GEOTECHNICAL INVESTIGATION, PROPOSED RESIDENTIAL DEVELOPMENT, TRACT 20337, SOUTH OF BANYAN STREET AND WEST OF LAUREL BLOSSOM PLACE CITY OF RANCHO CUCAMONGA, CALIFORNIA

Prepared For:

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Project No. 12968.001

December 23, 2020
(Revised July 22, 2021)

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Manning Homes, Inc.
20151 SW Birch Street, Suite 150
Newport Beach, California 92660

Attention: Mr. Craig Kozma
Vice President Development

Subject: Geotechnical Investigation, Proposed Residential Development, Tract 20337, South of Banyan Street and West of Laurel Blossom Place, City of Rancho Cucamonga, California

In accordance with Manning Homes, Inc.'s authorization, Leighton and Associates, Inc. (Leighton) has conducted this geotechnical investigation for the proposed residential development of Tract 20337 in the City of Rancho Cucamonga, California. The approximately 5.18-acre site is located south of Banyan Street and west of Luarel Blossom Place. The purpose of this study has been to collect subsurface data at the site, to evaluate the proposed development with respect to the site conditions, and to provide geotechnical recommendations for design and construction of the proposed residential development. This revised report contains several clerical and project description modifications; however, the findings, conclusions, and recommendations from the original report dated December 23, 2020, have not been changed by this report revision.

Based on this investigation, construction of the proposed residential development is feasible from a geotechnical standpoint. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, oversized material, and potentially compressible soils. Good planning and design of the project can limit the impact of these constraints. This report presents our findings, conclusions, and 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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TABLE OF CONTENTS

Figures (Rear of Text)

Appendices

1.0 INTRODUCTION

1.1 Site Location and Description

The property consists of approximately 5.18-acres of land located north of the State Route (SR) 210 freeway, east of Greenwood Place, south of Banyan Street, and west of Laurel Blossom Place in the City of Rancho Cucamonga, California. Single-family residences bound the site on the south, east and west. Site access will be from Banyan Street, which bounds the site on the north.

The site is currently vacant with scattered vegetation throughout. Based on a review of historical aerial imagery, the site appears to have been vacant and undeveloped for the past 26 years. A row of trees previously dividing property lines has been removed.

An elevation survey for the site was not available at the time of this report. Based on elevations obtained from Google Earth, the site drains to the southeast, the highest elevation is approximately 1,547 feet at the northwest corner of the site, and the lowest elevation is approximately 1,511 feet in the southeast corner of the site, an approximate elevation difference of 36 feet.

1.2 Proposed Development

The preliminary site plan depicts nine residential lots with associated streets, sidewalks and utilities. The preliminary site layout shows three open-space areas, Lots A through D, as part of the tract. Lots A and B are locaed eithin the northern portion of the site, and Lot C (park site) is located within the southeastern portion. Lot C is designated for buried infiltration chambers. Vehicle entries will be off Banyan Street. We assume residential units are planned with one- to two-story structures, in addition to drainage, utility, street, sidewalk, landscape and associated improvements.

A preliminaty site plan prepared by MDS Consulting dated September 21, 2020 shows the existing and proposed site grades. The site appears to be planned primarily as cut, with cuts up to 8 feet required to reach proposed grades. Slopes will be constructed at each residential lot, with retaining walls less than 5 feet between lots. Estimated earthwork quantities for the proposed development are 20,075 cubic yards of cut and 3,419 cubic yards of fill, based on the

March 11, 2021 plan by Madole and Associates, Inc., which are raw values without remedial earthwork.

1.3 Purpose of Investigation

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

Our geotechnical exploration included observations and sampling of test pits, laboratory testing, infiltration testing, and geotechnical analysis to evaluate existing geotechnical conditions and develop the conclusions and recommendations contained in this report.

1.4 Scope of Investigation

The scope of our study has included the following tasks:

- Background Review: We reviewed available, relevant geotechnical geologic maps and reports and aerial photographs available from our in-house library or available online.

- Utility Coordination: We contacted Underground Services Alert (USA) prior to excavating test pits so that utility companies could mark public utilities onsite.

- Field Exploration: Our field exploration included excavating test pits and infiltration testing. Logs of the geotechnical test pits and infiltration testing are presented in Appendix B.

  - A total of 6 exploratory test pits were logged and sampled onsite to evaluate subsurface soil conditions. The test pits were excavated to depths ranging from 4.5 to 12.5 feet below the existing ground surface (bgs). The test pits were logged and sampled by our field representative during excavation. Representative bulk soil samples were collected from the test pits for laboratory testing.

  - Pit infiltration tests were conducted within two of our test pits (TP-5a, and TP-5b) to evaluate general infiltration characteristics of subsurface soils at the depths and locations tested. Infiltration tests were conducted in general accordance with the San Bernardino County Stormwater Program

Technical Guidance Document (San Bernardino County, 2011). Tests were conducted at depths of approximately 5 to 6 feet bgs to estimate infiltration rates.

Excavations were backfilled with spoils and tamped with the backhoe bucket. Logs of the geotechnical test pits are presented in Appendix B. Approximate test pit locations are shown on the accompanying Geotechnical Exploration Map, Figure 2.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected 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:

  - In situ moisture content and dry density
  - Maximum dry density and optimum moisture content
  - Sieve analysis for grain-size distribution
  - Expansion index
  - Resistivity, sulfate content, chloride content and pH

  A description of test procedures and results are presented in Appendix C, *Laboratory Test Results*.

- <u>Engineering Analysis</u>: Data obtained from our background review, along with data from our field exploration and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide recommendations presented in this report.

- <u>Report Preparation</u>: Results of our geotechnical exploration have been summarized in this report, presenting our findings, conclusions and geotechnical recommendations for design and construction of the proposed development.

2.0 FINDINGS

2.1 Regional Geologic Conditions

This site is located in the northernmost portion of the Peninsular Ranges Geomorphic Province of southern California, immediately south of the east-west trending Transverse Ranges Geomorphic Province. This is an area of large-scale crustal disturbance as the northward migrating Peninsular Ranges interacts with the Transverse Ranges. Compressional forces associated with this interaction have resulted in uplifting, which has produced the San Gabriel Mountains. The boundary between these two provinces is marked by east-west trending and mountain frontal faults of the Cucamonga fault zone. The frontal faults in the area of the site typically dip northward at shallow inclinations, placing igneous and metamorphic basement rock of the San Gabriel Mountains at the surface with stream channel sediments covering lower portions of the watersheds associated with the upper Santa Ana River and tributaries. The site and surrounding alluvial fan are mapped as Holocene (less than approximately 11,000 years) alluvial fan deposits (Qf and Qyf).

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain by young alluvial fan deposits (see Figure 3, *Regional Geology Map*).

The alluvial soil encountered within our excavations generally consisted of combinations of poorly graded gravel and sand, with some silty sand interspersed. These soils contains high amounts of gravel, cobbles and boulders and tended to be dry to slightly moist. The prevalence of the large clasts indicated that the sediment onsite was originally transported and deposited in a high-energy fluvial environment. The stresses applied to grains in this type of environment would have compacted them tightly together during deposition. Below is a table summarizing gravel and oversize material encountered in our test pits.

Encounterd Gravel and Cobble Amounts

Test Pit	Depth (ft)	% Gravel + Cobbles	No. of Cobbles	Cobble Size Range
TP-1	0 - 3	30 – 40	16	12”-16”
	3 - 10	30 – 35	10	12”-18”
TP-2	0 - 4	40	20	12”-22”
	4 - 4.5	40 - 50	5 – 10	12”-19”
TP-3	0 - 3	5 - 10	--	--
	3 – 5.5	15 - 20	6	12”-14”
	5.5 – 12.5	20 - 25	15 - 20	12”-18”
TP-4	0 – 3	5	--	--
	3 – 5	15	7	12”-16”
	5 – 6.5	20 – 30	2	12”
TP-5a	0 – 2.5	5	--	--
	2.5 – 5	15	7 – 10	12”-14”

More detailed descriptions of the subsurface soil are presented on the test pit logs (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 this study, the upper portion of native soils are considered slightly compressible. Partial removal/recompaction of near surface alluvium is recommended to 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. Because the sediment onsite was transported and deposited in a high-energy fluvial environment, undisturbed alluvial fan deposits are typically dense. Based on this understanding and the observations made in our test pits, the onsite soils are anticipated to have a negligible collapse potential when inundated with water.

2.2.2 Expansive Soils

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.

A near surface sample of the soil was tested for expansion potential. That test result indicated an Expansion Index of 0. Based on our testing the onsite near-surface soil is expected to have a negligible expansion potential.

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 American Concrete Institute (ACI) provisions, adopted by the 2016 CBC (CBC, 2016, Chapter 19, and ACI, 2014).

A near-surface soil sample was tested during this investigation for soluble sulfate content. The result of that test indicated a sulfate content of less than 0.1 percent by weight, indicating negligible sulfate Exposure Class S0. Recommendations for concrete in contact with the soil are provided in Section 3.11.

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. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, representative soil samples were tested during this investigation to determine minimum resistivity, chloride content, and pH. Those tests indicated a minimum resistivity of 12,990 ohm-cm, chloride content of 62 ppm, and pH of 7.6. Based on these results, the onsite soil is considered mildly corrosive to ferrous metals.

2.3 Groundwater

Groundwater was not encountered in any of our test pits excavated to a maximum depth of 12.5 feet bgs during our investigation. Regional groundwater data of State Well 341436N1175539W001 indicates that historically high groundwater at the site vicinity was in the order of 390 feet bgs in 2012 and 2015 (CDWR, 2020). This well is located approximately 0.5 mile northwest of the the site. Additionally, the generalized depth of groundwater in the area in 1960 has been mapped to be between 200 feet and 300 feet bgs (Fife, 1974).

