

**GEOTECHNICAL INVESTIGATION
PROPOSED 10-ACRE RESIDENTIAL DEVELOPMENT
TRACT 20334, EAST OF EAST AVENUE APPROXIMATELY
300 FEET SOUTH OF BANYAN STREET
CITY OF RANCHO CUCAMONGA, CALIFORNIA**

Prepared for:

MANNING HOMES

20151 SW Birch Street, Suite 150
Newport Beach, California 92660

Project No. 12696.002

June 10, 2020
(Updated February 22, 2021)



Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY



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Attention: Mr. Craig Kozma
Vice President Development

Subject: Geotechnical Investigation, Proposed 10-Acre Residential Development,
Tract 20334, East of East Avenue Approximately 300 Feet South of Banyan
Street, City of Rancho Cucamonga, California

In response to Manning Homes' request and authorization, Leighton and Associates, Inc. (Leighton) has conducted a geotechnical investigation for the proposed residential development to be located in Tract 20334 in the City of Rancho Cucamonga, California. The 10-acre site is located east of East Avenue and approximately 300 feet south of Banyan Street. The purpose of this study has been to evaluate the geotechnical conditions at the site with respect to the proposed development and to provide geotechnical recommendations for design and construction. This updated report contains several clerical and project description modifications; however, the findings, conclusions, and recommendations from the original report dated June 10, 2020, have not been changed by this report update.

Based on this study, the proposed residential development is feasible from a geotechnical standpoint provided our recommendations are incorporated into the design and construction of the project. The most significant geotechnical issues with respect to the project are those related to the potential for strong seismic shaking and the presence of potentially compressible soil. These and other geotechnical issues are

presented in this report which provides our findings, conclusions, and preliminary geotechnical recommendations for the project.

We appreciate the opportunity to work with Manning Homes on the development of this project. If Manning Homes has any questions regarding this report, please call us.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.



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Attachment: GBA Important Information About this Geotechnical Engineering Report

Figures (Rear of Text)

Figure 1 - Site Location Map
 Figure 2 - Regional Geology Map
 Figure 3 - Exploration Location Map
 Figure 4 - Retaining Wall Backfill and Subdrain Detail

Appendices

Appendix A - References
 Appendix B - Test Pit and Infiltration Test Logs
 Appendix C - Laboratory Test Results
 Appendix D - Summary of Secondary Seismic Hazard Analysis
 Appendix E - General Earthwork and Grading Specifications

1.0 INTRODUCTION

1.1 Site Location and Description

The project site is located east of East Avenue and approximately 300 feet south of Banyan Street in the City of Rancho Cucamonga, San Bernardino County, California. The site location (latitude 34.1418°, longitude -117.5123°), and immediate vicinity are shown on Figure 1, Site Location Map. The project area is approximately 10 acres. Based on review of aerial photographs, the site has been vacant or used for agriculture since at least 1938.

Review of 40-scale *Tentative Tract Map, Tentative Tract No. 20334, Residential Lots 1-17*, (Sheet C-2 of 11) prepared by Madole & Associates, dated January, 2021, indicates the site is gently sloping with approximate elevations across the site ranging from 1,490 feet above mean sea level (msl) in the northwestern corner of the site to 1,459 feet above msl in the southeastern corner of the site. Sheet flow across the site is generally directed to the east-southeast.

1.2 Previous Geotechnical Reports

Previous exploration was performed by Associated Soils Engineering, Inc. (ASE) on February 11, 2020 (ASE, 2020). Seven exploratory borings were drilled, sampled, and logged to depths ranging from 5 feet to 26 feet below the ground surface (bgs). ASE observed loose to very dense young alluvial fan deposits covering the site to depths extending to their deepest boring.

ASE also performed percolation testing in the southeastern portion of the site, where infiltration systems are currently proposed. Based on their measurements and applying a factor of safety of 2, ASE reported converted infiltration rates of 1.2 and 5.8 in/hr.

In their February 28, 2020 report, ASE stated that the construction of the residential development would be geotechnically feasible provided that their reported recommendations and criteria were incorporated into the design and implemented during construction.

1.3 Proposed Development

The Tentative Tract Map 17 single-family residential lots with associated street, sidewalks, sewer and water lines. Four open-space areas, Lots A through D, are designated for the tract, one at each corner of the site, adjacent to both East Avenue and Golden Lock Place. Lot C (southeast corner) is designated for buried infiltration chambers. A V-ditch is proposed adjacent to the north property line, and an equestrian trail is proposed adjacent to the south property line. Vehicle entries will be off of both East Avenue and Golden Lock Place.

Based on the Conceptual Grading Plan by Madole (January, 2021), proposed grading consists of cut and fill on each lot, except Lot A, which is planned for only cut. Both design cuts and fills are up to approximately 5 feet deep.

1.4 Purpose of Investigation

The purpose of this study has been to evaluate the general geotechnical conditions at the site with respect to the proposed development and to provide preliminary geotechnical recommendations for design and construction.

1.5 Scope of Work

Our geotechnical study included subsurface exploration, infiltration testing, laboratory testing, and geotechnical analysis to evaluate existing conditions and to develop conclusions and preliminary geotechnical recommendations for design and construction of the proposed development. Our scope of work included the following tasks:

- Background Review: We reviewed readily available geotechnical reports, literature, aerial photographs, and maps relevant to the site available from our in-house library, in the public domain, or provided by Manning Homes. We evaluated geological hazards and potential geotechnical issues that may significantly impact the site. The documents reviewed are listed in Appendix A, References.
- Utility Coordination: We contacted Underground Services Alert (USA) prior to excavating test pits so that utility companies could mark utilities onsite. We also coordinated site access with Manning Homes.

- Field Exploration: Our field exploration consisted of 11 backhoe test pits (designated TP-1 through TP-11) excavated to approximate depths ranging from 4.5 to 16 feet bgs. The approximate locations of the test pits are shown on Figure 3, *Test Pit Location Map*. Field tests were performed at selected locations and depths using a nuclear gauge to evaluate the onsite soils density and moisture content. Representative bulk samples were collected from selected test pits and transported to our laboratory for testing. Each test pit was photographed and logged by a geologist from our firm, then backfilled (but not compacted) with the previously excavated earth materials. Logs of the test pits presenting the encountered soil conditions are included in Appendix B.
- Infiltration Testing: We performed two infiltration tests in the area of proposed infiltration systems in the southeastern portion of the site. Two test pits excavated during our subsurface exploration (TP-7 and TP-8) were advanced to depths of approximately 9 to 10 feet bgs prior to infiltration testing. Infiltration/percolation tests were conducted at the bottom of the two test pits within roughly 17.5-cubic-foot excavations. We performed constant head, open pit infiltration tests within the two pits. We will obtained water from a nearby water meter that Manning Homes provided. After testing, TP-7 was excavated to a depth of approximately 16 feet bgs to observe the soil and moisture conditions below the test elevation. The test pits were subsequently backfilled with uncompacted soil cuttings.
- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. This laboratory testing program was designed to evaluate engineering characteristics of site soils. Laboratory tests conducted during this investigation include:
 - Maximum dry density, optimum moisture content
 - Expansion index
 - Consolidation
 - Collapse
 - Grain Size Distribution
 - Water-soluble sulfate concentration
 - Resistivity, chloride content and pH

- Engineering Analysis: Data obtained from our background review, field exploration and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide preliminary recommendations presented in this report.
- Report Preparation: Results of our preliminary geotechnical investigation have been summarized in this report, presenting our findings, conclusions and preliminary geotechnical recommendations for design and construction of the development.

2.0 FINDINGS

2.1 Regional Geologic Setting

The site is located on the gently sloping alluvial plain descending southward from the San Gabriel Mountains. It is located within the Chino Basin in the northern portion of the Peninsular Range geomorphic province of California. Major structural features surrounding the region include the Cucamonga fault and the San Gabriel Mountains to the north, the Chino and Puente Chino Hills faults to the southwest, and the San Jacinto fault to the east. This is an area of large-scale crustal disturbance as the relatively northwestward-moving Peninsular Range Province interacts with the Transverse Range Province (San Gabriel Mountains) to the north. Several active or potentially active faults have been mapped in the region and are believed to accommodate compression associated with this crustal interaction. The site is located approximately 0.9 miles southeast of the Red Hill-Etiwanda Avenue fault, 1.8 miles south of the Cucamonga fault, 4.9 miles southwest of the San Bernardino Valley section of the San Jacinto fault zone, and 8.7 miles southwest of the San Bernardino Mountains section of the San Andreas fault zone.

Regional geologic mapping of the project site and vicinity indicates that near-surface native soils beneath the site consist of young alluvial fan deposits (Morton and Miller, 2006). The surficial geologic units mapped in the vicinity of the project site are shown on Figure 2, Regional Geology Map.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature, previous reports and our subsurface exploration, the site is underlain by alluvial deposits. Based on our field exploration, finer-grained alluvium consisting of silty sand was observed to be approximately 3.5 to 5 feet thick in the eastern portion of the site and approximately 6 to 8 feet thick in the western portion of the site. Field density testing indicated that the finer-grained alluvium had in-situ dry densities ranging from 94 to 102 pcf and in-situ moisture contents ranging from 8 to 14 percent. Beneath the fine-grained alluvium was coarser-grained alluvium consisting of sand with gravel, cobbles, and boulders up to approximately 18 inches in dimension to depths of our deepest test pit (approximately 16 feet bgs). Rock larger than 12 inches in dimension appeared to make up less and 3 percent of the soil encountered in our test pits.

More detailed descriptions of the subsurface soil are presented on the test pit logs from our subsurface exploration (Appendix B).

2.2.1 Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a fill surcharge. Based on our investigation, the finer-grained alluvium encountered is generally considered to be compressible. Partial removal and recompaction of this material under shallow foundations will help reduce the potential for adverse total and differential settlement of the proposed improvements.

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Test results conducted during our investigation and tests previously performed by ASE (2020) generally indicated a negligible to low collapse potential, with one sample exhibiting higher collapse potential.

2.2.2 Expansive Soil

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subjected to large uplifting forces caused by the swelling. Without proper measures taken, heaving and cracking of building foundations and slabs-on-grade could result.

Based on laboratory testing for this site and their granular nature, onsite soils are expected to have very low expansion potential ($EI \leq 20$).

2.2.3 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on the American Concrete Institute (ACI) provisions, adopted by the 2019 California Building Code (CBC, 2019, Chapter 19; and ACI, 2014).

A representative near-surface soil sample collected during our field investigation yielded a soluble sulfate content of less than 0.1 percent. Based on this, onsite soils are expected to have negligible sulfate exposure.

2.2.4 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content, and pH level. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive, while soil having a minimum resistivity of 2,000 to 10,000 is considered moderately corrosive. Soil with a chloride content of 500 ppm or greater is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, a representative soil sample was tested during this investigation and previous studies to determine minimum resistivity, chloride content, and pH of the samples. These test results indicated a pH of 6.4 and 7.5, minimum soil resistivity of 9,275 and 14,880 ohm-cm, and a chloride content of 26 and 308 ppm. Based on the results of testing, the onsite soil is considered moderately corrosive to ferrous metals.

2.3 Groundwater

The site is located within the Upper Santa Ana Valley Groundwater Basin, Chino Subbasin (CDWR, 2006). The Chino Subbasin covers approximately 240 square miles and underlies a portion of the upper Santa Ana Valley in southwestern San Bernardino County and northwestern Riverside County. This subbasin is bounded by the San Gabriel Mountains on the north, the Rialto-Colton fault on the east, the Puente Hills on the south, and the San Jose fault on the northwest. San Antonio and Cucamonga Creeks drain this part of the valley southward to their confluence with the Santa Ana River in the southern part of the subbasin.

Groundwater was not encountered in any of our test pits onsite, which reached a maximum depth of 16 feet bgs, or in the borings previously performed by ASE (2020) extending to a maximum depth of 26 feet bgs. Regional groundwater data indicates that shallow groundwater conditions have not existed historically. Generalized depth to groundwater in 1960 in the area was on the order of 200 to 300 feet bgs (Fife et al., 1976).

According to groundwater information available from California Statewide Groundwater Elevation Monitoring Program (CASGEM, 2020), measurements from a nearby groundwater monitoring well maintained by the Chino Basin Watermaster, located approximately 1.4 miles south of the project site, indicated the shallowest groundwater level during a monitoring period between March 2011 and November 2019 was approximately 575 feet bgs. Groundwater flow is anticipated to be to the south, generally following the surface topography.

Based on the currently proposed development, groundwater is not expected to pose a constraint during and after construction.

2.4 Faulting and Seismicity

2.4.1 Surface Faulting

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (CGS, 2018).

The location of the closest active faults to the site was evaluated using available mapping. The closest active faults to the site the Red Hill-Etiwanda Avenue fault located approximately 0.9 mile away, the Cucamonga fault located approximately 1.8 miles away, the San Bernardino Valley section of the San Jacinto fault zone located approximately 4.9 miles away, and the San Bernardino Mountains section of the San Andreas fault zone located approximately 8.7 miles away from the site.

Considering the location of active and potentially active faults relative to the site, the potential for surface fault rupture at the site is expected to be low and a surface fault rupture hazard evaluation is not mandated for this site.

