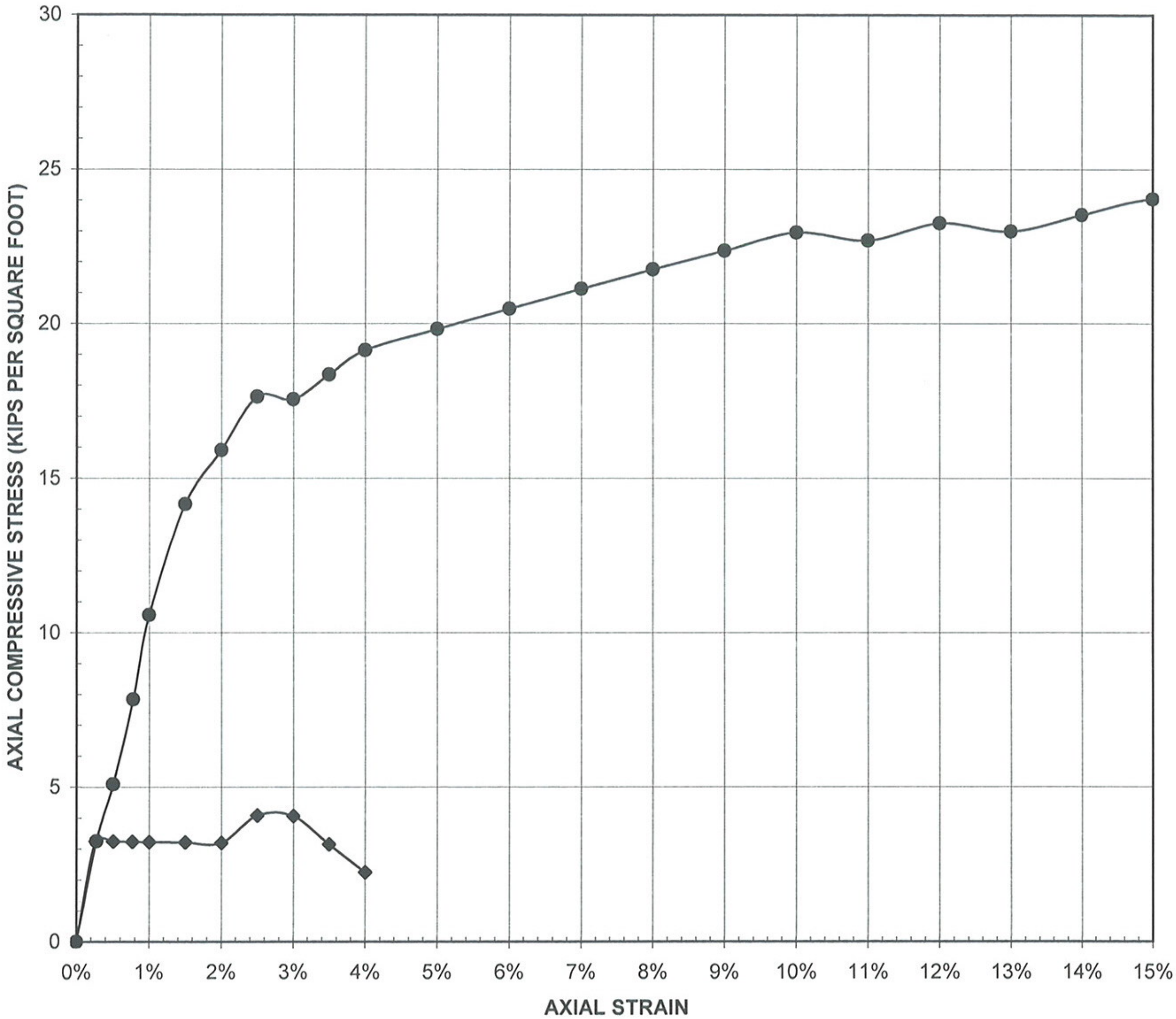
EXPANSION INDEX TEST RESULTS

PROJECT NO.	DATE
402090001	5/13

ASHLAND FAMILY HOUSING
16309, 16325, 16327, AND 16331 KENT AVENUE
SAN LORENZO, CALIFORNIA

FIGURE
B-3

402090001 B-3.xls



SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	STRAIN RATE (%/min.)	UNDRAINED SHEAR STR s_u, (ksf)
◆	Medium brown silty CLAY	CL	B-1	8.0 - 8.5	30.0	89.6	1.00	2.04
●	Dark gray silty CLAY	CL	B-1	16.0 - 16.5	23.6	99.0	1.00	12.02