2.4 Faulting and Seismicity

In general, the primary seismic hazards for sites in the region include surface rupture along active faults and strong ground shaking. The potential for fault rupture and seismic shaking are discussed below.

2.4.1 Surface Faulting

The State of California and the County of San Bernardino have mapped the site to be outside of an Earthquake Fault Zone. Additionally, these maps and other published geologic maps have not indicated any fault traces through or trending toward the site. The closest mapped active or potentially active faults are presented in the following table.

Fault Name	Approximate Distance from Site
Cucamonga Fault	4.4 miles to the northeast
San Jacinto	5.9 miles to the northeast
San Andreas-San Bernardino	10 miles to the northeast

A listing of active faults within a 60-mile search radius is presented in Appendix D. Based on our understanding of the current geologic framework, the potential for future surface rupture of active faults onsite is considered very low.

2.4.2 Seismic Design Parameters

Based on current understanding of local faulting, the principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in southern California. The project should be designed in accordance with applicable current building codes and standards utilizing appropriate seismic design parameters intended to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117A (CGS, 2008). The following are seismic design parameters for new structures based on the 2019 California Building Code (CBC). The mapped-based seismic parameters presented were obtained from United States Geological Survey in accordance with American Society of Civil Engineers (ASCE) Publication ASCE 7-16 and the 2019 CBC, Chapter 16. The following table should be considered for design under the 2019 CBC:

2019 CBC Parameters (CBC or ASCE 7-16 reference)	**Value 2019 CBC**
Site Latitude and Longitude: 34.1424, -117.5262	
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	D**
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_s	1.809 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.613 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.000
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	*1.700**
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.809 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	*1.042** g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	1.206 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), 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}	1.100
Site-Modified Peak Ground Acceleration (1803.5.12; 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 1613.2.3(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 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.

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 7.9 (M_W) at a distance on the order of 11.3 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years), and corresponding peak ground acceleration of 0.88 g.

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 the loss of soil strength or stiffness due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine-to-medium grained, cohesionless soils. As the shaking action of an earthquake progresses, the soil grains are rearranged and the soil densifies within a short period of time. Rapid densification of the soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, the soil reduces greatly in strength and temporarily behaves similarly to a fluid. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

The State of California has not prepared a map delineating zones of liquefaction potential for the quadrangle that contains the site. The County of San Bernardino has mapped the site to be outside a zone of generalized liquefaction susceptibility (San Bernardino County, 2010). Collected data indicated that groundwater depths at and near this site have been historically roughly 200 feet to 300 feet deep beneath the site. Based on the absence of shallow groundwater and the existence of dense granular soil onsite, liquefaction is unlikely to occur at the site.

2.5.2 Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). During a strong seismic event, seismically induced settlement can occur within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement.

Based on the dense nature of the alluvial deposits in this area and considering the recommended earthwork overexcavation requirements presented later in this report, we believe the onsite soils are not susceptible to significant seismically induced settlements.

2.6 Infiltration Testing

A total of two infiltration tests were conducted in select locations of the proposed development to estimate the infiltration rate of native soils. Pit infiltration tests were conducted in test pits TP-5a, and TP-5b. These infiltration tests were performed within poorly graded gravel with sand at approximately 5 to 6 feet bgs.

The smaller pits within the larger excavated pit tested were excavated with approximate dimensions of 4 feet long, 5 feet wide, and 2 feet deep (refer to Appendix B for photo documentation of the test pit witin the larger pit). Once each open-pit was excavated, a perforated cylinder casing was placed inside and then each open-pit was backfilled with gravel around the casings within the test zone and with onsite soils above the test zone to existing grade. Each well was then steadily filled with water and flow was measured over time. Infiltration test results are presented in Appendix B.

Infiltration rates were measured at the two tested locations and ranged from approximately 4.2 to 6.0 inches per hour (no factor of safety or correction factors applied) at the depths tested. Water was observed to infiltrate rapidly into native soils, with the water level remaining at steady levels near the bottom of the well. A conservative cross-sectional area was utilized to derive the approximate infiltration rates, based on the well construction method and observed rapid infiltration; actual rates are anticipated to be higher than the conservative estimates noted herein. See Section 3.7 for a discussion of infiltration characteristics and considerations.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, construction of the proposed 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, oversized material, the presence of artificial fill, and the existence of potentially compressible soils. 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.

Although not identified during this investigation, abandoned septic tanks, seepage pits, or other buried structures, trash pits, or items related to past site uses may be present. As such items are encountered during grading, they will require further evaluation and special consideration.

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 debris, which should be disposed of offsite. Any underground obstructions should be removed. 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 Removal of Uncontrolled Artificial Fill

Prior to overexcavation and recompaction of the onsite alluvial soil, any clean uncontrolled artificial fill should be removed and may be used as compacted fill for the project.

3.1.3 Overexcavation and Recompaction

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

All artificial fill should be removed to firm native soil. In addition, we recommend that the onsite soils in areas of proposed structures to be overexcavated to a minimum depth of 3 feet below the existing ground surface or 18 inches below the bottom of the proposed footings, whichever is deeper. Where possible, the removal bottom should extend horizontally a minimum of 5 feet from the outside edges of the footings, or a distance equal to the depth of overexcavation below the footings, whichever is farther. During overexcavation, the soil conditions should be observed by Leighton to further evaluate these recommendations based on actual field conditions encountered. A firm removal bottom should be established across the building footprint to provide uniform foundation support for the proposed structure. Leighton should observe and test the removal bottom prior to placing fill. Deeper overexcavation and recompaction may be recommended locally until a firm removal bottom is achieved.

Areas outside the overexcavation limits of structures planned for asphalt or concrete pavement, flatwork, sidewalks, and areas to receive fill should be overexcavated a minimum depth of 12 inches below the existing ground surface or 12 inches below the proposed subgrade, whichever is deeper.

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 or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D1557 laboratory maximum density.

3.1.4 Fill Placement and Compaction

Onsite soil to be used for compacted structural fill should also be free of organic material 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 to near optimum moisture content, and compacted to a minimum 90 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.

3.1.5 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.

3.1.6 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 and our experience. We preliminarily estimate the following earth volume changes will occur during grading:

Shrinkage	Approximately 8 +/-3 percent
Subsidence (overexcavation bottom processing)	Approximately 0.1 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.

3.1.7 Rippability and Oversized Material

Oversized material (rock or rock fragments greater than 12 inches in dimension) was observed during our investigation. Oversized material should not be used within structural fill areas. Section 2.2 of our report includes a table summarizing the amount of cobbles encountered at each of our test pits.

Oversize material should not be buried unless specifically approved by Leighton. For this site we recommend that no rock larger than 12 inches in largest dimension be placed as compacted fill.. Any oversized material larger than 12 inches should either be reduced in size, hauled offsite, or buried in deeper fills. The owner may wish to limit the amount of larger rocks in planned utility trench areas, to facilitate the construction of utilities.

3.2 Shallow Foundation Recommendations

Overexcavation and recompaction of the footing subgrade should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a negligible to very low expansion potential.

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment per code requirements, with a minimum width of 24 and 12 inches for isolated and continuous footings, respectively.

3.2.2 Allowable Bearing

An allowable bearing pressure of 2,000 pounds-per-square-foot (psf) may be used, based on an assumed embedment depth of 18 inches and minimum width described 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.

3.2.3 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 0.4. The passive resistance may be computed using an allowable equivalent fluid pressure of 300 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.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

The allowable bearing pressure and coefficient of friction values may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces.

3.2.5 Settlement Estimates

The recommended allowable bearing pressure is generally based on a total allowable, post-construction static settlement of 1 inch. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 40 feet. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for soil with a very 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 10-mil moisture retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a a sand blotter layer should be placed over the vapor retarder. The moisture barrier may be placed directly on subgrade provided gravel or other protruding objects that could puncture the moisture retarder are removed from the subgrade prior to placement. A heavier vapor retarder (such as 15 mil Stego Wrap) placed directly on prepared subgrade may also be used. Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Institute, ASTM International, and California Building Code requirements and guidelines.

- 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, 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 concrete cracking. The structural engineer should consider these components in slab design and specifications.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Floor covering manufacturers should be consulted for specific recommendations.

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.

3.4 Seismic Design Parameters

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the current CBC. The CBC seismic design parameters listed in Section 2.4.2 of this report should be considered for the seismic analysis of the subject site.

3.5 Retaining Walls

We understand that retaining walls onsite will have heights shorter than 6 feet. We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 4 (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	2:1 Backfill
Active	35	57 pcf
At-Rest	56	86 pcf
Passive	300 (Maximum of 3,000 pcf)	N/A

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 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 0.4 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 seismic increment load of 43 pcf (equivalent fluid pressure) should be added to the active case when checking seismic stability of walls over 6 feet tall.

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

3.6 Pavement Design

Based on the design procedures outlined in the current Caltrans Highway Design Manual, and using an assumed design R-value of 50, flexible pavement sections may consist of the following for the Traffic Index indicated. 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		
Traffic Index	**Asphaltic Concrete (AC) Thickness (inches)**	**Class 2 Aggregate Base Thickness (inches)**
5 or less	3	4
7	4	4

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.

PCC sidewalks should be at least 4 inches thick over prepared subgrade soil, with construction joints no more than 8 feet on center each way, with sections as nearly square as possible. Use of reinforcing will help reduce severity of cracking.