2.4.2 Seismic Design Parameters

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California. The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the

distance from the seismic source, and the site response characteristics. The site should be expected to experience strong ground shaking after the proposed project is developed resulting from an earthquake occurring along one or more of the major active faults in the region. Accordingly, the project should be designed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

The following parameters should be considered for design under the 2019 CBC:

2019 CBC Seismic Design Parameters

2019 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2019 CBC
Site Latitude and Longitude: 34.1418, -117.5123	
Site Class Definition (1613A.2.2, ASCE 7-16 Ch 20)	D
Mapped Spectral Response Acceleration at 0.2s Period (1613A.2.1), S_s	1.809 g
Mapped Spectral Response Acceleration at 1s Period (1613A.2.1), S_1	0.613 g
Short Period Site Coefficient at 0.2s Period ($T1613A.2.3(1)$), F_a	1.0
Long Period Site Coefficient at 1s Period ($T1613A.2.3(2)$), F_v	1.7*
Adjusted Spectral Response Acceleration at 0.2s Period (1613A.2.3), S_{MS}	1.809 g
Adjusted Spectral Response Acceleration at 1s Period (1613A.2.3), S_{M1}	1.042* g
Design Spectral Response Acceleration at 0.2s Period (1613A.2.4), S_{DS}	1.206 g
Design Spectral Response Acceleration at 1s Period (1613A.2.4), S_{D1}	0.695* g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), PGA	0.741 g
Site Coefficient for Mapped MCE_G PGA (11.8.3.2), F_{PGA}	1.1
Peak Ground Acceleration (1803A.5.12; 11.8.3.2), PGA_M	0.815 g

* Per Table 11.4-2 of Supplement 1 of ASCE 7-16, this value of F_v may only be used to calculate T_s [that note is not included in Table 1613A.2.3(2)]; note that S_{D1} and S_{M1} are functions of F_v . In addition, per Exception 2 of 11.4.8 of ASCE 7-16, special equations for C_s are required. This is in lieu of a site-specific ground motion hazard analysis per ASCE 7-16 Chapter 21.2.

** Site Class D, and all of the resulting parameters in this table, may only be used for structures without seismic isolation or seismic damping systems.

Based on the 2019 CBC Table 1613A.2.3(2) footnote c., F_v should be determined in accordance with Section 11.4.8 of ASCE 7-16, since the mapped spectral response acceleration at 1 second is greater than 0.2g for Site Class D; in accordance with Section 11.4.8 of ASCE 7-16, a site-specific seismic analysis is required. However, the values provided in the table above may be utilized if design is performed in accordance with Exception (2) in Section 11.4.8 of ASCE 7-16, with special requirements for the seismic response coefficient (C_s), and F_v is only used for calculation of T_s . This exception does not apply (and the values in the table above would not be applicable) for proposed structures with a fundamental period of vibration greater than 0.5 s on sites with potentially liquefiable soils; it also does not apply for structures with seismic isolation or seismic damping systems. The project structural engineer should review the seismic parameters. A site-specific seismic ground motion analysis can be performed upon request.

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement, landsliding, and earthquake-induced flooding. The potential for secondary seismic hazards at the site is discussed below.

2.5.1 Liquefaction Potential

Liquefaction is a seismic phenomenon in which loose, saturated, fine-grained granular soils behave similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density, fine, clean sandy soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose and medium dense, near-surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. In general, liquefaction hazards are the most severe in the upper 50 feet bgs.

The State of California has not evaluated the quadrangle that includes the project site for Liquefaction Zones. However, the County of San Bernardino has mapped the project site outside of a zone of liquefaction susceptibility.

Historical high groundwater levels are expected to be on the order of 200 to 300 feet bgs. Considering the lack of shallow groundwater, the potential for liquefaction manifestation and damage at the site is considered low.

2.5.2 Seismically Induced Settlement

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, dry or saturated granular soil. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement.

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed, and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_m) and the geotechnical borings previously performed by ASE (2020). The results of our analyses indicate that the onsite soils are susceptible to low seismic settlement (1.0 inch or less, with differential settlement of 0.5 inch over a horizontal distance of 40 feet based on the MCE), considering the removal of loose soils per our recommendations in Section 3.1. Our seismic settlement analysis is include in Appendix D.

2.5.3 Seismically Induced Landslides

The site is generally level without significant slopes. This site is not considered susceptible to static slope instability or seismically induced landslides.

2.6 Infiltration Testing

Two constant head, open pit percolation tests were conducted within test pits TP-7 and TP-8 in the southeastern portion of the site to estimate the infiltration characteristics of native soils at those locations and depths. Well permeameter tests were conducted at depths of approximately 10 and 9 feet below the existing ground surface, respectively, within granular soils.

Conducting open pit percolation tests at the bottom of larger test pits are useful for field measurements of soil infiltration rates. It should be noted that this is a clean-water, small-scale test, and that correction factors need to be applied. Relatively large test pits were excavated approximately to the depth of the test and a smaller pit for the percolation testing was excavated at the bottom. A layer of gravel was placed in the pit bottom to support temporary perforated standpipe and a float valve within the percolation pit. In addition, gravel was poured around the outside of the standpipe within the test zone to prevent the sides of the pit from caving/collapsing or eroding when water was added. The float valve, lowered into the standpipe, controlled the flow of water into the percolation pit as the water infiltrated into the soil, while maintaining a relatively constant water head within the percolation pit. Incremental infiltration rates were then measured. The test was conducted based on the USBR 7300-89 test method.

The raw infiltration rate at pit location TP-7 was approximately 6 inches per hour and at pit location TP-8 was approximately 2 inches per hour (no factor of safety or correction factors applied) at the depths tested for an average infiltration rate of 4 inches per hour. We believe that the variation in infiltration rates is due to the variation in soil types encountered in each pit. See Section 3.5 for recommendations for infiltration rates.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this investigation, construction of the proposed residential development is feasible from a geotechnical standpoint. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed improvements. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, and potentially compressible soil. Good planning and design of the project can limit the impact of these constraints. Remedial recommendations for these and other geotechnical issues are provided in the following sections.

3.1 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, unless specifically revised or amended below or by future recommendations based on final development plans.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of vegetation, trash and debris, which should be disposed of offsite. Any underground obstructions should be removed, as should large trees and their root systems. Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted.

3.1.2 Overexcavation and Recomaction

To reduce the potential for adverse differential settlement of the proposed improvements, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved.

We recommend that onsite alluvial soils be overexcavated to a depth of 3 feet below the bottom of footings or 5 feet below existing grade whichever is deeper. Overexcavation and recompaction should extend a minimum horizontal distance of 5 feet from perimeter edges of the proposed structures where possible. Local conditions may require that deeper

overexcavation be performed; such areas should be evaluated by Leighton during grading.

Areas outside these overexcavation limits planned for asphalt or concrete pavement, flatwork, and site walls, and areas to receive fill should be overexcavated to a minimum depth of 18 inches below the existing ground surface or below the proposed subgrade, whichever is deeper. In addition, any undocumented artificial fill should be overexcavated.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to well above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

3.1.3 Fill Placement and Compaction

Onsite soil to be used for compacted structural fill should also be free of debris and oversized material (greater than 12 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be reviewed and possibly tested by Leighton.

All fill soil should be placed in thin, loose lifts, moisture conditioned, as necessary, and compacted to a minimum 90 percent relative compaction. Subgrade soils in the upper 6 inches of pavement areas should be compacted to a minimum 95 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.4 Import Fill Soil

Import soil to be placed as fill should be geotechnically accepted by Leighton. Preferably at least 3 working days prior to proposed import to the site, the contractor should provide Leighton pertinent information of the proposed import soil, such as location of the soil, whether stockpiled or native in place, and pertinent geotechnical reports if available. We recommend that a Leighton representative visit the proposed import site to observe the soil conditions and obtain representative soil

samples. Potential issues may include soil that is more expansive than onsite soil, soil that is too wet, soil that is too rocky or too dissimilar to onsite soils, oversize material, organics, debris, etc.

3.1.5 Rippability and Oversized Material

Oversized material (rock or rock fragments greater than 12 inches in dimension) were observed during our investigation, but is expected to make up less than 3 percent of onsite soils. Oversize rock should be placed in deeper fills (deeper than 4 feet below finish grade) or removed from structural fill areas.

3.1.6 Shrinkage and Subsidence

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. This value does not factor in removal of debris or other materials. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as in processing an overexcavation bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site, the measured in-place densities of soils encountered and our experience. We preliminarily estimate the following earth volume changes will occur during grading:

Shrinkage	Approximately 15 percent $\pm$ 4
Subsidence (overexcavation bottom processing)	Approximately 0.2 foot

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

3.2 Shallow Foundation Recommendations

Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a very low expansion potential. Additional testing of the onsite soils should be conducted at the completion of grading to confirm the expansion index of the soil present in the upper portion of pad grade.

Minimum Embedment and Width

Based on our investigation, footings should have a minimum embedment per code requirements, but no less than 18 inches below the lowest adjacent grade, with a minimum width of 15 and 24 inches for square and continuous footings, respectively.

Allowable Bearing

An allowable bearing pressure of 2,000 pounds-per-square-foot (psf) may be used for footings, based on the minimum embedment depth and width above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 4,000 psf. If higher bearing pressures are required, this should be reviewed on a case-by-case basis and may include additional overexcavation and/or soil reinforcement. These allowable bearing pressures are for total dead load and sustained live loads.

Footing reinforcement should be designed by the structural engineer. However, as a minimum, footing reinforcement should consist of one No. 4 rebar at the top and at the bottom of continuous footings and No. 4 rebar spaced at 18 inches on center in each direction for isolated footings. A plasticity index of 10 may be assumed for preliminary design.

Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.35. The passive resistance may be computed using an allowable equivalent fluid pressure of 250 pounds per cubic foot (pcf), assuming there is constant contact between the

footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

Increase in Bearing and Friction – Short Duration Loads

For the case of short term loading (such as those imposed by seismic and wind loading), the allowable bearing pressure and coefficient of friction values may be increased by one-third.

Settlement

The recommended allowable bearing pressure for shallow footings is generally based on a post-construction static settlement of 1 inch. Post-construction static differential settlement is estimated to be on the order of ½ inch over a horizontal distance of 30 feet for shallow footings. Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

3.3 Post-Tensioned Foundations

Post-tensioned foundations, if used, should be designed by a qualified structural engineer in accordance with the 2019 CBC. Based on onsite soils having a very low expansion potential, post tensioned foundations may be designed using the spanability method.

An average allowable bearing pressure of 2,000 pounds per square foot (psf) for dead plus live loads with maximum localized bearing pressure of 2,500 psf for column or wall loads may be used for designing a rigid slab. A subgrade modulus of 150 pounds per cubic inch (pci) may be assumed. The values may be increased by one-third for short-term loading including wind and seismic loads. Regardless of the method used for designing the slabs, the structural engineer should provide the slab with adequate stiffness to minimize potential cracking. The design of post-tensioned slab foundations should follow the procedures described in the latest edition of the Design of Post-Tensioned Slabs-on-Ground by the Post-Tensioning Institute.

The above recommended design criteria may subject to change if the expansion potential of the subgrade soil is found to be different during the construction phase.

With these recommendations we assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage so ponding, which causes significant moisture change below the foundation, does not occur. Our recommendations do not account for excessive irrigation and incorrect landscape design. Sunken planters placed adjacent to the foundation should either be designed to prevent moisture infiltration below the foundation or have efficient drainage system liners. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters. Based on the design parameters we have provided, and our experience with monitoring similar sites on these types of soils, we would expect that with overwatering, up to 1 inch of uplift would occur at the perimeter of the foundation relative to the central portion of the slab.

Future homeowners should be informed and educated regarding the importance of maintaining a constant level of soil moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing expansive soils to become too dry. The soil will undergo shrinkage as it dries up, followed by swelling during the winter, rainy season or when irrigation is resumed, resulting in distress to improvements and structures.

3.4 Slab-on-Grade Recommendations

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for soil with a low expansion potential. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the expansion index of near-surface subgrade soils. In addition, slabs-on-grade should have the following minimum recommended components:

- Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to at least 2 percentage points above optimum moisture content to a minimum depth of 12 inches prior to placing the moisture vapor retarder, steel or concrete.
- Moisture Retarder: A minimum of a 10-mil vapor retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. A Stego 15-mil vapor barrier would provide additional protection.

Since moisture will otherwise be transmitted up from the soil through the concrete, it is important that an intact vapor retarder be installed. We recommend that the vapor retarder meet the requirements of ASTM E 1745 and be installed per ASTM E 1643. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a capillary break (4 inches of 1/2-inch-minimum clean crushed stone) should be placed under the vapor retarder and whether or not a sand blotter layer should be placed over the vapor retarder. Gravel or other protruding objects that could puncture the moisture retarder should be removed from the subgrade prior to placing the vapor retarder, or a stronger vapor retarder intended for the specific conditions present can be used.

- Concrete and Structural Design Thickness: Slabs-on-grade should be designed by the structural engineer, but should be at least 4 inches thick (this is referring to the actual minimum thickness, not the nominal thickness). Reinforcing steel should be designed by the structural engineer, but as a minimum (for conventionally reinforced slabs) should be No. 3 rebar placed at 18 inches on center, each direction, mid-depth in the slab.

Minor cracking of the concrete as it cures, due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, aggregate that is not sufficiently clean, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for shrinkage cracking. The structural engineer should consider these and other pertinent concrete design and construction considerations in slab design and specifications.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide

recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate. The recommendations in this report and our services in general are not intended to address mold prevention, since we, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations are desired, a professional mold prevention consultant should be contacted.