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2166

Ninyo & Moore	UNCONFINED COMPRESSION RESULTS	FIGURE
PROJECT NO. / DATE	ASHLAND FAMILY HOUSING 16309, 16325, 16327, AND 16331 KENT AVENUE SAN LORENZO, CALIFORNIA	B-4
402090001 / 5/13		

APPENDIX C

LIQUEFACTION AND DYNAMIC SETTLEMENT CALCULATIONS

DYNAMIC SETTLEMENT WORKSHEET

JOB NO.: 402090001 **JOB NAME:** RCD/Ashland Family Housing
CALCULATION BY: NSD **DATE:** 4/29/2013
CHECKED BY: PCC **DATE:** 4/30/2013
BORING/PROFILE: B-1

Depth (z) of Layer Top	Bottom	Formation	Soil Type	Thick. t (ft.)	Midpoint of Layer (ft.)	Layer γ (pcf)	Total Stress σ_v (ksf)	Effect. Stress σ_v' (ksf)	$(N_1)_{60}$	Fines Content FC (%)	α Note 1	β Note 2	$(N_1)_{60cs}$ Note 3	$CRR_{7.5}$ Note 4	r_d Note 5	CSR_M Note 6	FOS_{liq} Note 8	$CSR_{7.5}$ Note 9	ε_v (%) Fig. 4-3	Sat. Sand Settlemnt ΔH_{sat} (in) Note 10	τ_{avg} (tsf)	mean effective stress σ_m' (tsf)	max Shear Modulus G_{max} (tsf)	a	b	γ (%)	ε_{15} (%)	ε_{Nc} (%)	dry sand settlement ρ_e (in.)
0	2.5	Fill	CL	2.5	1.25	120	0.15	0.15	10	50	5	1.2	17	0.18083	0.997	0.305		0.228	2.6		0.02	0.05	226						
2.5	5.5	Alluvium	CL	3	4	125	0.49	0.49	7	50	5	1	13	0.14436	0.991	0.303		0.227	3.2		0.07	0.16	362						
5.5	6	Alluvium	SP-SC	0.5	5.75	125	0.71	0.66	7	12	2	1	9	0.10248	0.987	0.323	0.42	0.242	3.2	0.19	0.11	0.21	421						
6	51.5	Alluvium	CL	45.5	28.75	125	3.58	2.10	9	50	5	1	16	0.1682	0.933	0.486		0.365	2.8		0.51	0.68	817						
																			Total Settlement =	0.19									0.00

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

NOTES:

1. fines content correction factor α = 0 for FC<=5%; exp[1.76-(190/FC^2)] for 5%<FC<35%; 5.0 for FC>=35%
2. fines content correction factor β = 1.0 for FC<=5%; [0.99+(FC$^{1.5}$/1000)] for 5%<FC<35%; 1.2 for FC>=35%
3. clean sand blowcounts @ 1 tsf overburden @ 60% energy ratio, $(N_1)_{60cs} = \alpha+\beta^*(N_1)_{60}$
4. cyclic resistance ratio @ M=7.5, $CRR_{7.5} = 1/[34-(N_1)_{60cs}] + (N_1)_{60cs}/135 + 50/[10^*(N_1)_{60cs}+45]^2 - 1/200$ for $(N_1)_{60cs}$<30 else nonliquefiable
5. stress reduction factor, r_d, = 1.0-0.007652*z for z<=9.15m; 1.174-0.0267*z for 9.15m<z<23m
6. cyclic stress ratio @ M, $CSR_M = \tau_{avg}/\sigma_v' = 0.65(a_{max}/g)(\sigma_v)(r_d)/(\sigma_v')$
7. magnitude scaling factor, $MSF = 10^{2.24}/M^{2.56}$ ==> MSF = 1.33
8. factor of safety against liquefaction, $FOS_{liq} = (CRR_{7.5}/CSR_M)MSF$
9. cyclic stress ratio @ M=7.5, $CSR_{7.5} = CSR_M/MSF$
10. settlement of saturated sand, $\Delta H_{sat} = \varepsilon_v{}^*t$
11. Coefficient of Lateral Earth Pressure at Rest, Ko: Ko = 0.47
12. Number of Strain Cycles, Nc: Nc = 8.63097