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction. 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 recompacted to a minimum of 90 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

3.7 Infiltration Recommendations

Infiltration Rate:

Infiltration testing performed in test pits TP-1a and TP-1b were within sands to silty sands with gravel layers. For onsite alluvial soils that are granular with a low fines content and that are approximately 5 feet deep or deeper, we recommend an unfactored (small-scale) incremental infiltration rate of 5 inches per hour. These rates are applicable at the specific locations and depths indicated. Infiltration rates may vary significantly at various depths or locations across the site. It is anticipated that higher rates can be obtained if dry wells are used.

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.

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 can 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 overfilling 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.

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.

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 can 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. 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 in accordance with Greenbook specifications. The native 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 the results of laboratory testing, concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil (Exposure Class S0). There is no cement type restriction for Exposure Class S0 per ACI 318. Concrete should be designed in accordance with ACI 318-14, Section 19.3 (ACI, 2014), adopted by the 2019 CBC (Section 1904.2).

The onsite soil is considered to be mildly corrosive to ferrous metals. Corrosion information presented in this report should be provided to the underground utility subcontractors.

3.12 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our supplemental 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 and Associates, Inc. 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. Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of Manning Homes, Inc. 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.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Project: 12968.001	Eng/Geol: JDH
Scale:1 " = 2,000 '	Date: December 2020

Base Map: ESRI ArcGIS Online 2020
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

SITE LOCATION MAP

Manning Homes, Tract 20337
South of Banyan Street and South Blossom Place
Rancho Cucamonga, California

Figure 1

Leighton

Map Saved as V:\Drafting\12968\001\Maps\12968-001_F01_SLM_2020-12-18.mxd on 12/18/2020 12:58:01 PM

Project: 12968.001	Eng/Geol: JDH
Scale:1 " = 100 '	Date: December 2020

Base Map: ESRI ArcGIS Online 2020
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

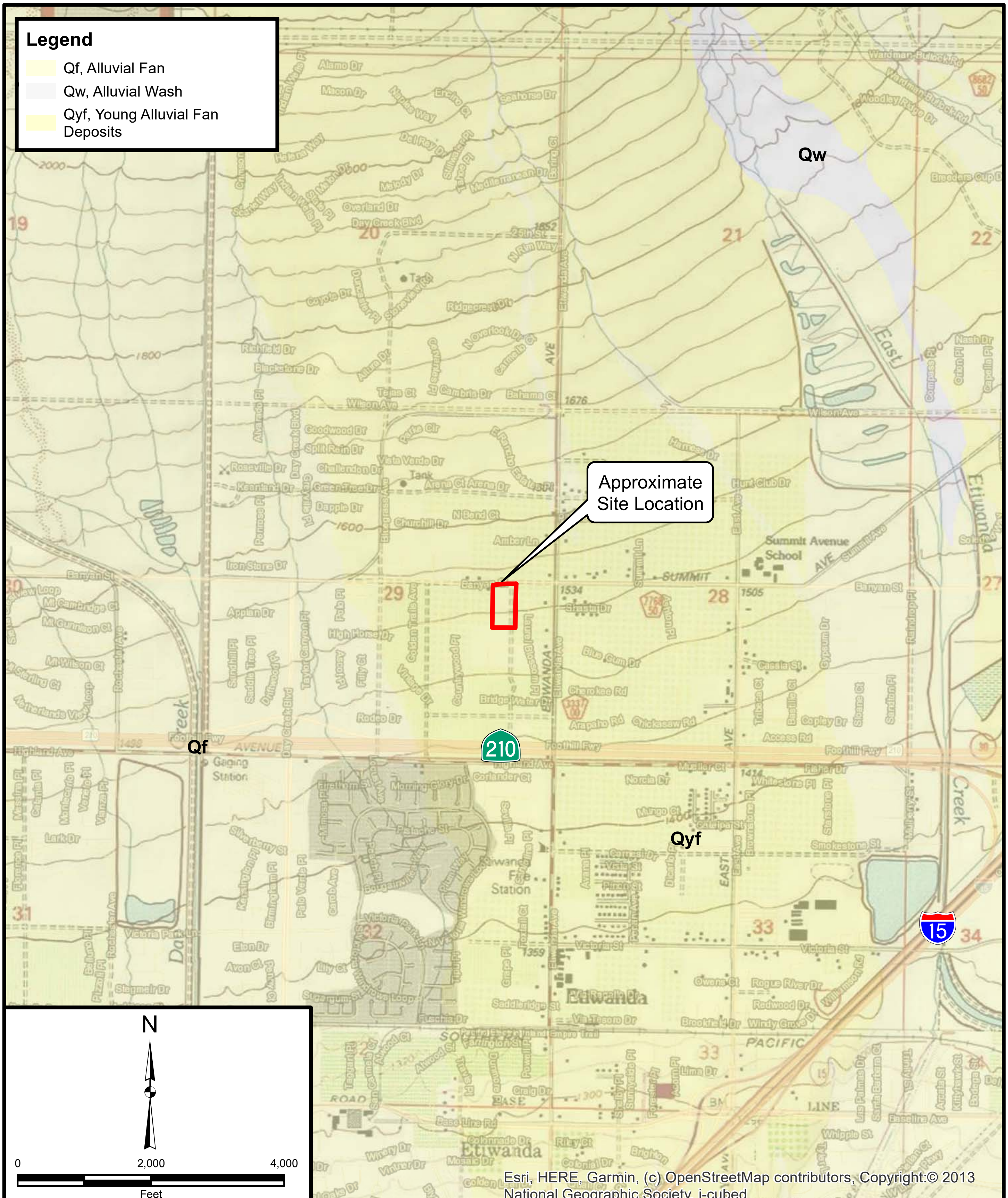
GEOTECHNICAL EXPLORATION MAP

Manning Homes, Tract 20337
South of Banyan Street and South Blossom Place
Rancho Cucamonga, California

Figure 2

Leighton

Map Saved as V:\Drafting\12968\001\Maps\12968-001_F02_GEM_2020-12-03.mxd on 12/3/2020 9:50:58 AM



Project: 12968.001	Eng/Geol: JDH
Scale:1 " = 2,000 '	Date: December 2020

Base Map: Geologic Compilation of Quaternary Surficial Deposits in Southern California, Orange County, July 2010
Thematic Information: Leighton, USGS
Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

Manning Homes, Tract 20337
South of Banyan Street and South Blossom Place
Rancho Cucamonga, California

Figure 3

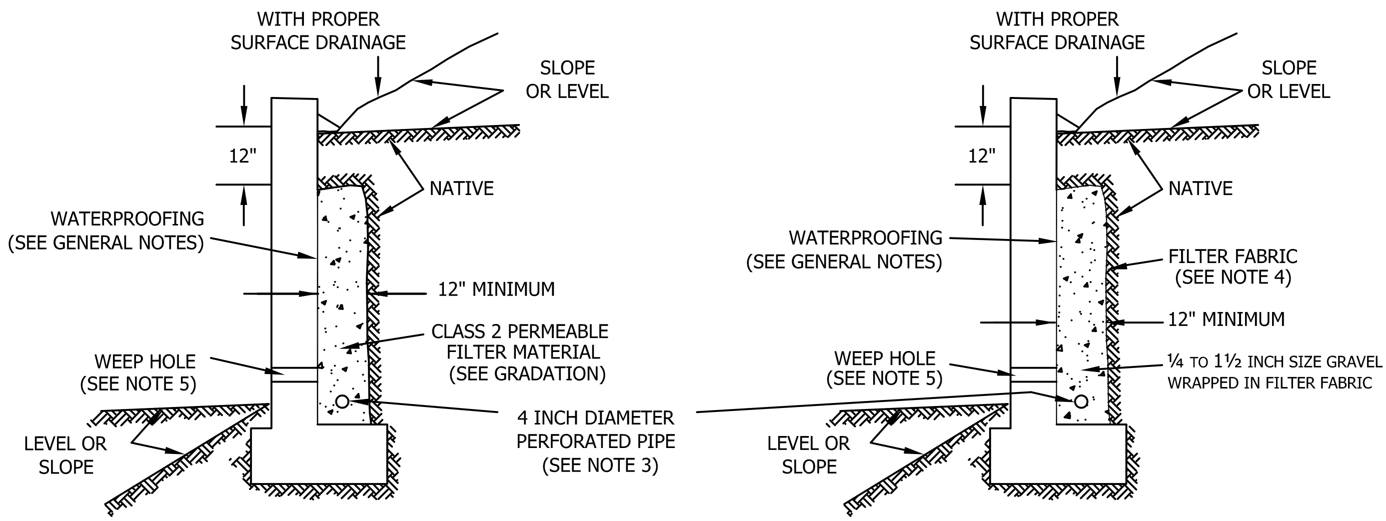
Leighton

Map Saved as V:\Drafting\12968\001\Maps\12968-001_F03_RGM_2020-12-18.mxd on 12/18/2020 12:56:58 PM

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

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
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) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes 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

APPENDIX A

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APPENDIX B

GEOTECHNICAL LOGS

APPENDIX B

FIELD EXPLORATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration. Six test pits (TP-1 through TP-5b) were excavated and logged to a maximum depth of approximately 12.5 feet below the existing ground surface. These test pit logs are included as part of this appendix. Approximate test pit locations are shown on Figure 2, *Geotechnical Exploration Map*.

Test Pits: On November 19, 2020, 6 test pits were excavated, logged, and sampled to depths ranging from 4.5 feet to 12.5 feet below the ground surface. Encountered soils were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Near surface bulk soil samples were also collected from the borings. Representative earth-material samples obtained from these subsurface explorations were transported to our geotechnical laboratory for evaluation and appropriate testing.