Lateral Resistance of Shallow Foundations

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.30. The passive resistance may be computed using an allowable equivalent fluid pressure of 225 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.5 Retaining Walls

We recommend that retaining walls be backfilled with clean sand and constructed with a backdrain in accordance with the recommendations provided on the attached Retaining Wall Backfill and Subdrain Detail (Figure 3, rear of text). Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Static Equivalent Fluid Weight (pcf)	
Condition	Level Backfill
Active	35 pcf
At-Rest	55 pcf
Passive	350 pcf (allowable) (Maximum of 5,000 psf)

The above values do not contain an appreciable factor of safety unless noted, so the structural engineer should apply the applicable factors of safety and/or load factors during design, as specified by the current California Building Code.

Cantilever walls that are designed to yield at least $0.001H$, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.3 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

For walls with a retained soil height over 6 feet, or where otherwise required by Code or deemed appropriate by the structural engineer, we recommend that an incremental seismic earth pressure load of 32 pcf be applied in addition to the static earth and surcharge load discussed above. Such walls that are designed in the static case assuming the at-rest condition should be checked seismically using the additive seismic load and the active condition. The seismic load should be applied with a standard Equivalent Fluid Pressure distribution (i.e. a triangular pressure distribution, with the final component acting at $1/3 \cdot H$ above the base of the wall, where H is the wall height).

3.6 Pavement Design

Preliminary recommended pavement sections presented below were calculated using the current Caltrans Highway Design Manual, the City of Rancho Cucamonga Standard Plan 100-A, Roadway Sections and an assumed design R-value of 50. Laboratory tests performed by ASE (2020) resulted on an R-Value of 74 for a sample of onsite silty sand soils. Final pavement design should be based on the Traffic Index determined by the project civil engineer and R-value testing provided near the end of grading.

Asphalt Pavement Section Thickness, Type I Subgrade Soil			
Street Designation (Traffic Index)	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base Thickness (inches)	Total Pavement Section Thickness (inches)
Local (5)	3	4	7
Collector (6)	4	4	8

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction or Caltrans Specifications. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled.

Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompact to a minimum of 95 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

If the pavement is to be constructed prior to construction of the structures, we recommend that the full depth of the pavement section be placed in order to support heavy construction traffic.

3.7 Infiltration Recommendations

Infiltration Rate:

Open pit percolation tests at TP-7 and TP-8 yielded small-scale, clean-water infiltration rate averaging 4 inches per hour in the area of the planned infiltration facilities, with a depth of a minimum of 8 feet below existing grade (see Section 2.6). The subsurface soils encountered suggest infiltration will be feasible. We recommend an unfactored (small-scale) infiltration rate of 4 inches per hour be used for preliminary design. Correction factors need to be applied, as discussed below.

We recommend that a correction factor/safety factor be applied to the infiltration rate in conformance with San Bernardino County guidelines, since monitoring of actual facility performance has shown that actual infiltration rates are lower than for small-scale tests. The small-scale infiltration rate should be divided by a

correction factor of at least 2 for buried chambers, and at least 3 for open basins or for conditions where retained water will be exposed to the open atmosphere, but the correction/safety factor may be higher based on project-specific aspects. If open basins are planned, we recommend that a low-flow infiltration trench with a minimum depth of 4 feet be constructed in the bottom of the basin; this low-flow trench should be backfilled with ASTM C33 fine Aggregate (as in clean washed concrete sand) with a special criterion that it contains a maximum fines content (passing the No. 200 sieve) of 2 percent by weight.

The infiltration rates described herein are for a clean, unsilted infiltration surface in native, sandy alluvial soil. These values may be reduced over time as silting of the infiltration facility occurs. Furthermore, if the basin or chamber bottom is allowed to be compacted by heavy equipment, this value is expected to be significantly reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of the soil particles, particle shape, fines content, clay content, and density. Small changes in soil conditions, including density, can cause large differences in observed infiltration rates. Infiltration is not suitable in compacted fill.

It should be noted that during periods of prolonged precipitation, the underlying soils tend to become saturated to greater and greater depths/extents. Therefore, infiltration rates tend to decrease with prolonged rainfall. It is difficult to extrapolate longer-term, full-scale infiltration rates from small-scale tests, and as such, this is a significant source of uncertainty in infiltration rates.

Additional Review and Evaluation:

Infiltration rates are anticipated to vary significantly based on the location and depth. Infiltration concepts should be discussed with Leighton as infiltration plans are being developed. Leighton should review all infiltration plans, including specific locations and depths of proposed facilities. Further testing may be needed based on the design of infiltration facilities, particularly considering their type, depth and location.

General Design Considerations:

The periodic flow of water carrying sediments into the infiltration facility, plus the introduction of wind-blown sediments and sediments from erosion of basin side walls, can eventually cause the bottom of the facility to accumulate a layer of silt, which has the potential of significantly reducing the overall infiltration rate of the facility. Therefore, we recommend that significant amounts of silt/sediment not

be allowed to flow into the facility within stormwater, especially during construction of the project and prior to achieving a mature landscape on site. We recommend that an easily maintained, robust silt/sediment removal system be installed to pretreat storm water before it enters the infiltration facility.

As infiltrating water can seep within the soil strata nearly horizontally for long distances, it is important to consider the impact that infiltration facilities can have on nearby subterranean structures, such as basement walls or open excavations, whether onsite or offsite, and whether existing or planned. Any such nearby features should be identified and evaluated as to whether infiltrating water can impact these. Such features should be brought to Leighton's attention as they are identified.

Infiltration facilities should not be constructed adjacent to or under buildings. Setbacks should be discussed with Leighton during the planning process.

Infiltration facilities should be constructed with spillways or other appropriate means that would cause overflowing to not be a concern to the facility or nearby improvements.

For buried chambers, control/access manhole covers should not contain holes or should be screened to prevent mosquitos from entering the chambers.

Additional Design Considerations (Particularly for Open Basins):

If open basins are planned, additional observation of the soils exposed at the bottom of the basin should be conducted, as these soils are critical to the basin's success. Soils at the bottom of buried chambers are also important, but not as critical to their success, provided the infiltration chamber cuts through sufficiently granular soils.

In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces with prolonged periods of infiltration. As such, water typically infiltrates much faster near the beginning of and/or immediately after storm events than at times well after a storm when the water level in the facility has receded, since the infiltration rate is then slower due to both lower head and longer overall duration of infiltration. In open basins with compacted or silty bottoms, this could be problematic, in that, even if the basin had already infiltrated significant amounts of storm water, the lower several inches or feet of water could remain in the basin for an extended period of time,

creating a prolonged open-water safety concern and potential for mosquitos. In a buried/covered infiltration chamber without direct access to the open atmosphere, these conditions would be of less concern.

Parks or play/recreation areas should not be constructed within basin bottoms or below the spillway level.

For open basins and swales, vegetation within the basin bottoms and sides is expected to help reduce erosion and help maintain infiltration rates.

Estimating infiltration rates, especially based on small-scale testing, is inexact and indefinite, and often involves known and unknown soil complexities, potentially resulting in a condition where actual infiltration rates of the completed facility are significantly less than design rates. In open infiltration basins, this could create nuisance water in the basin. As such, enhancements may be needed after completion of the basin if prolonged or frequent standing water is experienced. A potential basin enhancement, if needed, might be to install additional infiltration trenches or infiltration borings in the basin bottom to capture and infiltrate low flows and to help speed infiltration during/after storms; specific recommendations, such as minimum trench/boring depth, would be developed based on conditions observed.

Construction Considerations:

We recommend that Leighton evaluate the infiltration facility excavations, to confirm that granular, undisturbed alluvium is exposed in the bottoms and sides. Additional excavation or evaluation may be required if silty or clayey soils are exposed.

It is critical to infiltration that the basin or chamber bottom not be allowed to be compacted during construction or maintenance; rubber-tired equipment and vehicles should not be allowed to operate on the bottom. We recommend that at least the bottom 3 feet of the basins or chambers be excavated with an excavator or similar.

If fill material is needed to be placed in the basin, such as due to removal of uncontrolled artificial fill, the fill material should be select and free-draining sand, and should be observed and evaluated by Leighton.

Maintenance Considerations:

The infiltration facilities should be routinely monitored, especially before and during the rainy season, and corrective measures should be implemented as/when needed. Things to check for include proper upkeep, proper infiltration, absence of accumulated silt, and that de-silting filters/features are clean and functioning. Pretreatment desilting features should be cleaned and maintained per manufacturers' recommendations. Even with measures to prevent silt from flowing into the infiltration facility, accumulated silt may need to be removed occasionally as part of maintenance.

3.8 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

Cantilever shoring should be designed based on an active equivalent fluid pressure of 35 pcf. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to $25H$, where H is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA, standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

3.9 Trench Backfill

Utility-type trenches onsite may be backfilled with the onsite material, provided it is free of debris, significant organic material and oversized material. Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent of 30 or greater. Open-graded rock should be avoided, as surrounding soil would tend to migrate into the rock. The sand should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified in-place by mechanical means, or may be jetted in areas where the trench walls and bottom are in sandy soils with a minimum sand equivalent of 15. The native soil backfill should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction. The thickness of layers should be based on the compaction equipment used in accordance with the Standard Specifications for Public Works Construction (Greenbook).

3.10 Surface Drainage

Inadequate control of runoff water and/or poorly controlled irrigation can cause the onsite soils to expand and/or shrink, producing heaving and/or settlement of foundations, flatwork, walls, and other improvements. Maintaining adequate surface drainage, proper disposal of runoff water, and control of irrigation should help reduce the potential for future soil moisture problems.

Positive surface drainage should be designed to be directed away from foundations and toward approved drainage devices, such as gutters, paved drainage swales, or watertight area drains and collector pipes.

Surface drainage should be provided to prevent ponding of water adjacent to the structures. In general, the area around the buildings should slope away from the building. We recommend that unpaved landscaped areas adjacent to the buildings be avoided. Roof runoff should be carried to suitable drainage outlets by watertight drain pipes or over paved areas.

3.11 Sulfate Attack and Corrosion Protection

Based on our test results and our experience in the area, concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil. Type II cement may be used for concrete construction. The concrete

should be designed in accordance with Table 4.3.1 of the American Concrete Institute ACI 318-08 provisions (ACI, 2008).

Based on our laboratory testing, the onsite soil may be considered moderately corrosive to ferrous metals. Use of non-ferrous buried pipe may be prudent, or ferrous pipe can be protected by dielectric tape, polyethylene sleeves and/or other methods, with recommendations from a corrosion engineer. Corrosion information presented in this report should be provided to the underground utility subcontractors for the project. Additional testing and evaluation by a corrosion engineer may be warranted if corrosion protection is considered critical to the project.

Sulfate exposure and corrosivity to ferrous metals of onsite soils should be reviewed and updated when our laboratory testing is complete.

3.12 Additional Geotechnical Services

The preliminary geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our preliminary geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical investigation and analysis may be required based on final improvement plans. Leighton should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and preliminary recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided:

- After completion of site clearing.
- During overexcavation of compressible soil.
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This report was based in part on data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton and Associates, Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of Manning Homes for application to the design of the proposed residential development in accordance with generally accepted geotechnical engineering practices at this time in California.

See the GBA insert on the following page for important information about this geotechnical engineering report.

Important Information about This Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be, and, in general, if you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying it.* A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may

perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old*.

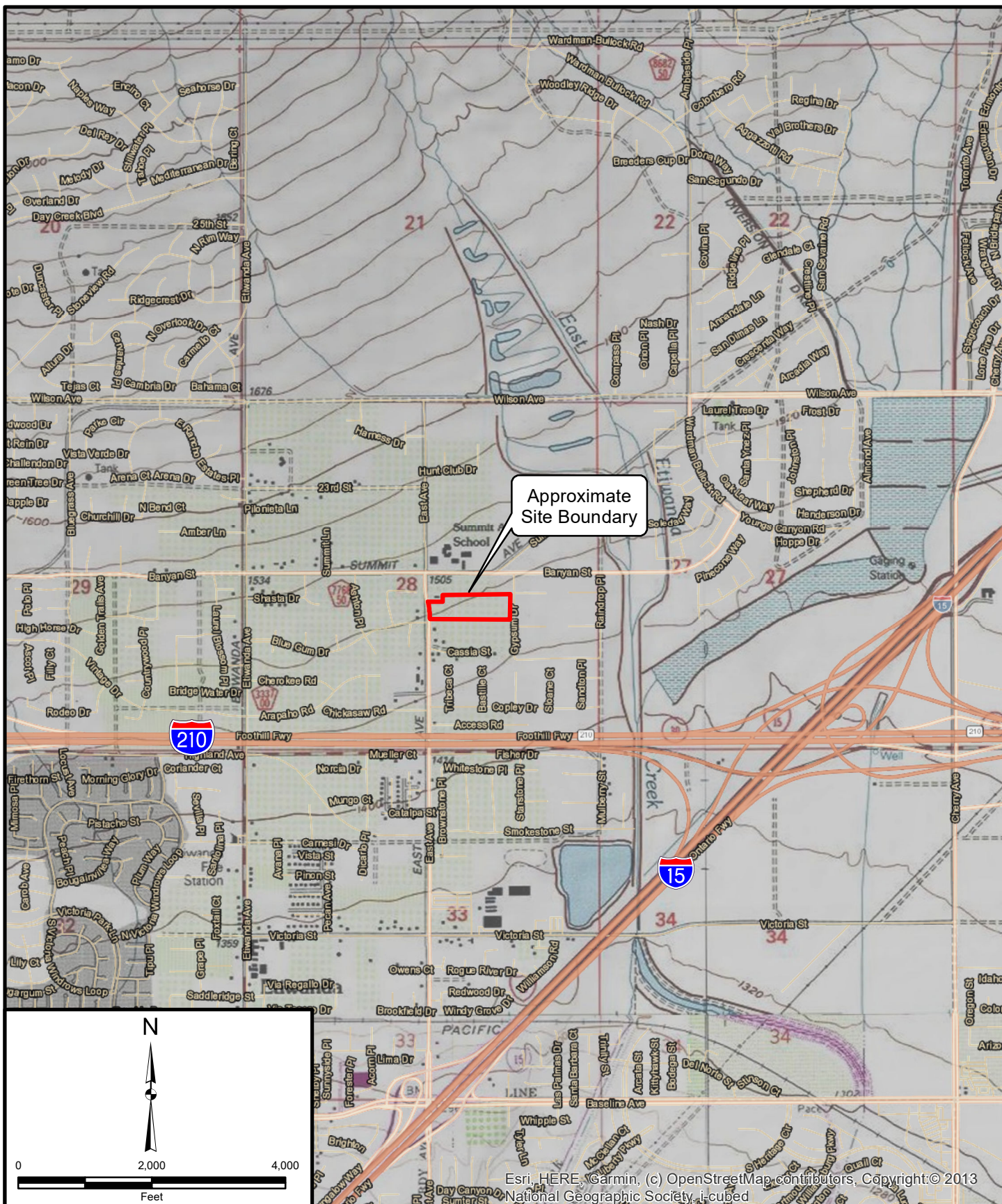
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration*. Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. *Geotechnical engineers are not building-envelope or mold specialists*.