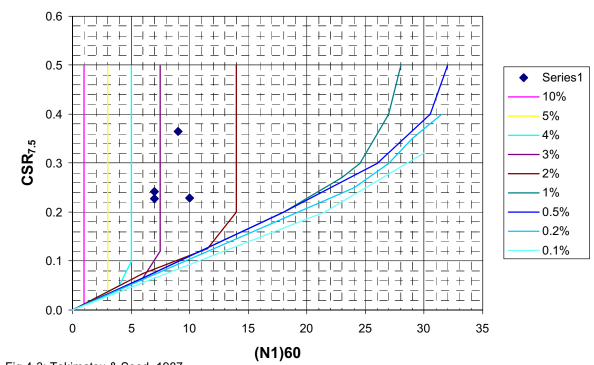


Fig 4-3: Tokimatsu & Seed, 1987

REFERENCES:

Youd, T.L. & Idriss, I.M., 2001, Summary Report from the 1996 NCEER and 1998 NCEEF/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 10
Tokimatsu, K. & Seed, H.B., 1987, Evaluation of Settlements in Sand Due to Earthquake Shaking, Journal of Geotechnical Engineering Division, ASCE, Vol 113, No 8.
Pradel, D.J., 1998, Procedure to Evaluate Earthquake Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol 124, No. 4.

DYNAMIC SETTLEMENT WORKSHEET

JOB NO.: 402090001 **JOB NAME:** RCD/Ashland Family Housing
CALCULATION BY: NSD **DATE:** 4/29/2013
CHECKED BY: PCC **DATE:** 4/30/2013
BORING/PROFILE: B-2

Depth (z) of Layer: Top	Depth (z) of Layer: Bottom	Formation	Soil Type	Thick. t (ft.)	Midpoint of Layer (ft.)	Layer γ (pcf)	Total Stress σ_v (ksf)	Effect. Stress σ_v' (ksf)	$(N_1)_{60}$	Fines Content FC (%)	α Note 1	β Note 2	$(N_1)_{60cs}$ Note 3	$CRR_{7.5}$ Note 4	r_d Note 5	CSR_M Note 6	FOS_{liq} Note 8
0	2	Fill	CL	2	1	120	0.12	0.12	7	50	5	1.2	13	0.14436	0.998	0.305	
2	5	Alluvium	SC-SM	3	3.5	125	0.43	0.43	4	41	5	1	10	0.11136	0.992	0.303	
5	7	Alluvium	SC-SM	2	6	125	0.74	0.68	8	41	5	1	15	0.15607	0.986	0.329	0.63
7	9	Alluvium	SM	2	8	125	0.99	0.80	8	16	3	1	11	0.12384	0.981	0.370	0.45
9	31	Alluvium	CL	22	20	125	2.49	1.55	12	50	5	1	19	0.20807	0.954	0.467	
31	33	Alluvium	SC	2	32	125	3.99	2.31	10	15	2	1	13	0.14035	0.915	0.484	0.39
33	51.5	Alluvium	CL	18.5	42.25	125	5.27	2.95	10	50	5	1	17	0.18083	0.832	0.455	

Depth Top	$CSR_{7.5}$ Note 9	ε_v (%) Fig. 4-3	Sat. Sand Settlemnt ΔH_{sat} (in) Note 10	τ_{avg} (tsf)	mean effective stress σ_m' (tsf)	max Shear Modulus G_{max} (tsf)	a	b	γ (%)	ε_{15} (%)	ε_{Nc} (%)	dry sand settlement ρ_e (in.)
0	0.228	3.2		0.02	0.04	180						
2	0.227	4.6		0.06	0.14	282	0.129377	20981	0.349516	2.411189	1.880256	1.35
5	0.247	2.9	0.70	0.11	0.22	447						
7	0.277	2.9	0.70	0.15	0.26	486						
9	0.350	2.3		0.36	0.50	774						
31	0.363	2.6	0.63	0.56	0.75	887						
33	0.341	2.6		0.67	0.95	1003						
Total Settlement =			2.03									1.35

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

NOTES:

1. fines content correction factor α = 0 for FC<=5%; exp[1.76-(190/FC^2)] for 5%<FC<35%; 5.0 for FC>=35%
2. fines content correction factor β = 1.0 for FC<=5%; [0.99+(FC$^{1.5}$/1000)] for 5%<FC<35%; 1.2 for FC>=35%
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5. stress reduction factor, r_d, = 1.0-0.007652*z for z<=9.15m; 1.174-0.0267*z for 9.15m<z<23m
6. cyclic stress ratio @ M, $CSR_M = \tau_{avg}/\sigma_{\varpi}' = 0.65(a_{max}/g)(\sigma_v)(r_d)/(\sigma_v')$
7. magnitude scaling factor, $MSF = 10^{2.24}/M^{2.56}$ ==> MSF = 1.33
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9. cyclic stress ratio @ M=7.5, $CSR_{7.5} = CSR_M/MSF$
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12. Number of Strain Cycles, Nc: Nc = 8.63097