APPENDIX B

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TEST PIT TP-1

Manning Homes Banyan Street

Logged By: ECB

Sampled By: ECB

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

Project No. 12968.001

Date Excavated: 11/19/2020

Elevation: 1541'

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 (%)
			surface: silty sand, gravel, cobbles, boulders					
0.0	3.0	SM	**Alluvium (Qal):** SILTY SAND with gravel, cobbles, and boulders: light brown, dry, 15% fines (field estimate), fine to coarse sand, 30-40% gravel and cobbles, loose, rootlets **Boulders:** approx 16 total ranging from 12-16" in dimension	**Qal**	B1	0-3'		4.3
3.0	10.0	SP	**Alluvium (Qal):** SAND with gravel, cobbles and boulders: yellow brown, slightly moist, <5% fines (field estimate), fine to coarse sand, 30-35% gravel and cobbles, medium dense **Boulders:** approx 10 total ranging from 12-18" in dimension	**Qal**	B2	8-10'		3.9

Total Depth = 10.0 feet
No groundwater encountered when excavating
Test pit back-filled and tamped with spoils on November 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-2

Manning Homes Banyan Street

Logged By: ECB

Sampled By: ECB

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

Project No. 12968.001

Date Excavated: 11/19/2020

Elevation: 1537'

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 (%)
			surface: sand, gravel, cobbles, boulders, dry brush					
0.0	4.0	SM	**Alluvium (Qal):** SILTY SAND with gravel, cobbles, and boulders: brown, dry, 20% fines (field estimate), fine to coarse sand, 40% gravel and cobbles, loose, rootlets **Boulders:** approx 20 total ranging from 12-22" in dimension	**Qal**				
4.0	4.5	SP	**Alluvium (Qal):** SAND with gravel, cobbles and boulders: yellow brown, slightly moist, 5% fines (field estimate), fine to coarse sand, 40-50% gravel and cobbles, loose **Boulders:** approx 5-10 total ranging from 12-19" in dimension	**Qal**				

Total Depth = 4.5 feet
No groundwater encountered when excavating
Test pit back-filled and tamped with spoils on November 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-3

Manning Homes Banyan Street
Logged By: ECB
Sampled By: ECB
Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12968.001
Date Excavated: 11/19/2020
Elevation: 1527'

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 (%)
			surface: sand, gravel, boulders					
0.0	3.0	SM	**Alluvium (Qal):** SILTY SAND with gravel: dark brown, dry, 15% fines (field estimate), fine to medium sand, 5-10% coarse gravel, no cobbles or boulders, loose, rootlets	**Qal**	B1	0-3'		6.4
3.0	5.5	SM	**Alluvium (Qal):** SILTY SAND with gravel, cobbles and boulders: brown, slightly moist, 20% fines (field estimate), fine to coarse sand, 15-20% gravel and cobbles, loose	**Qal**				
			Boulders: approx 5-6 total ranging from 12-14" in dimension					
5.5	12.5	SP	**Alluvium (Qal):** SAND with gravel, cobbles and boulders: yellow brown, dry, 5% fines (field estimate), fine to coarse sand, 20-25% gravel and cobbles, medium dense	**Qal**				
			Boulders: approx 15- 20 total ranging from 12-18" in dimension					

Total Depth = 12.5 feet
No groundwater encountered when excavating
Test pit back-filled and tamped with spoils on November 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-4

Manning Homes Banyan Street
Logged By: ECB
Sampled By: ECB
Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12968.001
Date Excavated: 11/19/2020
Elevation: 1522'

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 (%)
			surface: sand, gravel, boulders					
0.0	3.0	SM	**Alluvium (Qal):** SILTY SAND with gravel: dark brown, slightly moist, 20% fines (field estimate), fine to medium sand, 5% coarse gravel, no cobbles or boulders, loose	**Qal**				
3.0	5.0	SM	**Alluvium (Qal):** SILTY SAND with gravel, cobbles and boulders: brown, slightly moist, 20-30% fines (field estimate), fine to coarse sand, 15% gravel and cobbles, loose	**Qal**				
			Boulders: approx 7 total ranging from 12-16" in dimension					
5.0	6.5	SP	**Alluvium (Qal):** SAND with gravel, cobbles and boulders: yellow brown, slightly moist, 5-10% fines (field estimate), meidum to coarse sand, 20-30% gravel and cobbles, medium dense	**Qal**				
			Boulders: approx 2 total 12" in dimension					

Total Depth = 6.5 feet
No groundwater encountered when excavating
Test pit back-filled and tamped with spoils on November 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-5a

Manning Homes Banyan Street
Logged By: ECB
Sampled By: ECB
Location: (see Figure 2, *Geotechnical Exploration Map*)

Project No. 12968.001
Date Excavated: 11/19/2020
Elevation: 1515'

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 (%)
			surface: silty sand, gravel, cobbles, boulders					
0.0	2.5	SM	**Alluvium (Qal):** SILTY SAND with gravel: dark brown, 30% fines (field estimate), 5% gravel, no cobbles or boulders, very loose	**Qal**				
2.5	5.0	SM	**Alluvium (Qal):** SILTY SAND with gravel, cobbles and boulders: brown, slightly moist, 20-30% fines (field estimate), fine to coarse sand, 15% gravel and cobbles, loose	**Qal**				
			Boulders: approx 7-10 total ranging from 12-14" in dimension					
			Percolation Pit Testing Area: 3-5' depth, Area of infiltration chamber 4x5x2', 500 lbs of pea gravel to fill chamber, backfilled around test zone with spoils					

Total Depth = 5.0 feet
No groundwater encountered when excavating
Test pit back-filled and tamped with spoils on November 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-5b

Manning Homes Banyan Street Project No. 12968.001

Logged By: ECB Date Excavated: 11/19/2020

Sampled By: ECB Elevation: 1515'

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

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 (%)
			surface: silty sand, gravel, cobbles, boulders, grass					
0.0	1.0	SM	**Alluvium (Qal):** SILTY SAND with gravel: dark brown, slightly moist 30% fines (field estimate), 5% gravel, no cobbles or boulders, loose	**Qal**				
2.5	4.0	SM	**Alluvium (Qal):** SILTY SAND with gravel, cobbles and boulders: brown, 15% fines (field estimate), fine to coarse sand, 20-30% gravel and cobbles, loose	**Qal**				
4.0	6.0	SP	**Alluvium (Qal):** SAND with gravel, cobbles and boulders: yellow brown, slightly moist, 5% fines (field estimate), fine to coarse sand, 30% gravel and cobbles, dense	**Qal**	B1	4-6'		2.3
			Boulders: approx 5-6 total ranging from 12-14" in dimension					
			Percolation Pit Testing Area: 4-6' depth, Area of infiltration chamber 4x5x2', 500 lbs of pea gravel to fill chamber, backfilled around test zone with spoils					

Total Depth = 6.0 feet
No groundwater encountered when excavating
Test pit back-filled and tamped with spoils on November 19, 2020

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

Results of Well Permeameter, from USBR 7300-89 Method

Leighton

Project:	12968.001		Initial estimated Depth to Water Surface (in.):	48
Exploration #/Location:	*TP-5A*		Average depth of water in well, "h" (in.):	21
Depth Boring drilled to (ft):	5		approx. h/r:	1.2
Tested by:	LP		Tu (Fig. 8) (ft):	56.0
USCS Soil Type in test zone:			Tu>3h?:	yes, OK
Weather (start to finish):	SP-SM			
Liquid Used/pH:	from fire hydrant with meter (permited)			
Measured boring diameter (effective):	36	in. 18 in. Well Radius	Cross-sectional area for vol calcs (in.^2):	407.1
Approx Depth to GW BGS:	60	ft (GW or aquatarde)		

Well Prep: Dug TP to 3', Excavate 4'x5'x2' to 5'; pea gravel at bottom; pipe and backfill test zone w/pea gravel; backfill to surface

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	5. ft	9. in.	69	
Pilot Tube stickup (+ is above ground)		19. in.	19	
Depth to top of sand outside of casing from top of pilot tube				
Depth to top of DH valve/float assembly from top of pilot tube		31. in.	31	12 Depth below GS (in.)
Float Assembly ID		DHVA		
Float assembly Extension length (in.)		0		

Use of Barrels:	No
Use of Flow Meter:	Yes
Use of DH Valve:	Yes
Test Type:	Constant Head

Flow Meter:

Meter ID	3242
Meter Col	Black
Meter Unit	Gallons
DL ID	1

0.05 gallons/pulse

Field Data / **Calculations**

Date	Time	Data from Flow Meter		Depth to WL in Boring (measured from top of pilot tube)		Water Temp (deg F)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3)			Flow (in^3/min)	q, Flow (in^3/hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date	Start time:	Reading (cu-ft or gal)	Interval Pulse Count	ft	in.									from supply	from Δh	Total					
11/20/2020	11:40	Gallons																			
11/20/20	11:40	6071.6		5.7					0	49.4	19.6										
11/20/20	11:45	6078.1		5.7				5	5	49.4	19.6	0	20	1502	0	1502	300	18018	3235	0.9	5.14
11/20/20	11:50	6084.9		5.6				5	10	48.2	20.8	1.2	20	1571	-489	1082	216	12987	3302	0.9	3.63
11/20/20	11:55	6091.2		5.55				5	15	47.6	21.4	0.6	21	1455	-244	1211	242	14532	3404	0.9	3.94
11/20/20	12:00	6097.9		5.5				5	20	47.0	22.0	0.6	22	1548	-244	1303	261	15641	3472	0.9	4.15
11/20/20	15:15	6353		5.1				195	215	42.2	26.8	4.8	24	58928	-1954	56974	292	17530	3777	0.9	4.28
																			Minimum Rate:		3.63
								Raw Rate for design, prior to application of adjustment factors:													**1.20**