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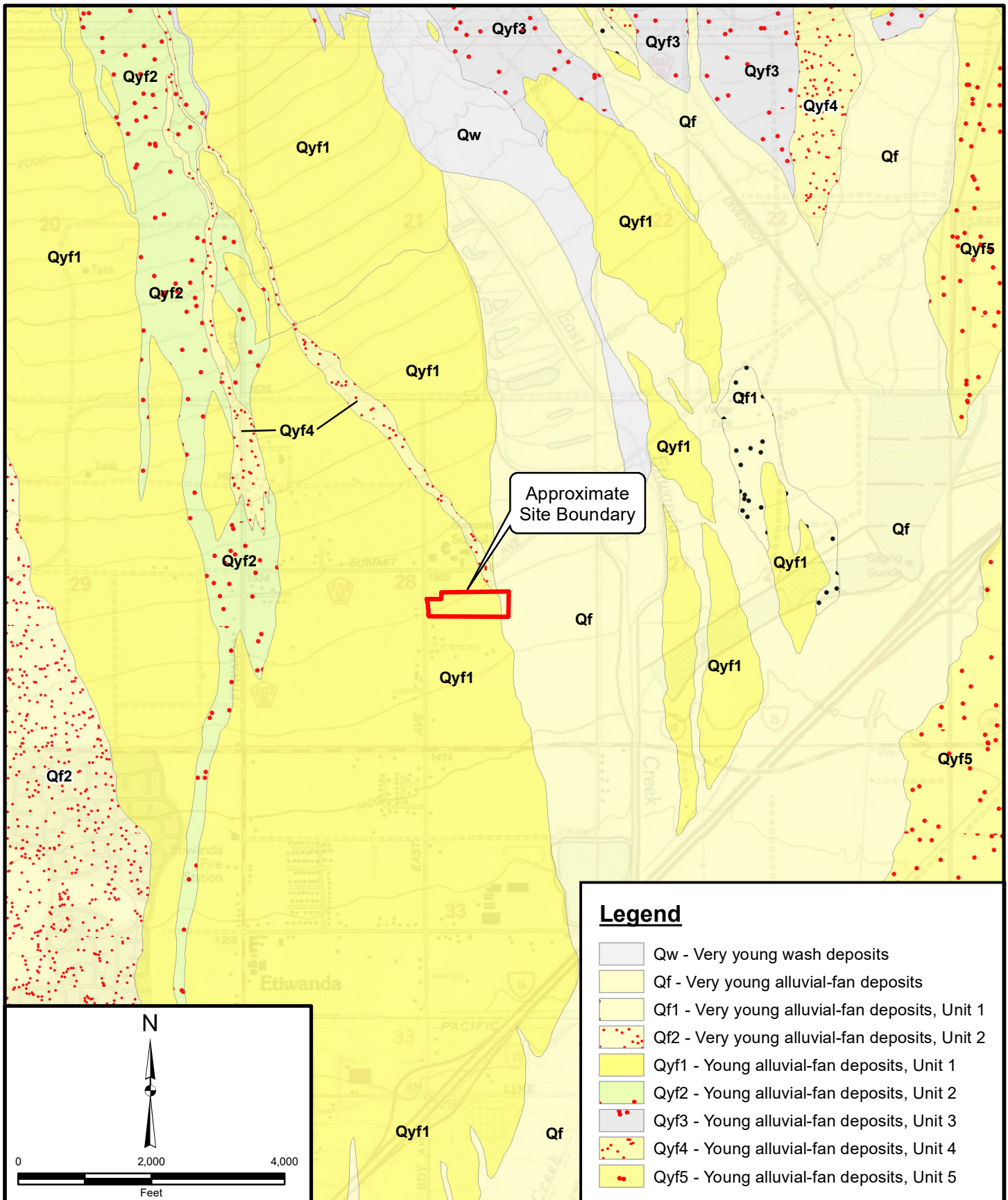


Project: 12696.002	Eng/Geol: JDH/PB
Scale: 1" = 2,000'	Date: June 2020
Base Map: ESRI ArcGIS Online 2020 Thematic Information: Leighton Author: Leighton Geomatics (mmurphy)	

SITE LOCATION MAP
Proposed Residential Development
East Avenue Approximately 300 Feet South of Banyan Street
Tentative Tract No. 20334
City of Rancho Cucamonga, California

Figure 1

Leighton



Project: 12696.002 Eng/Geol: JDH/PB

Scale: 1" = 2,000' Date: June 2020

Reference: USGS, 2006 Geologic map of the San Bernardino and Santa Ana 30'x60 quadrangle, California Version 1.0 Open File Report 2006-1217.
 Author: Leighton Geomatics (mmurphy)

REGIONAL GEOLOGY MAP

Proposed Residential Development
 East Avenue Approximately 300 Feet South of Banyan Street
 Tentative Tract No. 20334
 City of Rancho Cucamonga, California

Figure 2

Leighton

Legend

- TP-11
Gravel @ 5'
T.D.=6'
- Approximate Location of Test Pit
Showing Total Depth???
- Approximate Site Boundary



Project: 12696.002 Eng/Geol: JDH/PB

Scale: 1" = 100' Date: June 2020

Base Map: Sheet 1 of 1, Tentative Tract No. 20334,
Site Plan by Madole & Associates, Inc., Dated January, 2021

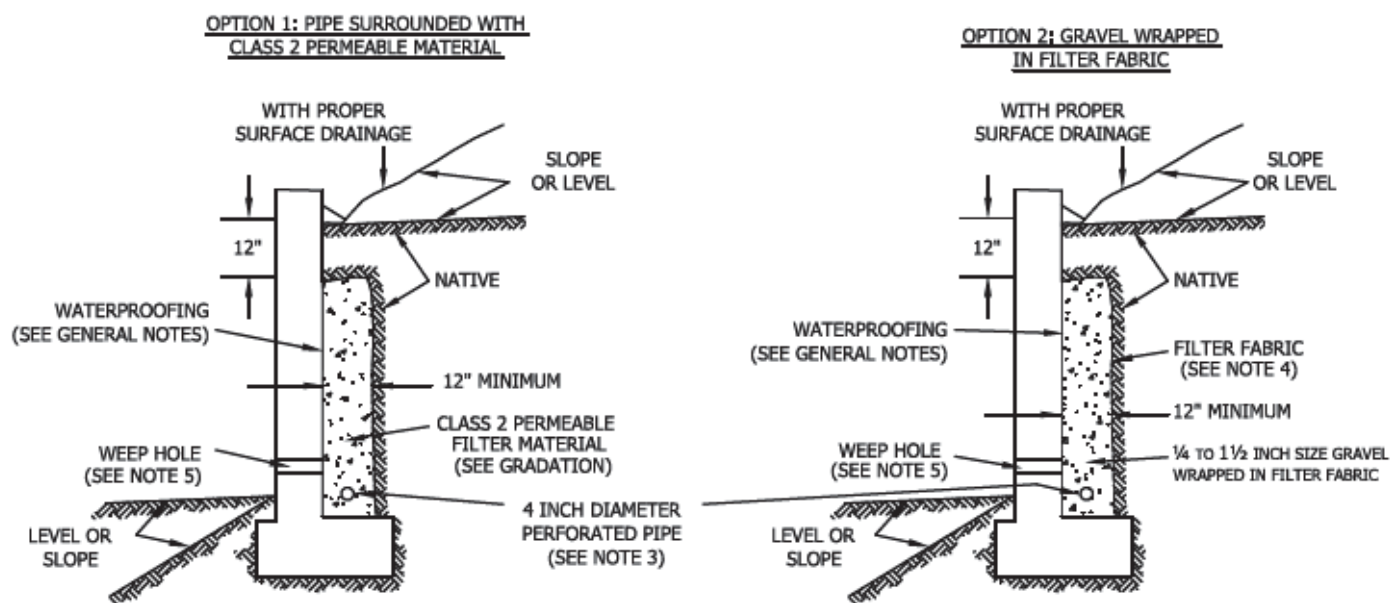
Author: (btran)

TEST PIT LOCATION MAP
Proposed Residential Development
East Avenue Approximately 300 Feet South of Banyan Street
Tentative Tract No. 20334
City of Rancho Cucamonga, California

Figure 3



SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤ 50



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

<u>Sieve Size</u>	<u>Percent Passing</u>
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

- * Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
- * Water proofing of the walls is not under purview of the geotechnical engineer
- * All drains should have a gradient of 1 percent minimum
- * Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
- * Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:

- 1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
- 2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
- 3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
- 4) Filter fabric should be Mirafi 140NC or approved equivalent.
- 5) Weep hole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weep holes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
- 6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
- 7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤ 50



Leighton
Figure 4

APPENDIX A
REFERENCES



Leighton

APPENDIX A

References

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APPENDIX B

GEOTECHNICAL EXPLORATION LOGS BY LEIGHTON

TEST PIT TP-1

Manning Homes

Logged By: ECB

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/18/2020

Elevation: 1484'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Sample Number and Depth	Engineering Properties		
Top	Bottom					Depth	Dry Density (pcf)	Moisture (%)
0.0	8.0	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND: brown, slightly moist, loose 40% (field estimate) fines, 5% (field estimate) gravel 1/4" to 3" in dimension, some 4" cobbles, some rootlets.	Qal	B-1 @ 6'	2'	101.5	8.1
8.0	10.0	SP-SW	Alluvium (Qal): SAND with silt and gravel: dense	Qal		7'	97.1	12.5

Total Depth = 10.0 feet

No groundwater encountered when excavating

Test pit back-filled and tamped with spoils on May 18, 2020



This log is a part of a report by Leighton and should not be used as a stand-alone document.

TEST PIT TP-2

Manning Homes

Logged By: ECB

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/18/2020

Elevation: 1486'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Sample Number and Depth	Engineering Properties		
Top	Bottom					Depth	Dry Density (pcf)	Moisture (%)
0.0	2.0	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND: brown, dry (upper 6") to slightly moist, loose, 30-40% fines (field estimate), 5% gravel (field estimate), some rootlets, burrowing insects.	Qal		2.5'	97.3	9.3
2.0	6.0	SM	Alluvium (Qal): SILTY SAND: brown, moist, loose, <5% gravel and cobbles (field estimate) up to 5 " in dimension, some rootlets	Qal		6'	99.0	12.7

Total Depth = 6.0 feet

No groundwater encountered when excavating

Test pit back-filled and tamped with spoils on May 18, 2020



TEST PIT TP-3

Manning Homes

Logged By: ECB

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/18/2020

Elevation: 1478'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Sample Number and Depth	Engineering Properties		
Top	Bottom					Depth	Dry Density (pcf)	Moisture (%)
0.0	1.5	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND: brown, dry, loose, 5-10% gravel (field estimate) 1/4 to 2" in dimension, some rootlets.	Qal				
1.5	7.5	SM	Alluvium (Qal): SILTY SAND: dark brown, moist, loose to medium dense, 50% fines (field estimate), 5-10% gravel (field estimate) 1/4" to 3" in dimension.	Qal	B-1 @ 5'	5'	94.4	12.6
7.5	8.0	SP-SW	Alluvium (Qal): SAND with silt, gravel and cobbles: dense, cobbles up to 6" in dimension.	Qal				

Total Depth = 8.0 feet

No groundwater encountered when excavating

Test pit back-filled and tamped with spoils on May 18, 2020



This log is a part of a report by Leighton and should not be used as a stand-alone document.