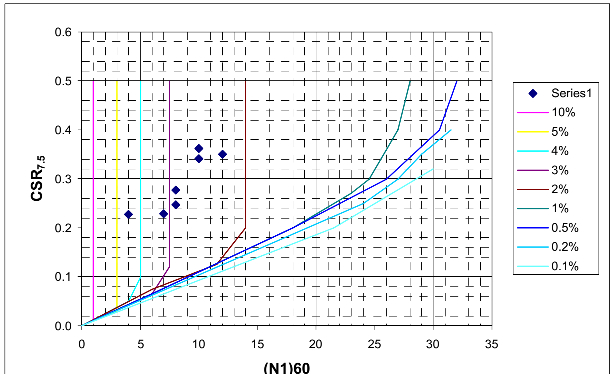

Fig 4-3: Tokimatsu & Seed, 1987

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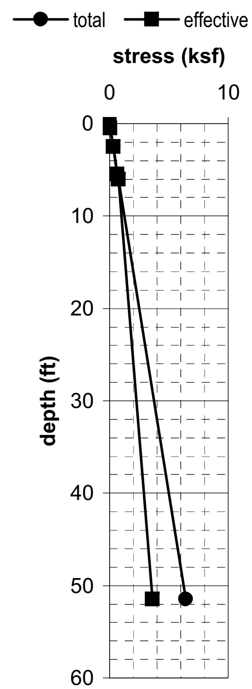
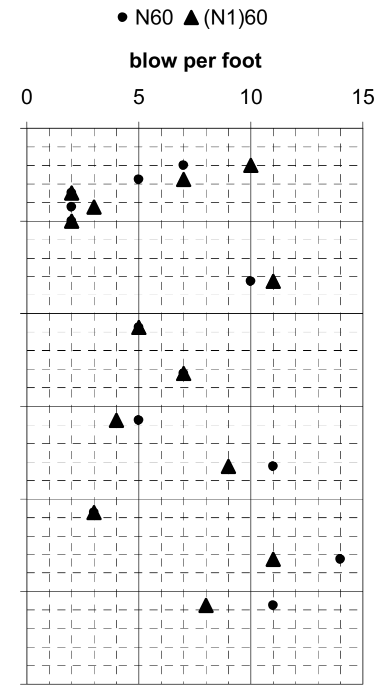
Pradel, D.J., 1998, Procedure to Evaluate Earthquake Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol 124, No. 4.

Project Name: RCD/Ashland Family Housing **Project No.:** 402090001
Calculation By: NSD **Date:** 4/29/13
Checked By: PCC **Date:** 4/30/13

Profile/Boring: B-1 **Depth to Groundwater (ft):** 7

Formation	USCS Class.	depth to top of layer (ft)	depth to base of layer (ft)	layer thickness (ft)	Wet Unit Weight (pcf)	tot. vert. stress at top (psf)	tot. vert. stress at base (psf)
Fill	SP-SM	0	0.5	0.5	120	0	60
Fill	CL	0.5	2.5	2	120	60	300
Alluvium	CL	2.5	5.5	3	125	300	675
Alluvium	SP-SC	5.5	6	0.5	125	675	738
Alluvium	CL	6	51.5	45.5	125	738	6425
		#N/A		#N/A		#N/A	#N/A
		#N/A		#N/A		#N/A	#N/A
		#N/A		#N/A		#N/A	#N/A
		#N/A		#N/A		#N/A	#N/A
		#N/A		#N/A		#N/A	#N/A





hammer type/method: auto-trip hammer **energy ratio correction, Cer:** 1.50
borehole diameter (in): 4 **borehole diameter correction, Cb:** 1.00

sample depth (ft)	tot. vert. stress at depth (psf)	eff. vert. stress at depth (psf)	blowcnt N, (bpf)	cohesive? yes=1	sampler type?	equiv. SPT blowcnt Nspt, (bpf)	drill rod length	drill rod length correction Cr	N60 (bpf)	overbrdn correction Cn, (bpf)	N1 (bpf)	(N1)60
4	488	488	6	1	0	6	5	0.75	7	1.54	9	10
5.5	675	675	3	0	0	3	10	1.00	5	1.45	4	7
7	863	863	1	1	0	1	10	1.00	2	1.37	1	2
8.5	1050	956	2	1	1	1	10	1.00	2	1.33	2	3
10	1238	1050	1	1	0	1	10	1.00	2	1.30	1	2
16.5	2050	1457	9	1	1	6	20	1.00	10	1.16	7	11
21.5	2675	1770	3	1	0	3	25	1.00	5	1.08	3	5
26.5	3300	2083	7	1	1	5	30	1.00	7	1.01	5	7
31.5	3925	2396	3	1	0	3	35	1.00	5	0.94	3	4
36.5	4550	2709	10	1	1	7	40	1.00	11	0.89	6	9
41.5	5175	3022	2	1	0	2	45	1.00	3	0.84	2	3
46.5	5800	3335	13	1	1	9	50	1.00	14	0.79	7	11
51.5	6425	3648	7	1	0	7	55	1.00	11	0.75	5	8