Results of Well Permeameter, from USBR 7300-89 Method

Leighton

Project: 12968.001

Exploration #/Location: *TP-5B*

Depth Boring drilled to (ft): 5

Tested by: LP

USCS Soil Type in test zone:

Weather (start to finish): SP

Liquid Used/pH: from fire hydrant with meter (permited)

Measured boring diameter (effective): 36 in. 18 in. Well Radius

Approx Depth to GW BGS: 60 ft (GW or aquatarde)

Well Prep: Dug TP to 4', Excavate 4'x5'x2' to 6'; pea gravel at bottom; pipe and backfill test zone w/pea gravel; backfill to surface

Initial estimated Depth to Water Surface (in.): 55

Average depth of water in well, "h" (in.): 19

approx. h/r: 1.1

Tu (Fig. 8) (ft): 55.4

Tu>3h?: yes, OK

Cross-sectional area for vol calcs (in.^2): 407.1

Use of Barrels: No

Use of Flow Meter: Yes

Use of DH Valve: Yes

Test Type: Constant Head

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	6. ft	2. in.	74	
Pilot Tube stickup (+ is above ground)		12. in.	12	
Depth to top of sand outside of casing from top of pilot tube				
Depth to top of DH valve/float assembly from top of pilot tube		36. in.	36	24 Depth below GS (in.)
Float Assembly ID		DHVA		
Float assembly Extension length (in.)		0		

Flow Meter:

Meter ID: 3242

Meter Col: Black

Meter Unit: Gallons

DL ID: 1

0.05 gallons/pulse

Field Data | **Calculations**

Date	Time	Data from Flow Meter: Reading (cu-ft or gal)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of pilot tube): ft	Depth to WL in Boring: in.	Water Temp (deg F)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3): from supply	Vol Change (in.^3): from Δh	Vol Change (in.^3): Total	Flow (in^3/min)	q, Flow (in^3/hr)	V (Fig 9)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date	Start time:	Gallons																		
11/20/2020	15:40																			
11/20/20	15:40	6381.1		5.8			Valve		0	57.6	16.4									
11/20/20	15:45	6392		5.7				5	5	56.4	17.6	1.2	17	2518	-489	2029	406	24352	0.9	7.63
11/20/20	15:50	6399.9		5.65				5	10	55.8	18.2	0.6	18	1825	-244	1581	316	18967	0.9	5.75
11/20/20	16:35	6483.4		5.45				45	55	53.4	20.6	2.4	19	19289	-977	18311	407	24415	0.9	7.01
11/20/20	16:40	6492.2		5.4				5	60	52.8	21.2	0.6	21	2033	-244	1789	358	21462	0.9	5.85
11/20/20																				
11/20/20	16:43	6495.5		5.55			Full Hose		63	54.6	19.4									
11/20/20	16:47	6501.6		5.5			28 psi	4	67	54.0	20.0	0.6	20	1409	-244	1165	291	17472	0.9	4.96
11/20/20	16:50	6509		5.4				3	70	52.8	21.2	1.2	21	1709	-489	1221	407	24417	0.9	6.72

Minimum Rate: 4.96

Raw Rate for design, prior to application of adjustment factors: **1.20**

APPENDIX C

LABORATORY TEST RESULTS

APPENDIX C

GEOTECHNICAL LABORATORY TESTING

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

In-Situ Moisture: The natural water content (ASTM D 2216) was determined for recovered bulk soil samples, from our subsurface explorations. Results of these tests are shown on the logs at the appropriate sample depths, in Appendix B.

Modified Proctor compaction Curve: A laboratory modified Proctor compaction test (ASTM D 1557) was performed on a bulk soil sample to determine maximum laboratory dry density and optimum moisture content. Result of this test is presented on the following "*Modified Proctor Compaction Test*" plot in this appendix.

Sieve Analysis: Sieve analyses (ASTM D 422) were performed on selected subsurface soil samples. These tests were performed to assist in the classification of the soil. Results of these tests are presented on the "*Particle Size Analysis of Soils*" figures.

Expansion Index: Expansion Index of a representative bulk sample was determined by the ASTM D 4829 standard test method to identify expansion potential. The expansion index is presented in this appendix.

Corrosivity Tests: To evaluate the corrosion potential of the subsurface soils at the site, we tested representative bulk samples collected during our subsurface investigation for pH, resistivity and soluble sulfate and chloride content testing. Results of these tests are presented at the end of this appendix.

Leighton

MOISTURE CONTENT

ASTM D 2216

Project Name: **Manning Homes Banyan St Residential**
Project No.: **12968.001**

Tested By: Y. Nguyen
Date: 12/07/20
Checked By: A. Santos
Date: 12/10/20

Boring No.	TP-1	TP-1	TP-3	TP-5B	
Sample No.	B-1	B-2	B-1	B-1	
Depth (ft)	0-3	8-10	0-3	4-6	
Sample Type	Bulk	Bulk	Bulk	Bulk	
Sample Description	Olive brown silty sand with gravel (SM)g, noted organics	Olive poorly-graded sand with gravel (SP)g	Olive brown poorly-graded sand with silt and gravel (SP-SM)g	Light olive brown poorly-graded gravel with sand (GP)s	
Wt. wet soil + container (g)	5938.50	727.90	6218.10	12863.80	
Wt. dry soil + container (g)	5702.80	703.40	5885.80	12586.70	
Weight of container (g)	231.00	77.75	731.60	777.10	
Moisture Content (%)	**4.3**	**3.9**	**6.4**	**2.3**	

Boring No.					
Sample No.					
Depth (ft)					
Sample Type					
Sample Description					
Wt. wet soil + container (g)					
Wt. dry soil + container (g)					
Weight of container (g)					
Moisture Content (%)					

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Leighton

Project Name:	Manning Homes Banyan St Residential	Tested By:	G. Bathala	Date:	12/08/20
Project No.:	12968.001	Checked By:	A. Santos	Date:	12/11/20
Boring No.:	TP-3	Depth (feet):	0-3		
Sample No.:	B-1				
Soil Identification:	Olive brown poorly-graded sand with silt and gravel (SP-SM)g				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	SP-02	57	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	5885.8	602.8	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	731.6	107.3	Wt. of Container No.______(g)	1.0	1.0
Dry Wt. of Soil (g)	5154.2	495.5	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve		
	Container No.	57
	Wt. of Dry Soil + Container (g)	540.6
	Wt. of Container (g)	107.3
	Dry Wt. of Soil Retained on # 200 Sieve (g)	433.3

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
6"	152.400			
3"	75.000	0.0		100.0
1 1/2	37.500	901.5		82.5
1"	25.000	1109.9		78.5
3/4"	19.000	1195.4		76.8
1/2"	12.500	1431.7		72.2
3/8"	9.500	1527.4		70.4
#4	4.750	1720.3		66.6
#8	2.360		28.0	62.8
#16	1.180		67.3	57.6
#30	0.600		140.3	47.7
#50	0.300		247.3	33.4
#100	0.150		349.0	19.7
#200	0.075		426.8	9.2
PAN				