TEST PIT TP-4

Manning Homes

Logged By: ECB

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/18/2020

Elevation: 1472'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Sample Number and Depth	Engineering Properties		
Top	Bottom					Depth	Dry Density (pcf)	Moisture (%)
0.0	1.5	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND: brown, dry, loose, 5% gravel (field estimate) 1/4" to 1/2" in dimension, some rootlets.	Qal				
1.5	4.5	SM	Alluvium (Qal): SILTY SAND: brown, moist, loose to medium dense, some gravel and cobbles up to 5" in dimension.	Qal		4.5'	99	14

Total Depth = 4.5 feet

No groundwater encountered when excavating

Test pit back-filled and tamped with spoils on May 18, 2020



TEST PIT TP-5

Manning Homes

Logged By: ECB

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/18/2020

Elevation: 1476'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Sample Number and Depth	Engineering Properties		
Top	Bottom					Depth	Dry Density (pcf)	Moisture (%)
0.0	1.0	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND: brown, dry, loose	Qal				
1.0	4.0	SM	Alluvium (Qal): SILTY SAND: brown, moist, loose to medium dense, 10% gravel (field estimate) 1/2" to 3" in dimension.	Qal				
4.0	7.0	SP-SW	Alluvium (Qal): SAND with silt and gravel: yellowish brown, moist, dense, 30% subrounded gravel and cobbles (field estimate) up to 9" in dimension.	Qal	B-1 @7'			

Total Depth = 7.0 feet

No groundwater encountered when excavating

Test pit back-filled and tamped with spoils on May 18, 2020



This log is a part of a report by Leighton and should not be used as a stand-alone document.

TEST PIT TP-6

Manning Homes

Logged By: ECB

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/18/2020

Elevation: 1472'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Sample Number and Depth	Engineering Properties		
Top	Bottom					Depth	Dry Density (pcf)	Moisture (%)
0.0	1.0	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND: brown, dry, loose	Qal				
1.0	4.0	SM	Alluvium (Qal): SILTY SAND: brown, moist, loose to medium dense, 5-10% gravel (field estimate) 1/2" to 3" in dimension, rootlets.	Qal				
4.0	5.5	SP-SW	Alluvium (Qal): SAND with silt and gravel: yellowish brown, moist, dense, subrounded gravel, cobbles, and boulders up to 18" in dimension, 8 boulders 12" to 18" in dimension observed in spoils.	Qal				

Total Depth = 5.5 feet

No groundwater encountered when excavating

Test pit back-filled and tamped with spoils on May 18, 2020



This log is a part of a report by Leighton and should not be used as a stand-alone document.

TEST PIT TP-7

Manning Homes

Logged By: ECB

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/18/2020

Elevation: 1464'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Sample Number and Depth	Engineering Properties		
Top	Bottom					Depth	Dry Density (pcf)	Moisture (%)
0.0	1.0	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND: light brown, dry, loose, 5% gravel (field estimate) 1/4" to 1/2" in dimension, rootlets.	Qal				
1.0	3.5	SM	Alluvium (Qal): SILTY SAND: brown, moist, loose to medium dense, 10% gravel (field estimate) 1/2" to 3" in dimension, rootlets.	Qal				
3.5	16.0	SP-SW	Alluvium (Qal): SAND with silt and gravel: yellowish brown, moist, dense, coarse sand, 20% gravel, cobbles, and boulders (field estimate) up to 18" in dimension, 7 boulders 12" to 18" in dimension observed in spoils.	Qal				

Total Depth = 16.0 feet (backhoe arm length limit)

No groundwater encountered when excavating

Test pit bottom use for percolation testing @ 8 feet

Test pit back-filled and tamped with spoils on May 19, 2020



This log is a part of a report by Leighton and should not be used as a stand-alone document.

TEST PIT TP-8

Manning Homes

Logged By: ECB

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/18/2020

Elevation: 1465'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Sample Number and Depth	Engineering Properties		
Top	Bottom					Depth	Dry Density (pcf)	Moisture (%)
0.0	1.0	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND: brown, dry, loose, 5-10% gravel (field estimate) 1/4" to 1/2" in dimension.	Qal				
1.0	4.0	SM	Alluvium (Qal): SILTY SAND: brown, moist, loose to medium dense, 15% gravel and cobbles up to 4" in dimension.	Qal				
4.0	9.3	SP-SW	Alluvium (Qal): SAND with silt and gravel: yellowish brown, moist, dense, 30% subrounded gravel, cobbles, and boulders (field estimate) up to 18" in dimension, 4 subrounded boulders 12" to 18" in dimension observed in spoils.	Qal				

Total Depth = 9.3 feet

No groundwater encountered when excavating

Test pit bottom use for percolation testing @ 8 feet

Test pit back-filled and tamped with spoils on May 19, 2020



This log is a part of a report by Leighton and should not be used as a stand-alone document.

TEST PIT TP-9

Manning Homes

Logged By: JDO

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/19/2020

Elevation: 1487'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Sample Number and Depth	Engineering Properties		
Top	Bottom					Depth	Dry Density (pcf)	Moisture (%)
0.0	7.5	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND with gravel: brown, moist, fine to medium sand, low to medium dense	Qal	Drive1 @5'			
7.5	9.0	SP-SW	Alluvium (Qal): SAND with silt, gravel, and cobbles: dense, many cobbles, few boulders 12"+ in dimension.	Qal	Drive2 @7'			

Total Depth = 9.0 feet

No groundwater encountered when excavating

Test pit back-filled and tamped with spoils on May 19, 2020



This log is a part of a report by Leighton and should not be used as a stand-alone document.

TEST PIT TP-10

Manning Homes

Logged By: JDO

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/19/2020

Elevation: 1480'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Laboratory Test Results			
Top	Bottom				Sample Number	Depth (feet)	Dry Density (pcf)	Moisture (%)
0.0	7.0	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND with gravel: brown, moist, loose to medium dense, fine to medium grained sand.	Qal				
7.0	8.0	SP-SW	Alluvium (Qal): SAND with silt, gravel, and cobbles: dense, 5-10% fines (field estimate).	Qal				

Total Depth = 8.0 feet

No groundwater encountered when excavating

Test pit back-filled and tamped with spoils on May 19, 2020



This log is a part of a report by Leighton and should not be used as a stand-alone document.

TEST PIT TP-11

Manning Homes

Logged By: JDO

Sampled By: JDO

Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12696.002

Date Excavated: 05/19/2020

Elevation: 1475'

This soil description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Laboratory Test Results			
Top	Bottom				Sample Number	Depth (feet)	Dry Density (pcf)	Moisture (%)
0.0	5.0	SM	surface: loose silty sand, dry patchy vegetation Alluvium (Qal): SILTY SAND with gravel: brown, moist, loose to medium dense, fine to medium grained sand.	Qal				
5.0	6.0	SP-SW	Alluvium (Qal): SAND with gravel and cobbles: light brown, moist, dense, fine to coarse grained sand, few boulders 12"+ in dimension.	Qal				

Total Depth = 6.0 feet

No groundwater encountered when excavating

Test pit back-filled and tamped with spoils on May 19, 2020



This log is a part of a report by Leighton and should not be used as a stand-alone document.

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13

16

0.8

98.9

Tu>3h?: yes, OK

99.1

-31 Depth below GS (in.)

-31 Depth below GS (in.)

Calculations

template updated: 8/14/19

Leighton

<u>Initial estimated Depth to Water Surface (in.):</u>	11
<u>Average depth of water in well, "h" (in.):</u>	8
approx. h/r:	0.3
Tu (Fig. 8) (ft):	99.0
Tu>3h?:	yes, OK

Average depth of water in well, "h" (in.): 8

approx. h/r: 0.3

Tu (Fig. 8) (ft): 99.0

Tu>3h?: yes, OK

Water

46.48

100

--	--

Cross-sectional area for vol calcs (in.^2): 678.3

<i>ft</i>	<i>in.</i>	Total (<i>in.</i>)
	19.5 in.	19.5
	23. in.	23
	-13. in.	-13
	F	
	34	

	23. in.	23
--	---------	----

	-13. in.	-13
--	----------	-----

	F
--	---

34

[illegible]

Calculations

[illegible]

APPENDIX C

LABORATORY TEST RESULTS



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Manning Homes Etiwanda Tested By: J. Gonzalez Date: 06/01/20
 Project No.: 12696.002 Input By: A. Santos Date: 06/05/20
 Boring No.: TP3 Depth (ft.): 5.0
 Sample No.: B1
 Soil Identification: Olive brown silty sand with gravel (SM)g

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

Preparation Method:	☒	Moist	Scalp Fraction (%)	Rammer Weight (lb.) =	10.0
		Dry	#3/4	Height of Drop (in.) =	18.0
Compaction Method:	☒	Mechanical Ram	#3/8		
		Manual Ram	#4	Mold Volume (ft ³)	0.03330

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3968	4049	3963			
Weight of Mold (g)	1868	1868	1868			
Net Weight of Soil (g)	2100	2181	2095			
Wet Weight of Soil + Cont. (g)	288.1	249.8	357.0			
Dry Weight of Soil + Cont. (g)	272.0	231.7	323.1			
Weight of Container (g)	39.8	40.2	38.8			
Moisture Content (%)	6.93	9.45	11.92			
Wet Density (pcf)	139.0	144.4	138.7			
Dry Density (pcf)	130.0	131.9	123.9			

Maximum Dry Density (pcf) **132.5**

Optimum Moisture Content (%) **8.7**

Corrected Dry Density (pcf) **137.1**

Corrected Moisture Content (%) **7.5**

☒ **Procedure A**
 Soil Passing No. 4 (4.75 mm) Sieve
 Mold : 4 in. (101.6 mm) diameter
 Layers : 5 (Five)
 Blows per layer : 25 (twenty-five)
 May be used if + #4 is 20% or less

☐ **Procedure B**
 Soil Passing 3/8 in. (9.5 mm) Sieve
 Mold : 4 in. (101.6 mm) diameter
 Layers : 5 (Five)
 Blows per layer : 25 (twenty-five)
 Use if + #4 is >20% and + 3/8 in. is 20% or less

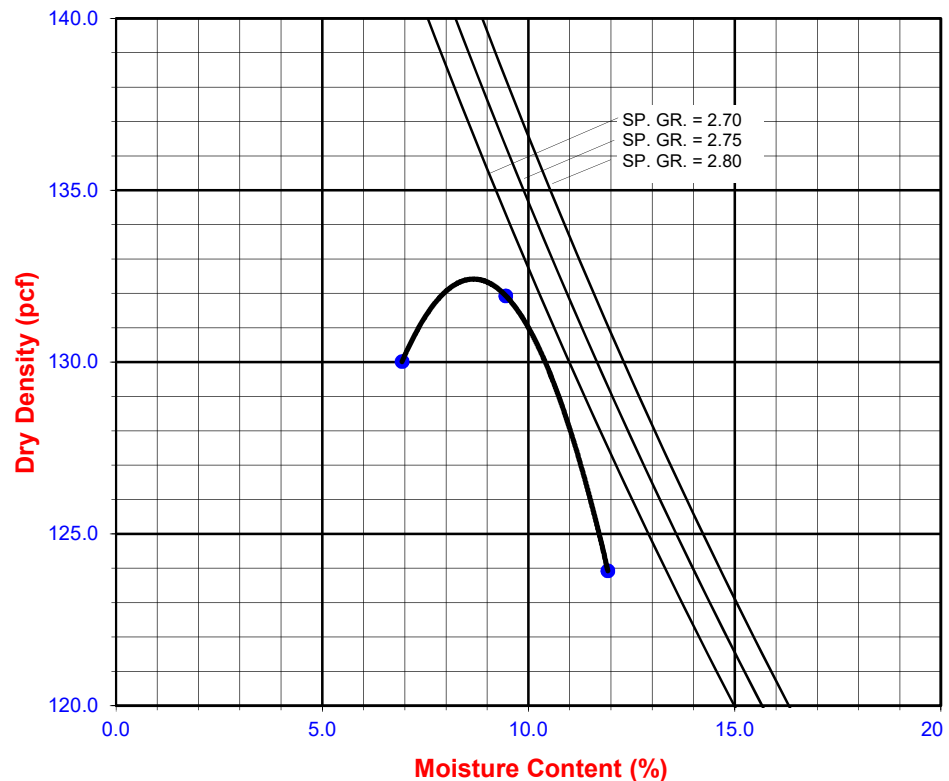
☐ **Procedure C**
 Soil Passing 3/4 in. (19.0 mm) Sieve
 Mold : 6 in. (152.4 mm) diameter
 Layers : 5 (Five)
 Blows per layer : 56 (fifty-six)
 Use if + 3/8 in. is >20% and + 3/4 in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI





EXPANSION INDEX of SOILS

ASTM D 4829

Project Name: Manning Homes Etiwanda Tested By: S. Felter Date: 06/02/20
Project No.: 12696.002 Checked By: A. Santos Date: 06/05/20
Boring No.: TP3 Depth (ft.): 5.0
Sample No.: B1
Soil Identification: Olive brown silty sand with gravel (SM)/g

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0015
Wt. Comp. Soil + Mold (g)	582.30	443.70
Wt. of Mold (g)	163.40	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (g)	842.70	607.10
Dry Wt. of Soil + Cont. (g)	780.30	551.27
Wt. of Container (g)	0.00	163.40
Moisture Content (%)	8.00	14.39
Wet Density (pcf)	126.4	133.6
Dry Density (pcf)	117.0	116.8
Void Ratio	0.441	0.443
Total Porosity	0.306	0.307
Pore Volume (cc)	63.3	63.6
Degree of Saturation (%) [S _{meas}]	49.0	87.7

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
06/02/20	10:53	1.0	0	0.6035
06/02/20	11:03	1.0	10	0.6035
Add Distilled Water to the Specimen				
06/02/20	11:10	1.0	7	0.6050
06/03/20	6:20	1.0	1157	0.6050
06/03/20	8:00	1.0	1257	0.6050

Expansion Index (EI _{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	1
---	---