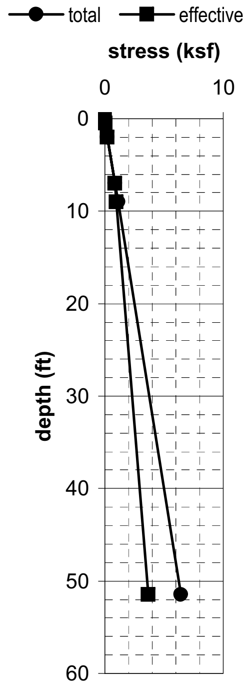
Project Name: RCD/Ashland Family Housing
Calculation By: NSD
Checked By: PCC

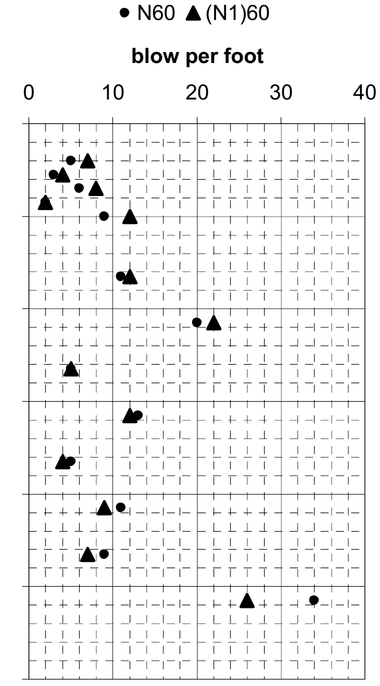
Project No.: 402090001
Date: 4/29/13
Date: 4/30/13

Profile/Boring: B-2

Depth to Groundwater (ft): 7

Formation	USCS Class.	depth to top of layer (ft)	depth to base of layer (ft)	layer thickness (ft)	Wet Unit Weight (pcf)	tot. vert. stress at top (psf)	tot. vert. stress at base (psf)
Fill	SP-SM	0	0.5	0.5	120	0	60
Fill	CL	0.5	2	1.5	120	60	240
Alluvium	SC-SM	2	7	5	125	240	865
Alluvium	SM	7	9	2	125	865	1115
Alluvium	CL	9	51.5	42.5	125	1115	6428
		#N/A		#N/A		#N/A	#N/A
		#N/A		#N/A		#N/A	#N/A
		#N/A		#N/A		#N/A	#N/A
		#N/A		#N/A		#N/A	#N/A
		#N/A		#N/A		#N/A	#N/A





hammer type/method: auto-trip hammer
borehole diameter (in): 4

energy ratio correction, Cer: 1.50
borehole diameter correction, Cb: 1.00

sample depth (ft)	tot. vert. stress at depth (psf)	eff. vert. stress at depth (psf)	blowcnt N, (bpf)	cohesive? yes=1	sampler type?	equiv. SPT blowcnt Nspt, (bpf)	drill rod length	drill rod length correction Cr	N60 (bpf)	overbrdn correction Cn, (bpf)	N1 (bpf)	(N1)60
4	490	490	8	0	1	4	5	0.75	5	1.54	6	7
5.5	678	678	2	0	0	2	10	1.00	3	1.45	3	4
7	865	865	4	0	0	4	10	1.00	6	1.37	5	8
8.5	1053	959	1	0	0	1	10	1.00	2	1.33	1	2
10	1240	1053	6	1	0	6	10	1.00	9	1.30	8	12
16.5	2053	1460	10	1	1	7	20	1.00	11	1.16	8	12
21.5	2678	1773	19	1	1	13	25	1.00	20	1.08	15	22
26.5	3303	2086	3	1	0	3	30	1.00	5	1.01	3	5
31.5	3928	2399	12	1	1	8	35	1.00	13	0.94	8	12
36.5	4553	2712	3	1	0	3	40	1.00	5	0.89	3	4
41.5	5178	3025	10	1	1	7	45	1.00	11	0.84	6	9
46.5	5803	3338	6	1	0	6	50	1.00	9	0.79	5	7
51.5	6428	3651	32	1	1	23	55	1.00	34	0.75	17	26