GRAVEL: **33 %**

SAND: **58 %**

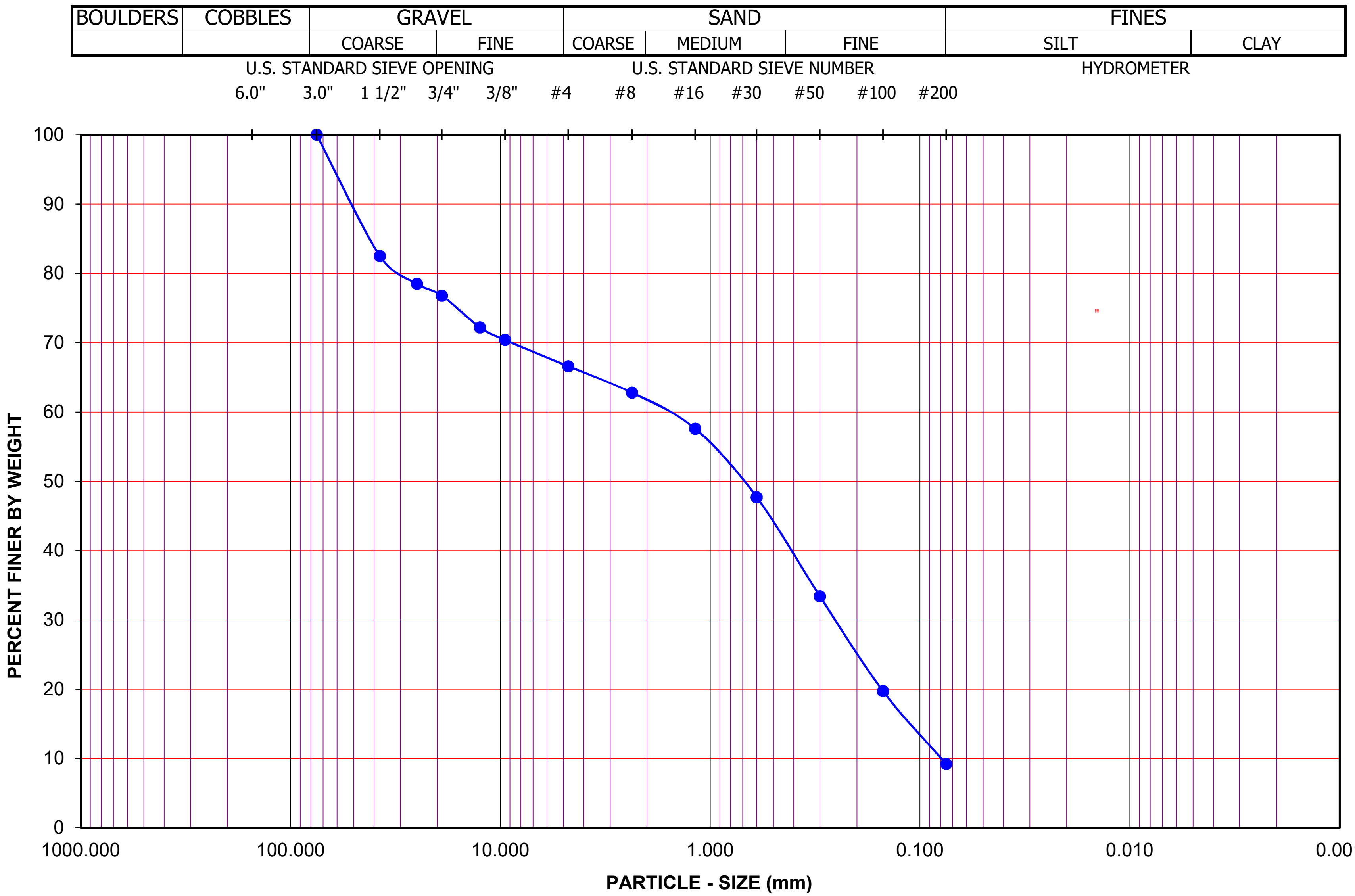
FINES: **9 %**

GROUP SYMBOL: **(SP-SM)g**

$Cu = D60/D10 =$ 20.00

$Cc = (D30)^2/(D60*D10) =$ 0.53

Remarks:



Project Name: Manning Homes Banyan St Residential

Project No.: 12968.001

Boring No.: TP-3 Sample No.: B-1

Depth (feet): 0-3 Soil Type : (SP-SM)g

Soil Identification: Olive brown poorly-graded sand with silt and gravel (SP-SM)g

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 33 : 58 : 9

Dec-20

Sieve TP-3, B-1 @ 0-3

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Leighton

Project Name:	Manning Homes Banyan St Residential	Tested By:	G. Bathala	Date:	12/08/20
Project No.:	12968.001	Checked By:	A. Santos	Date:	12/15/20
Boring No.:	TP-5B	Depth (feet):	4-6		
Sample No.:	B-1				
Soil Identification:	Light olive brown poorly-graded gravel with sand (GP)s				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	SP-04	923	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	12586.7	606.1	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	777.1	108.2	Wt. of Container No.______(g)	1.0	1.0
Dry Wt. of Soil (g)	11809.6	497.9	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve		
	Container No.	923
	Wt. of Dry Soil + Container (g)	564.1
	Wt. of Container (g)	108.2
	Dry Wt. of Soil Retained on # 200 Sieve (g)	455.9

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
6"	152.400	0.0		100.0
3"	75.000	1328.1		88.8
1 1/2	37.500	3714.8		68.5
1"	25.000	4856.7		58.9
3/4"	19.000	5411.0		54.2
1/2"	12.500	6259.8		47.0
3/8"	9.500	6662.5		43.6
#4	4.750	7389.8		37.4
#8	2.360		61.1	32.8
#16	1.180		113.4	28.9
#30	0.600		182.9	23.7
#50	0.300		291.1	15.5
#100	0.150		388.0	8.3
#200	0.075		450.7	3.5
PAN				

GRAVEL: **63 %**

SAND: **33 %**

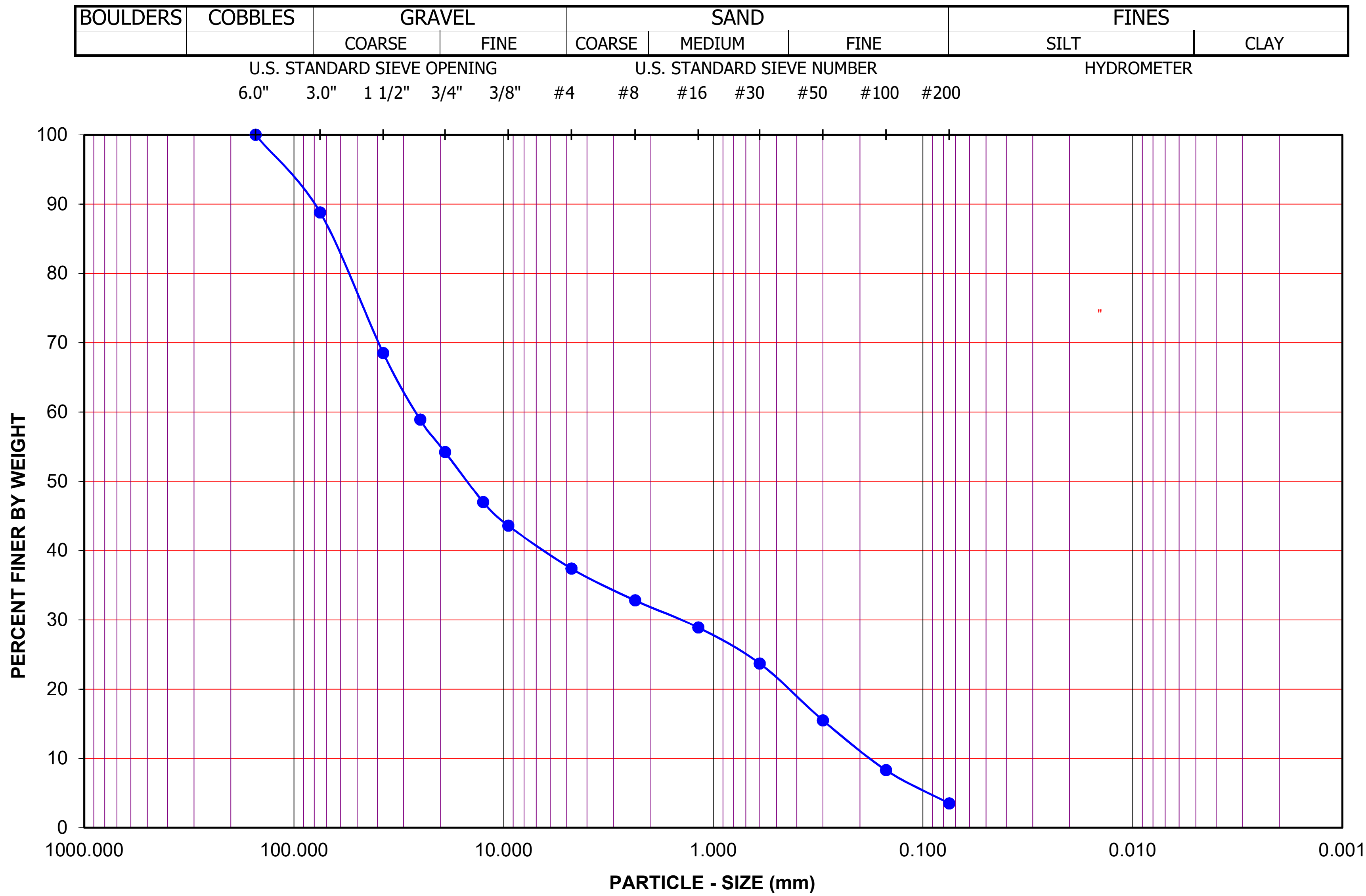
FINES: **4 %**

GROUP SYMBOL: **(GP)s**

Cu = D60/D10 = 150.00

Cc = $(D30)^2/(D60*D10)$ = 0.46

Remarks:



Project Name: Manning Homes Banyan St Residential

Project No.: 12968.001

Boring No.: TP-5B   Sample No.: B-1

Depth (feet): 4-6   Soil Type : (GP)s

Soil Identification: Light olive brown poorly-graded gravel with sand (GP)s

GR:SA:FI : (%) 63 : 33 : 4

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Dec-20

EXPANSION INDEX of SOILS

ASTM D 4829

Leighton

Project Name:	Manning Homes Banyan St. Residential	Tested By:	J. Gonzalez	Date:	12/10/20
Project No.:	12968.001	Checked By:	A. Santos	Date:	12/15/20
Boring No.:	TP-1	Depth (ft.):	N/A		
Sample No.:	B-1				
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	0.9995
Wt. Comp. Soil + Mold (g)	547.00	408.80
Wt. of Mold (g)	163.40	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	778.10	572.20
Dry Wt. of Soil + Cont. (g)	701.00	508.99
Wt. of Container (g)	0.00	163.40
Moisture Content (%)	11.00	18.29
Wet Density (pcf)	115.7	123.4
Dry Density (pcf)	104.2	104.3
Void Ratio	0.617	0.616
Total Porosity	0.382	0.381
Pore Volume (cc)	79.0	78.9
Degree of Saturation (%) [S_{meas}]	**48.1**	80.1

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.)
12/10/20	15:55	1.0	0	0.5785
12/10/20	16:05	1.0	10	0.5780
Add Distilled Water to the Specimen				
12/10/20	16:10	1.0	5	0.5780
12/12/20	12:50	1.0	2685	0.5780
12/12/20	13:50	1.0	2745	0.5780

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**0**

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	Manning Homes Banyan St Residential	Tested By :	G. Bathala	Date:	12/07/20
Project No. :	12968.001	Checked By:	A. Santos	Date:	12/15/20