Boring No.	TP3							
Sample No.	B1							
Depth (ft.)	5.0							
Sample Type	Bulk							
Soil Identification	Olive Brown silty sand with gravel (SM)/g							

No Moisture Correction; ASTM D 1140 modified to include splitting the sample on the #4 sieve

Total Sample Dry Weight Determination

Dry Weight of Soil + Container (g)	4066.40							
Weight of Container (g)	234.70							
Dry Weight of Soil (g)	3831.70							

Sample Dry Weight Determination, Retained on Sieve #4

Dry Weight of Sample + Cont. (g)	839.65							
Weight of Container (g)	234.70							
Weight of Dry Sample (g)	604.95							

Sample Dry Weight Determination, Passing Sieve #4

Dry Weight of Sample + Cont. (g)	610.60							
Weight of Container (g)	108.30							
Weight of Dry Sample (g)	502.30							

After Wash

Method (A or B)	A							
Dry Weight of Sample + Cont. (g)	494.69							
Weight of Container (g)	108.30							
Weight of Dry Sample (g)	386.39							

% Passing No. 4 Sieve	84.2							
% Retained No. 4 Sieve	15.8							
% Passing No. 200 Sieve	19.4							

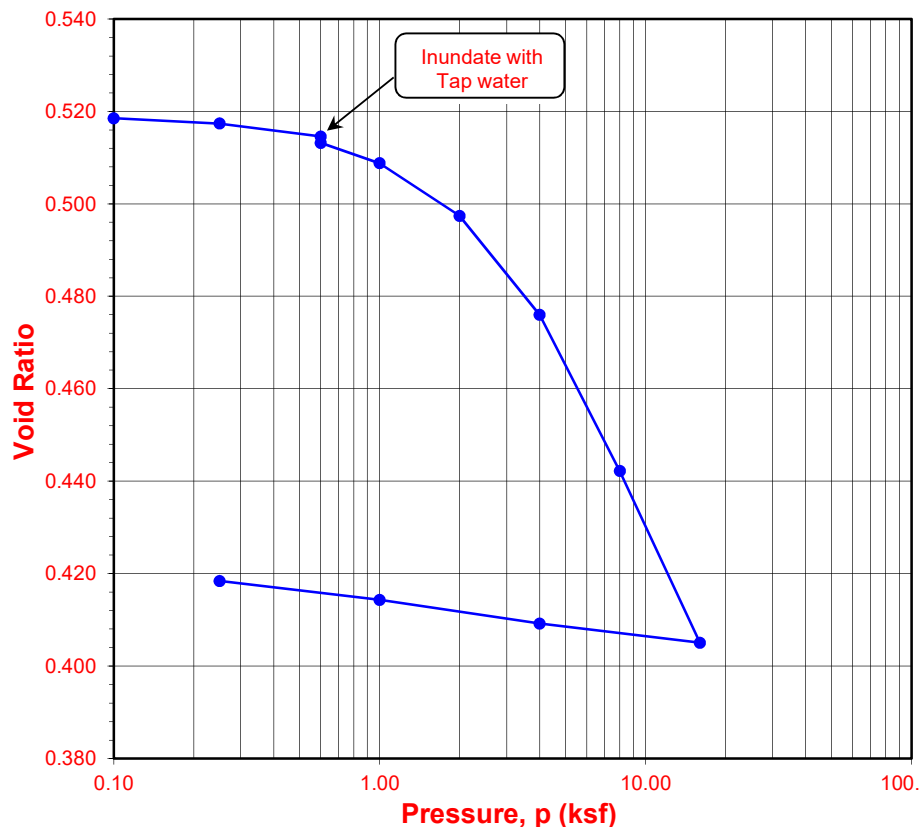
	PERCENT PASSING No. 200 SIEVE ASTM D 1140			Project Name: <u>Manning Homes Etiwanda</u>				
				Project No.: <u>12696.002</u>				
				Client Name: _____				
				Tested By: <u>A. Santos</u>		Date: <u>06/02/20</u>		



ASTM D 2435

Tested By: G. Bathala Date: 05/28/20
 Checked By: A. Santos Date: 06/05/20
 Depth (ft.): 5.0
 Sample Type: Ring

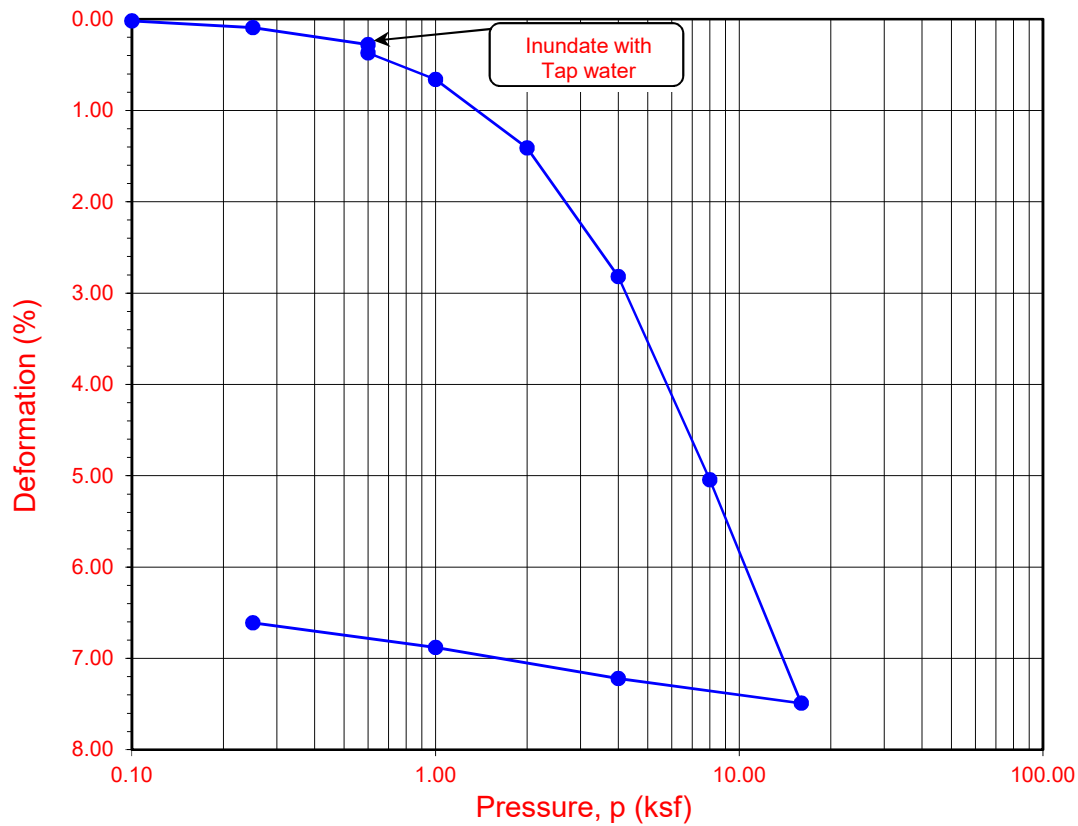
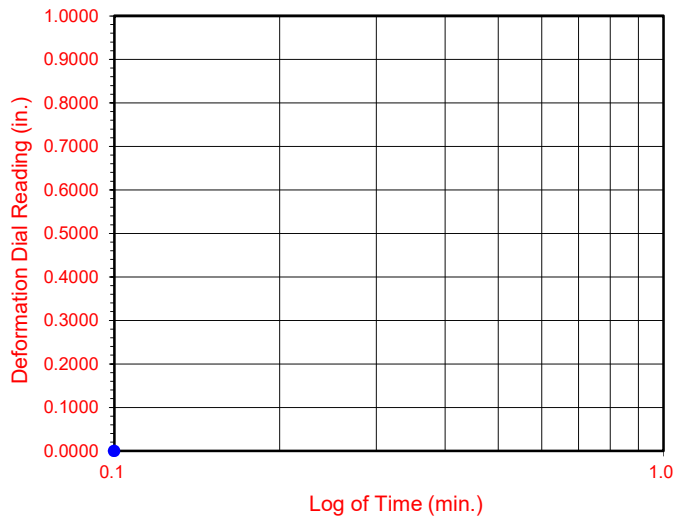
Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	190.87
Weight of Ring (g)	46.52
Height after consol. (in.)	0.9339
Before Test	
Wt.Wet Sample+Cont. (g)	252.33
Wt.of Dry Sample+Cont. (g)	237.57
Weight of Container (g)	56.92
Initial Moisture Content (%)	8.2
Initial Dry Density (pcf)	111.0
Initial Saturation (%)	43
Initial Vertical Reading (in.)	0.3099
After Test	
Wt.of Wet Sample+Cont. (g)	259.86
Wt. of Dry Sample+Cont. (g)	240.85
Weight of Container (g)	63.21
Final Moisture Content (%)	14.50
Final Dry Density (pcf)	116.8
Final Saturation (%)	88
Final Vertical Reading (in.)	0.2405
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43



Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3097	0.9998	0.00	0.02	0.519	0.02
0.25	0.3087	0.9988	0.03	0.12	0.517	0.09
0.60	0.3064	0.9965	0.07	0.35	0.515	0.28
0.60	0.3055	0.9956	0.07	0.44	0.513	0.37
1.00	0.3022	0.9923	0.11	0.77	0.509	0.66
2.00	0.2938	0.9839	0.20	1.61	0.497	1.41
4.00	0.2784	0.9685	0.33	3.15	0.476	2.82
8.00	0.2547	0.9448	0.48	5.53	0.442	5.05
16.00	0.2283	0.9184	0.67	8.16	0.405	7.49
4.00	0.2327	0.9228	0.50	7.72	0.409	7.22
1.00	0.2372	0.9273	0.39	7.27	0.414	6.88
0.25	0.2405	0.9306	0.33	6.94	0.418	6.61

[illegible]

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
TP9	D1	5.0	8.2	14.5	111.0	116.8	0.519	0.418	43	88

Soil Identification: Dark yellowish brown silty, clayey sand (SC-SM)



ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS ASTM D 2435

Project No.: 12696.002

Manning Homes Etiwanda

06-20



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS ASTM D 4546

Project Name: Manning Homes Etiwanda
 Project No.: 12696.002
 Boring No.: TP9
 Sample No.: D2
 Sample Description: Dark yellowish brown silty, clayey sand (SC-SM)

Tested By: G. Bathala Date: 06/02/20
 Checked By: A. Santos Date: 06/05/20
 Sample Type: Ring
 Depth (ft.): 7.0

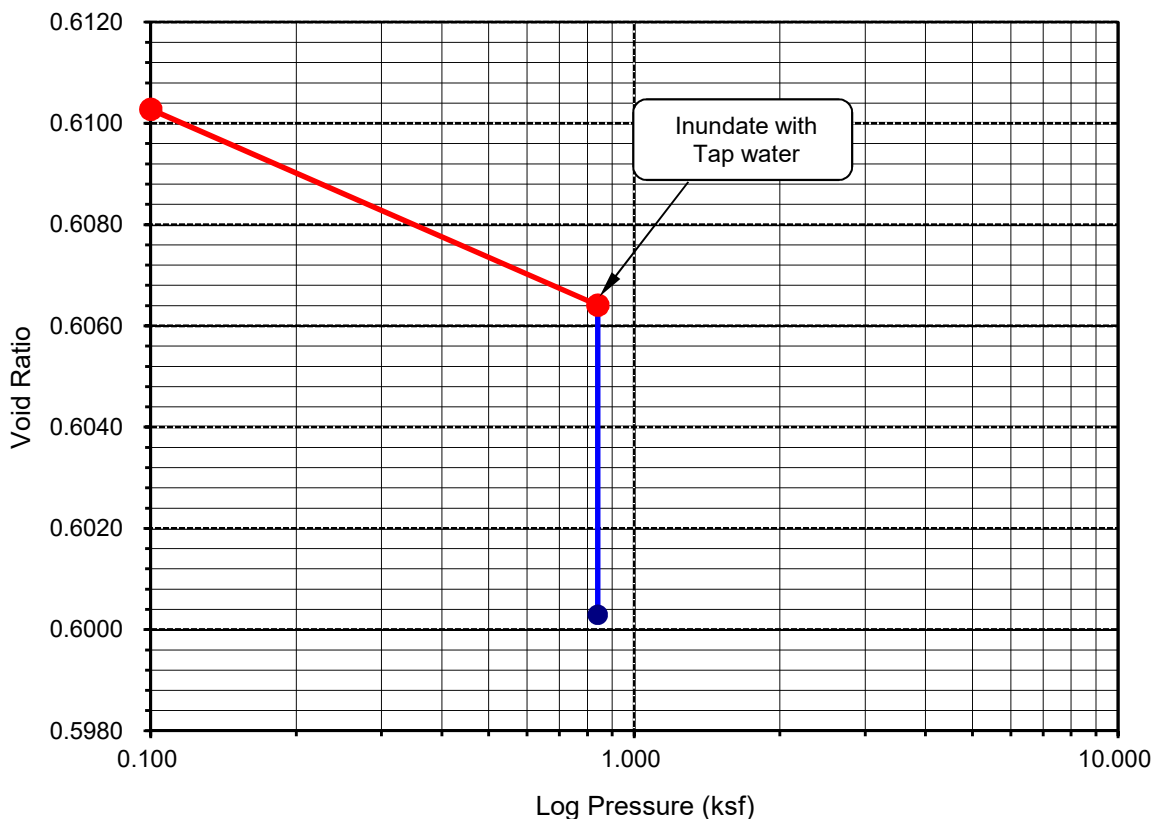
Initial Dry Density (pcf):	104.7
Initial Moisture (%):	9.84
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2563
Diameter(in):	2.415