Boring No.	TP-1			
Sample No.	B-1			
Sample Depth (ft)	0-3			
Soil Identification:	Olive brown (SM)g			
Wet Weight of Soil + Container (g)	100.79			
Dry Weight of Soil + Container (g)	99.20			
Weight of Container (g)	53.73			
Moisture Content (%)	3.50			
Weight of Soaked Soil (g)	100.30			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	303			
Crucible No.	1			
Furnace Temperature (°C)	860			
Time In / Time Out	11:20/12:05			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	21.5480			
Wt. of Crucible (g)	21.5470			
Wt. of Residue (g) (A)	0.0010			
PPM of Sulfate (A) x 41150	41.15			
PPM of Sulfate, Dry Weight Basis	**43**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.5			
PPM of Chloride (C -0.2) * 100 * 30 / B	60			
PPM of Chloride, Dry Wt. Basis	**62**			

pH TEST, DOT California Test 643

pH Value	**7.61**			
Temperature °C	20.7			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Manning Homes Banyan St Residential	Tested By :	G. Bathala Date: 12/10/20
Project No. :	12968.001	Checked By:	A. Santos Date: 12/15/20
Boring No.:	TP-1	Depth (ft.) :	0-3
Sample No. :	B-1		

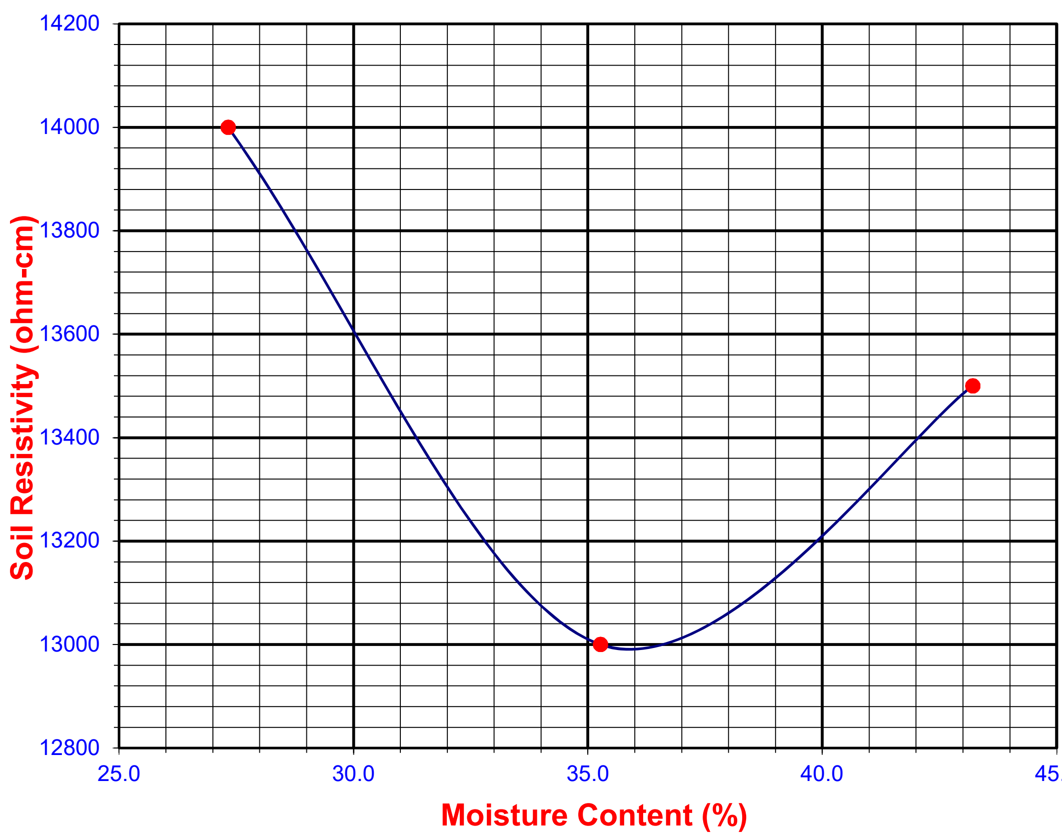
Soil Identification:* Olive brown (SM)g

*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	30	27.33	14000	14000
2	40	35.27	13000	13000
3	50	43.21	13500	13500
4				
5				

Moisture Content (%) (MCi)	3.50
Wet Wt. of Soil + Cont. (g)	100.79
Dry Wt. of Soil + Cont. (g)	99.20
Wt. of Container (g)	53.73
Container No.	
Initial Soil Wt. (g) (Wt)	130.30
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

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	
12990	**34.8**	**43**	**62**	**7.61**	**20.7**



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	Manning Homes Banyan St. Residential	Tested By:	J. Gonzalez	Date:	12/09/20
Project No.:	12968.001	Input By:	A. Santos	Date:	12/11/20
Boring No.:	TP-3	Depth (ft.):	0-3		
Sample No.:	B-1				
Soil Identification:	Olive brown poorly-graded sand with silt and gravel (SP-SM)g				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles; Correction per ASTM D 4718 is considered valid for soils that include up to 30% oversize particles retained on sieve #3/4

Preparation Method:	X	Moist	Scalp Fraction (%)	
		Dry	#3/4	33.0
Compaction Method	X	Mechanical Ram	#3/8	
		Manual Ram	#4	

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) = 0.07490

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	6720	6955	7182	7133		
Weight of Mold (g)	2734	2734	2734	2734		
Net Weight of Soil (g)	3986	4221	4448	4399		
Wet Weight of Soil + Cont. (g)	629.8	652.9	645.1	679.6		
Dry Weight of Soil + Cont. (g)	590.5	597.8	577.8	596.3		
Weight of Container (g)	87.6	76.9	77.0	88.0		
Moisture Content (%)	7.81	10.58	13.44	16.39		
Wet Density (pcf)	117.3	124.2	130.9	129.5		
Dry Density (pcf)	108.8	112.4	115.4	111.2		

Maximum Dry Density (pcf)	**115.5**	**Optimum Moisture Content (%)**	**13.5**
Corrected Dry Density (pcf)	**128.9**	**Corrected Moisture Content (%)**	**9.4**

☐ **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

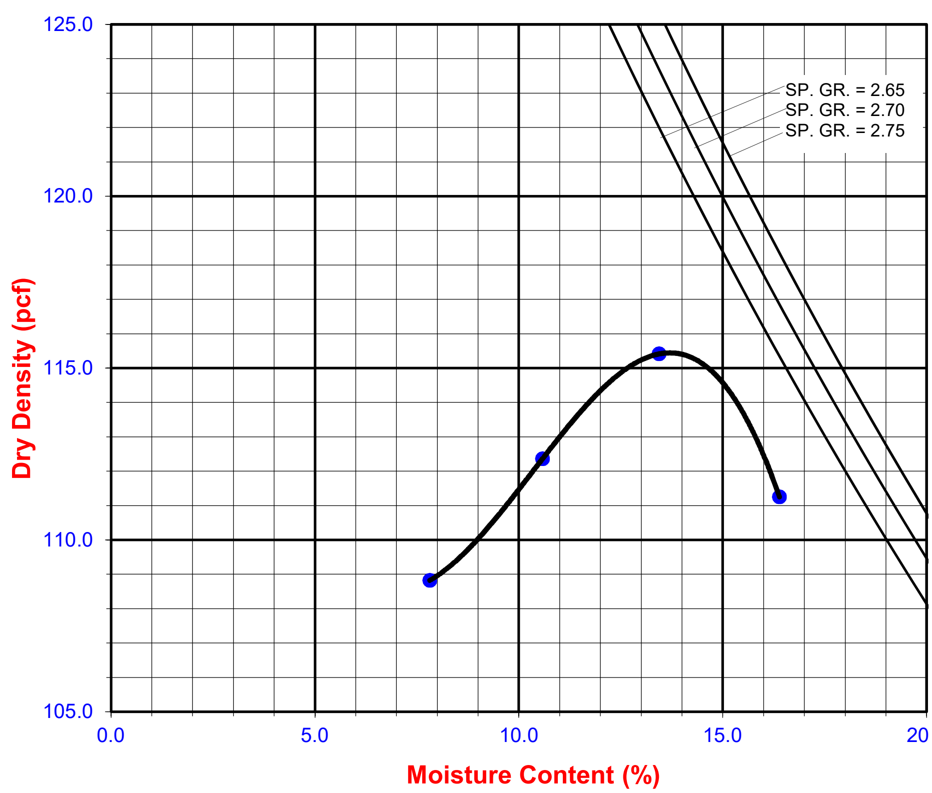
X **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



APPENDIX D

SUMMARY OF SEISMIC HAZARD ANALYSIS

Manning Homes

Latitude, Longitude: 34.1424, -117.5262

Date	12/18/2020, 12:25:16 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
SsRT	1.978	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.14	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.809	Factored deterministic acceleration value. (0.2 second)
S1RT	0.762	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.846	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.613	Factored deterministic acceleration value. (1.0 second)
PGAd	0.741	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.925	Mapped value of the risk coefficient at short periods
C_{R1}	0.901	Mapped value of the risk coefficient at a period of 1 s

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U.S. Geological Survey - Earthquake Hazards Program

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

34.1424

Longitude

Decimal degrees, negative values for western longitudes

-117.5262

Site Class

259 m/s (Site class D)