Final Dry Density (pcf):	105.3
Final Moisture (%) :	17.1
Initial Void Ratio:	0.6104
Specific Gravity(assumed):	2.70
Initial Saturation (%)	43.5

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2562	0.9999	0.00	-0.01	0.6103	-0.01
0.840	0.2520	0.9957	0.18	-0.43	0.6064	-0.25
H2O	0.2482	0.9919	0.18	-0.81	0.6003	-0.63

Percent Swell (+) / Settlement (-) After Inundation = -0.38

Void Ratio - Log Pressure Curve





TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Manning Homes Etiwanda Tested By : A. Santos Date: 06/01/20
Project No. : 12696.002 Checked By: G. Bathala Date: 06/05/20

Boring No.	TP3			
Sample No.	B1			
Sample Depth (ft)	5.0			
Soil Identification:	Olive brown (SM)g			
Wet Weight of Soil + Container (g)	106.90			
Dry Weight of Soil + Container (g)	104.17			
Weight of Container (g)	57.00			
Moisture Content (%)	5.79			
Weight of Soaked Soil (g)	100.10			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	306			
Crucible No.	21			
Furnace Temperature (°C)	860			
Time In / Time Out	9:10/9:55			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	22.1647			
Wt. of Crucible (g)	22.1620			
Wt. of Residue (g) (A)	0.0027			
PPM of Sulfate (A) x 41150	111.11			
PPM of Sulfate, Dry Weight Basis	118			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO ₃ Soln. Used in Titration (C)	3.1			
PPM of Chloride (C -0.2) * 100 * 30 / B	290			
PPM of Chloride, Dry Wt. Basis	308			

pH TEST, DOT California Test 643

pH Value	6.43			
Temperature °C	20.9			



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Manning Homes Etiwanda
 Project No. : 12696.002
 Boring No.: TP3
 Sample No. : B1

Tested By : Y. Nguyen Date: 06/04/20
 Checked By: G. Bathala Date: 06/05/20
 Depth (ft.) : 5.0

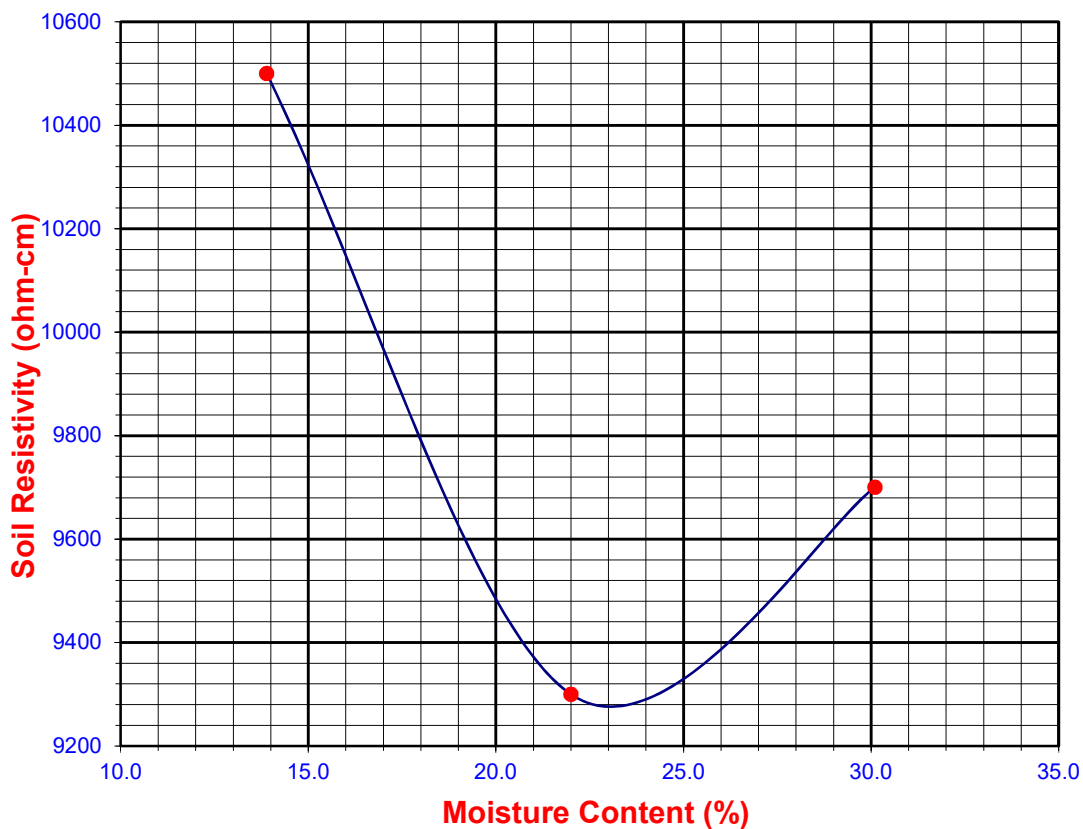
Soil Identification:* Olive brown (SM)g

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	13.89	10500	10500
2	20	22.00	9300	9300
3	30	30.11	9700	9700
4				
5				

Moisture Content (%) (Mci)	5.79
Wet Wt. of Soil + Cont. (g)	106.90
Dry Wt. of Soil + Cont. (g)	104.17
Wt. of Container (g)	57.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.50
Box Constant	1.000
$MC = (((1 + Mci / 100) \times (Wa / Wt + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
9275	23.1	118	308	6.43	20.9



APPENDIX D

SUMMARY OF SECONDARY SEISMIC HAZARD ANALYSIS



Manning Etiwanda

Latitude, Longitude: 34.141775, -117.512304



Date	6/8/2020, 1:54:25 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.809	MCE_R ground motion. (for 0.2 second period)
S_1	0.613	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.809	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.206	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.741	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.815	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
S_{sRT}	2.02	Probabilistic risk-targeted ground motion. (0.2 second)
S_{sUH}	2.192	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
S_{sD}	1.809	Factored deterministic acceleration value. (0.2 second)
S_{1RT}	0.78	Probabilistic risk-targeted ground motion. (1.0 second)
S_{1UH}	0.868	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S_{1D}	0.613	Factored deterministic acceleration value. (1.0 second)
PGA_d	0.741	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.922	Mapped value of the risk coefficient at short periods
C_{R1}	0.898	Mapped value of the risk coefficient at a period of 1 s

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Unified Hazard Tool



Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (v4.2.0)

Latitude

Decimal degrees

34.1418

Longitude

Decimal degrees, negative values for western longitudes

-117.5123

Site Class

259 m/s (Site class D)

Spectral Period

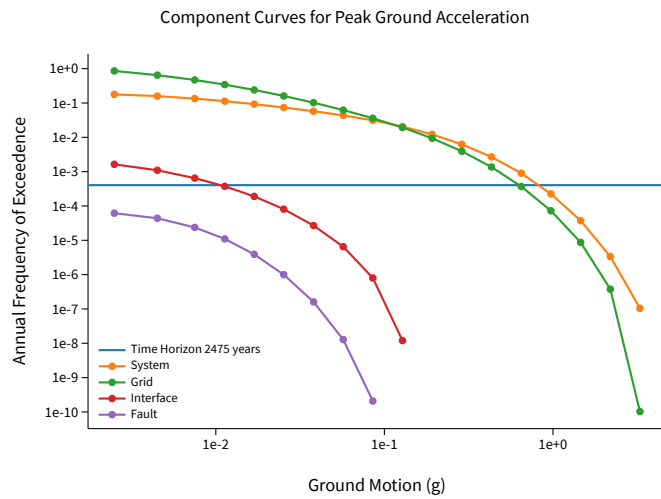
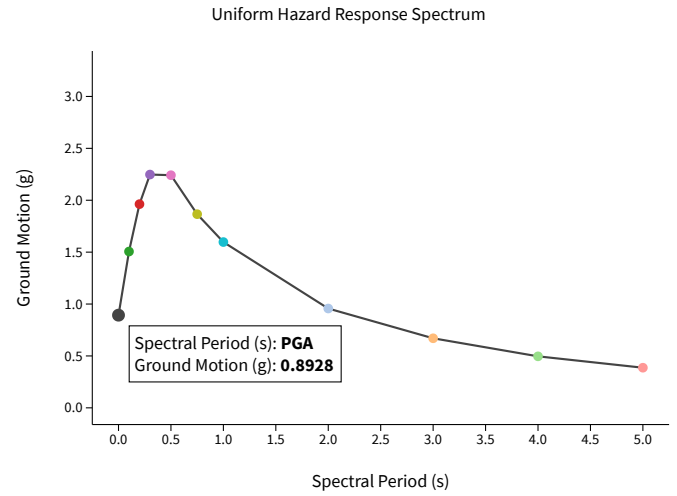
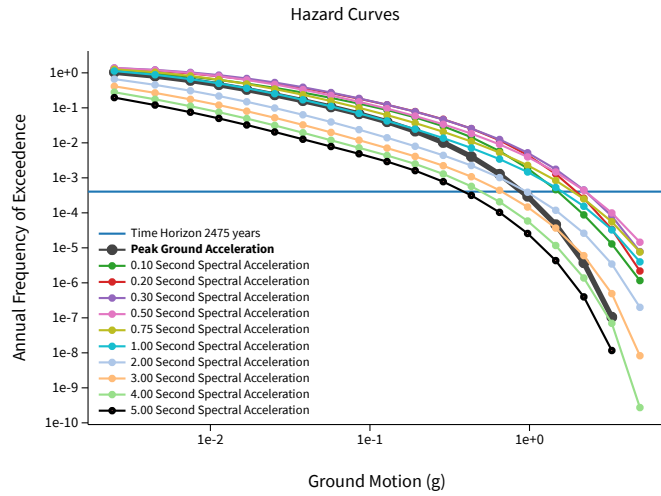
Peak Ground Acceleration

Time Horizon

Return period in years

2475

^ Hazard Curve

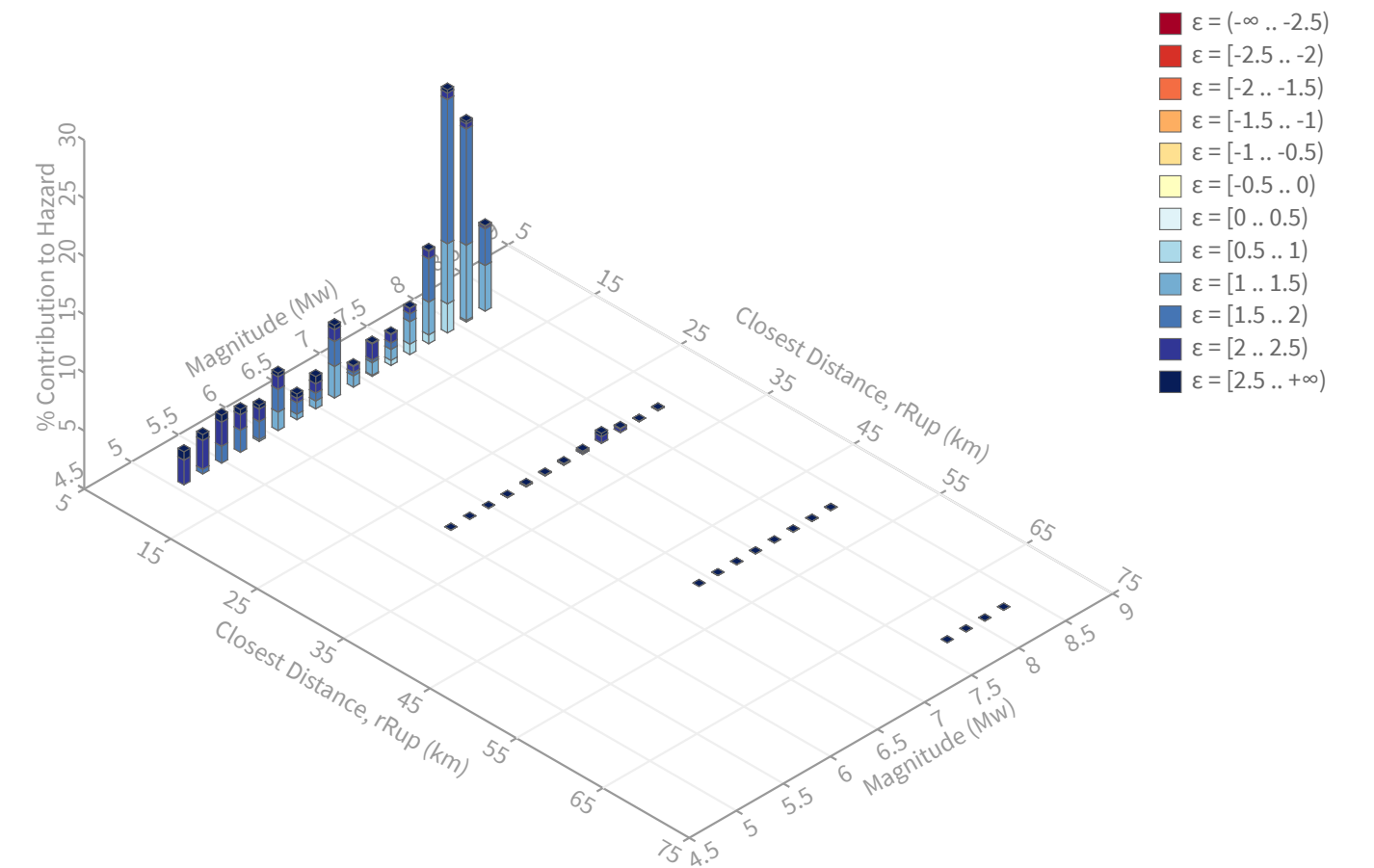


[View Raw Data](#)

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr⁻¹
PGA ground motion: 0.89279393 g

Recovered targets

Return period: 3189.3567 yrs
Exceedance rate: 0.00031354286 yr⁻¹

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.02 %

Mean (over all sources)

m: 7.22
r: 10.41 km
ε₀: 1.71 σ

Mode (largest m-r bin)

m: 7.9
r: 10.91 km
ε₀: 1.56 σ
Contribution: 20.88 %

Mode (largest m-r-ε₀ bin)

m: 7.91
r: 13.68 km
ε₀: 1.75 σ
Contribution: 12.4 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

- ε0: [-∞ .. -2.5)
- ε1: [-2.5 .. -2.0)
- ε2: [-2.0 .. -1.5)
- ε3: [-1.5 .. -1.0)
- ε4: [-1.0 .. -0.5)
- ε5: [-0.5 .. 0.0)
- ε6: [0.0 .. 0.5)
- ε7: [0.5 .. 1.0)
- ε8: [1.0 .. 1.5)
- ε9: [1.5 .. 2.0)
- ε10: [2.0 .. 2.5)
- ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set	Source	Type	r	m	ϵ_0	lon	lat	az	%
UC33brAvg_FM31		System							37.90
	San Andreas (San Bernardino N) [2]		14.90	7.80	1.86	117.412°W	34.247°N	38.00	12.05
	San Jacinto (San Bernardino) [0]		11.34	8.07	1.56	117.437°W	34.222°N	37.84	8.48
	Cucamonga [0]		4.36	7.55	1.16	117.510°W	34.179°N	2.73	7.66
	San Jacinto (Lytle Creek connector) [1]		7.91	8.03	1.36	117.445°W	34.184°N	52.66	2.90
	Fontana (Seismicity) [0]		6.70	6.61	1.52	117.463°W	34.100°N	135.19	2.55
UC33brAvg_FM32		System							37.24
	San Andreas (San Bernardino N) [2]		14.90	7.81	1.86	117.412°W	34.247°N	38.00	12.24
	San Jacinto (San Bernardino) [0]		11.34	8.06	1.56	117.437°W	34.222°N	37.84	8.33
	Cucamonga [0]		4.36	7.58	1.15	117.510°W	34.179°N	2.73	7.62
	San Jacinto (Lytle Creek connector) [1]		7.91	8.02	1.36	117.445°W	34.184°N	52.66	2.82
	Fontana (Seismicity) [0]		6.70	6.61	1.52	117.463°W	34.100°N	135.19	2.09
UC33brAvg_FM31 (opt)		Grid							12.44
	PointSourceFinite: -117.512, 34.182		6.85	5.62	1.84	117.512°W	34.182°N	0.00	2.42
	PointSourceFinite: -117.512, 34.182		6.85	5.62	1.84	117.512°W	34.182°N	0.00	2.42
	PointSourceFinite: -117.512, 34.200		8.14	5.66	2.03	117.512°W	34.200°N	0.00	2.07
	PointSourceFinite: -117.512, 34.200		8.14	5.66	2.03	117.512°W	34.200°N	0.00	2.07
UC33brAvg_FM32 (opt)		Grid							12.42
	PointSourceFinite: -117.512, 34.182		6.85	5.62	1.84	117.512°W	34.182°N	0.00	2.42
	PointSourceFinite: -117.512, 34.182		6.85	5.62	1.84	117.512°W	34.182°N	0.00	2.42
	PointSourceFinite: -117.512, 34.200		8.14	5.66	2.03	117.512°W	34.200°N	0.00	2.07
	PointSourceFinite: -117.512, 34.200		8.14	5.66	2.03	117.512°W	34.200°N	0.00	2.07

Liquefaction Susceptibility Analysis: SPT Method

Based on Youd and Idriss (2001), Martin and Lew (1999).

Project: Manning Home Etiwanda
Project No.: 12696.002

Leighton

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Ground Surface Elev (ft)
B-2	100	100	0	-100
B-3	100	100	0	-100
B-4	100	100	0	-100
B-5	100	100	0	-100
				0
				0
				0
				0

General Parameters:	
$a_{max} = 0.89g$	MCE
$M_w = 7.9$	
MSF eq: 1	(Idriss, 2001)
MSF = 0.88	
Hammer Efficiency = 83	%
$C_E = 1.38$	
$C_B = 1$	
$C_{S(SPT)} = 1.2$	
$C_{S(ring)} = 1$	
Rod Stickup (feet) = 3	
Ring sample correction = 0.65	

Summary of Liquefaction Susceptibility Analysis: SPT Method

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: Manning Home Etiwanda

Project No.: 12696

Leighton

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thickness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	C_s	N_m (corrected for C_s and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	$CRR_{7.5}$	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	Liquefaction Factor of Safety	$(N_1)_{60CS}$ (for Settlement) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87)	Sat Sand Strain (%) (Tok/ Seed 87)	Seismic Sett. of Layer (in.)	Cummulative Seismic Settlement (in.)
B-2	0 to 4	2	4		20	116	7	2	1	4.6	232	8.0	12.3	0.134	232	0.58	0.66	NonLiq	12.3	1.36		0.65	1.2
B-2	4 to 8	6	4		20	119	39	2	1	25.4	702	44.7	51.9	>Range	702	0.57	0.65	NonLiq	51.9	0.03		0.02	0.5
B-2	8 to 13	10	5		20	125	24	2	1	15.6	1190	24.3	29.8	0.457	1190	0.57	0.65	NonLiq	29.8	0.73		0.39	0.5
B-2	13 to 18	15	5		20	132	100	2	1	65.0	1833	81.6	91.7	>Range	1832.5	0.56	0.64	NonLiq	91.7	0.03		0.02	0.1
B-2	18 to 23	20	5		20	113	100	2	1	65.0	2445	78.9	88.8	>Range	2445	0.55	0.63	NonLiq	88.8	0.04		0.03	0.1
B-2	23 to 27	25	5		4	123	100	2	1	65.0	3035	70.9	70.9	>Range	3035	0.54	0.62	NonLiq	70.9	0.09		0.05	0.0
B-3	0 to 6	2	6		20	117	7	2	1	4.6	234	8.0	12.3	0.134	234	0.58	0.66	NonLiq	12.3	1.37		0.99	1.1
B-3	6 to 13	10	7		20	126	48	2	1	31.2	1206	48.3	55.7	>Range	1206	0.57	0.65	NonLiq	55.7	0.08		0.06	0.1
B-3	13 to 18	15	5		20	132	45	2	1	29.3	1851	36.5	43.0	>Range	1851	0.56	0.64	NonLiq	43.0	0.07		0.04	0.1
B-3	18 to 22	20	5		20	122	100	2	1	65.0	2486	78.3	88.1	>Range	2486	0.55	0.63	NonLiq	88.1	0.04		0.02	0.0
B-4	0 to 5	3	5		20	119	7	2	1	4.6	357	8.0	12.3	0.134	357	0.57	0.66	NonLiq	12.3	2.08		1.25	1.6
B-4	5 to 9	7	4		20	112	14	2	1	9.1	819	16.1	21.0	0.228	819	0.57	0.65	NonLiq	21.0	0.61		0.26	0.3
B-4	9 to 12	10	3		20	123	51	2	1	33.2	1172	52.0	59.8	>Range	1171.5	0.57	0.65	NonLiq	59.8	0.07		0.03	0.1
B-4	12 to 15	13	4		20	125	100	2	1	65.0	1544	88.9	99.6	>Range	1543.5	0.56	0.64	NonLiq	99.6	0.06		0.03	0.0
B-5	0 to 5	3	5		20	123	9	2	1	5.9	369	10.3	14.8	0.158	369	0.57	0.66	NonLiq	14.8	1.93		1.16	2.0
B-5	5 to 9	7	4		20	117	14	2	1	9.1	849	15.8	20.7	0.224	849	0.57	0.65	NonLiq	20.7	0.65		0.27	0.8
B-5	9 to 13	10	4		20	102	20	2	1	13.0	1178	20.4	25.6	0.304	1177.5	0.57	0.65	NonLiq	25.6	0.85		0.41	0.5
B-5	13 to 17	15	5		4	124	40	2	1	26.0	1743	33.5	33.5	>Range	1742.5	0.56	0.64	NonLiq	33.5	0.25		0.13	0.1

Summary of Liquefaction Susceptibility Analysis: SPT Method

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

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Boring No.	Approx. Layer	SPT	Approx	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont	γ_t	N_m or B	Sampler Type (enter 2 if mod CA Ring)	C_s	N_m	Exist	$(N_1)_{60}$	$(N_1)_{60CS}$	$CRR_{7.5}$	Design			Liquefaction Factor of Safety	$(N_1)_{60CS}$	Dry Sand	Sat Sand	Seismic Sett. of Layer	Cumulative Seismic Settlement		
	Depth	Depth	Layer Thick- ness							(corrected for C_s and ring->SPT)					σ'_{vo}	σ'_{vo}	σ'_{vo}		$CSR_{7.5}$	CSR_M	(for Settle- ment)			Strain (%) (Tok/ Seed 87)	Strain (%) (Tok/ Seed 87)
	(ft)	(ft)	(ft)							(blows/ft)					(psf)	(psf)	(psf)		(psf)	(psf)	(blows/ft)			(%)	(%)
B-2	0 to 4	2	4		20	116	30	2	1	19.5	232	34.4	40.7	>Range	232	0.58	0.66	NonLiq	40.7	0.02		0.01	0.5		
B-2	4 to 8	6	4		20	119	39	2	1	25.4	702	44.7	51.9	>Range	702	0.57	0.65	NonLiq	51.9	0.03		0.02	0.5		
B-2	8 to 13	10	5		20	125	24	2	1	15.6	1190	24.3	29.8	0.457	1190	0.57	0.65	NonLiq	29.8	0.73		0.39	0.5		
B-2	13 to 18	15	5		20	132	100	2	1	65.0	1833	81.6	91.7	>Range	1832.5	0.56	0.64	NonLiq	91.7	0.03		0.02	0.1		
B-2	18 to 23	20	5		20	113	100	2	1	65.0	2445	78.9	88.8	>Range	2445	0.55	0.63	NonLiq	88.8	0.04		0.03	0.1		
B-2	23 to 27	25	5		4	123	100	2	1	65.0	3035	70.9	70.9	>Range	3035	0.54	0.62	NonLiq	70.9	0.09		0.05	0.0		
B-3	0 to 6	2	6		20	117	30	2	1	19.5	234	34.4	40.7	>Range	234	0.58	0.66	NonLiq	40.7	0.02		0.01	0.1		
B-3	6 to 13	10	7		20	126	48	2	1	31.2	1206	48.3	55.7	>Range	1206	0.57	0.65	NonLiq	55.7	0.08		0.06	0.1		
B-3	13 to 18	15	5		20	132	45	2	1	29.3	1851	36.5	43.0	>Range	1851	0.56	0.64	NonLiq	43.0	0.07		0.04	0.1		
B-3	18 to 22	20	5		20	122	100	2	1	65.0	2486	78.3	88.1	>Range	2486	0.55	0.63	NonLiq	88.1	0.04		0.02	0.0		
B-4	0 to 5	3	5		20	119	30	2	1	19.5	357	34.4	40.7	>Range	357	0.57	0.66	NonLiq	40.7	0.04		0.02	0.3		
B-4	5 to 9	7	4		20	112	14	2	1	9.1	819	16.1	21.0	0.228	819	0.57	0.65	NonLiq	21.0	0.61		0.26	0.3		
B-4	9 to 12	10	3		20	123	51	2	1	33.2	1172	52.0	59.8	>Range	1171.5	0.57	0.65	NonLiq	59.8	0.07		0.03	0.1		
B-4	12 to 15	13	4		20	125	100	2	1	65.0	1544	88.9	99.6	>Range	1543.5	0.56	0.64	NonLiq	99.6	0.06		0.03	0.0		
B-5	0 to 5	3	5		20	123	30	2	1	19.5	369	34.4	40.7	>Range	369	0.57	0.66	NonLiq	40.7	0.04		0.03	0.8		
B-5	5 to 9	7	4		20	117	14	2	1	9.1	849	15.8	20.7	0.224	849	0.57	0.65	NonLiq	20.7	0.65		0.27	0.8		
B-5	9 to 13	10	4		20	102	20	2	1	13.0	1178	20.4	25.6	0.304	1177.5	0.57	0.65	NonLiq	25.6	0.85		0.41	0.5		
B-5	13 to 17	15	5		4	124	40	2	1	26.0	1743	33.5	33.5	>Range	1742.5	0.56	0.64	NonLiq	33.5	0.25		0.13	0.1		

Overexcavation and Compaction

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADINGTable of Contents

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LEIGHTON AND ASSOCIATES, INC.
General Earthwork and Grading Specifications

1.0 General

- 1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).
- 1.2 The Geotechnical Consultant of Record: Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

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- 1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The

Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

- 2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

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If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

- 2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.
- 2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.
- 2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.
- 2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

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3.0 Fill Material

- 3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.
- 3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.
- 3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

- 4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.
- 4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

LEIGHTON AND ASSOCIATES, INC.
General Earthwork and Grading Specifications

- 4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.
- 4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.
- 4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).
- 4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.
- 4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety: The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill: All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.3 Lift Thickness: Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.4 Observation and Testing: The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.