Spectral Period

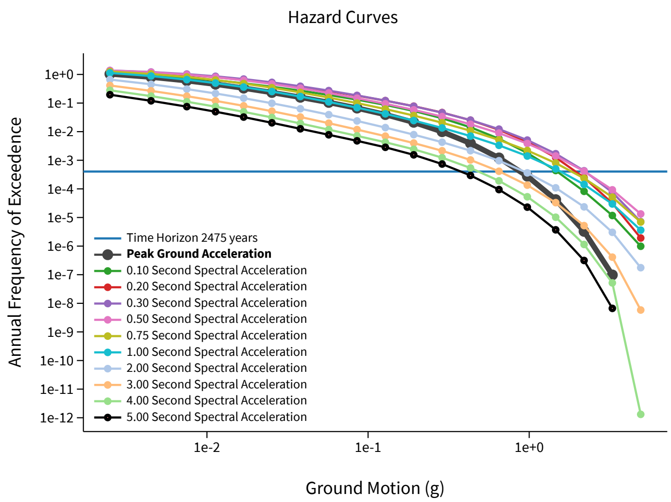
Peak Ground Acceleration

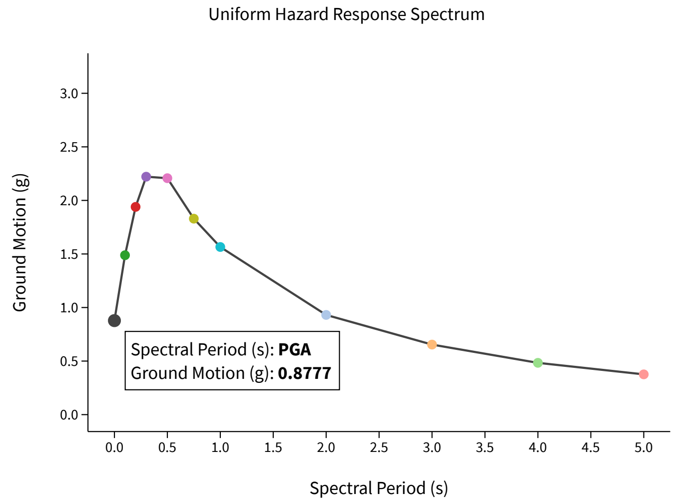
Time Horizon

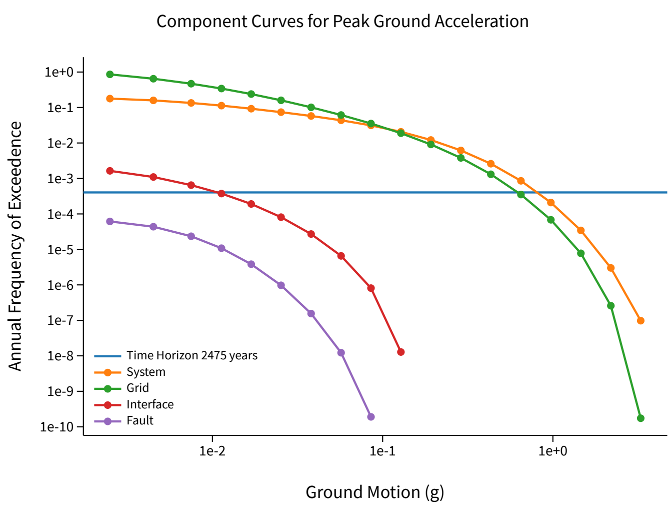
Return period in years

2475

Hazard Curve





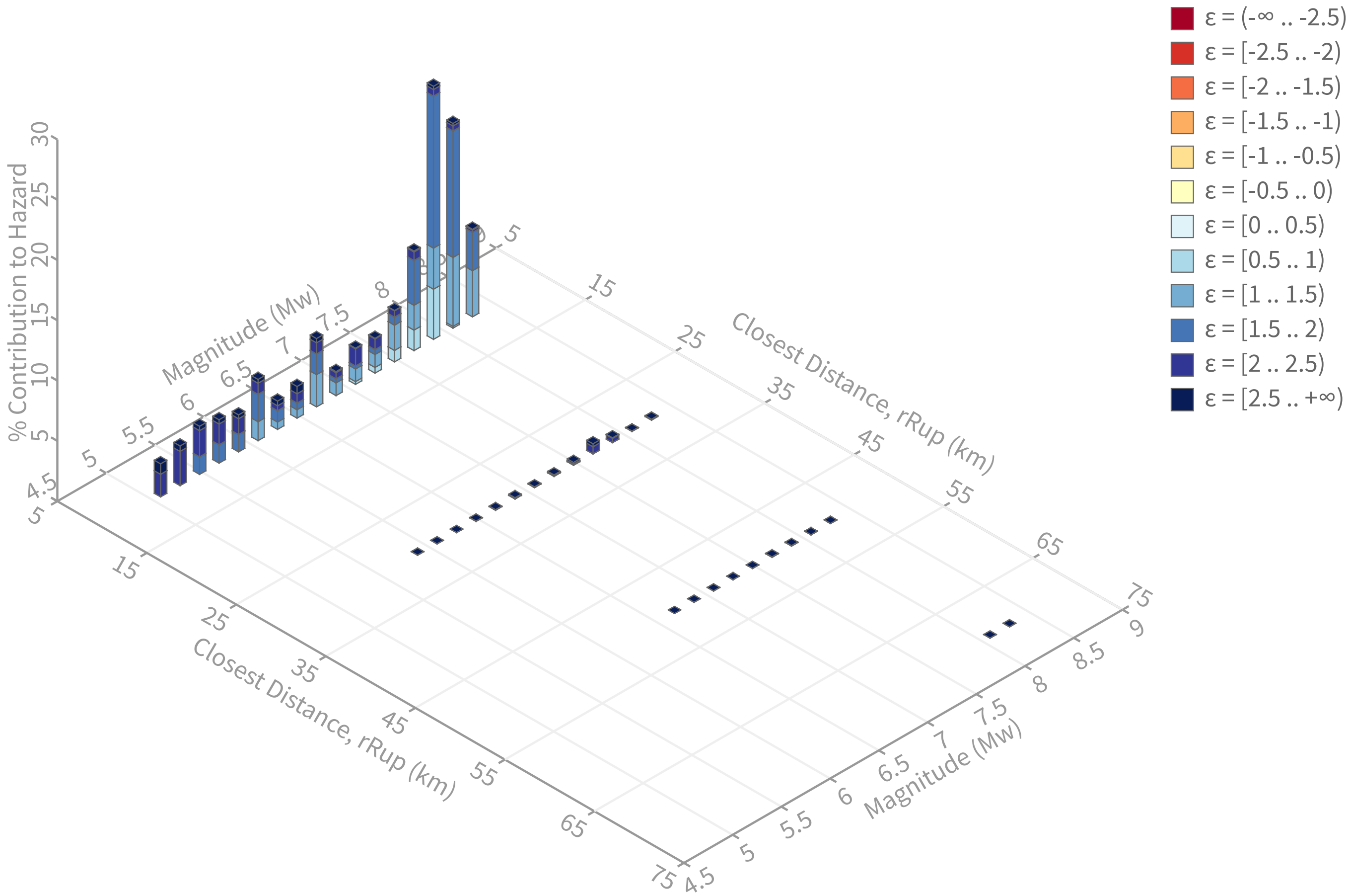


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.87774464 g

Recovered targets

Return period: 3193.942 yrs
Exceedance rate: 0.00031309272 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mean (over all sources)

m: 7.23
r: 10.89 km
ε_0: 1.71 σ

Mode (largest m-r bin)

m: 7.9
r: 11.34 km
ε_0: 1.55 σ
Contribution: 20.98 %

Mode (largest m-r-ε_0 bin)

m: 7.91
r: 14.3 km
ε_0: 1.76 σ
Contribution: 12.61 %

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							37.86
San Andreas (San Bernardino N) [2]		15.61	7.81	1.87	117.430°W	34.258°N	34.64	11.85
San Jacinto (San Bernardino) [0]		12.07	8.07	1.57	117.445°W	34.227°N	38.44	8.33
Cucamonga [0]		4.37	7.55	1.13	117.520°W	34.180°N	7.80	8.10
San Jacinto (Lytle Creek connector) [0]		8.89	8.04	1.39	117.452°W	34.191°N	51.79	2.66
Fontana (Seismicity) [1]		7.67	6.59	1.61	117.470°W	34.094°N	136.20	2.41
UC33brAvg_FM32	System							37.27
San Andreas (San Bernardino N) [2]		15.61	7.82	1.86	117.430°W	34.258°N	34.64	12.04
San Jacinto (San Bernardino) [0]		12.07	8.07	1.57	117.445°W	34.227°N	38.44	8.18
Cucamonga [0]		4.37	7.58	1.12	117.520°W	34.180°N	7.80	8.06
San Jacinto (Lytle Creek connector) [0]		8.89	8.03	1.39	117.452°W	34.191°N	51.79	2.59
Fontana (Seismicity) [1]		7.67	6.59	1.61	117.470°W	34.094°N	136.20	1.99
UC33brAvg_FM31 (opt)	Grid							12.45
PointSourceFinite: -117.526, 34.201		8.14	5.65	2.01	117.526°W	34.201°N	0.00	2.23
PointSourceFinite: -117.526, 34.201		8.14	5.65	2.01	117.526°W	34.201°N	0.00	2.23
PointSourceFinite: -117.526, 34.192		7.46	5.65	1.91	117.526°W	34.192°N	0.00	2.00
PointSourceFinite: -117.526, 34.192		7.46	5.65	1.91	117.526°W	34.192°N	0.00	2.00
UC33brAvg_FM32 (opt)	Grid							12.42
PointSourceFinite: -117.526, 34.201		8.14	5.65	2.01	117.526°W	34.201°N	0.00	2.23
PointSourceFinite: -117.526, 34.201		8.14	5.65	2.01	117.526°W	34.201°N	0.00	2.23
PointSourceFinite: -117.526, 34.192		7.46	5.65	1.91	117.526°W	34.192°N	0.00	2.00
PointSourceFinite: -117.526, 34.192		7.46	5.65	1.91	117.526°W	34.192°N	0.00	2.00

```
**********************
*                    *
*    E Q F A U L T   *
*                    *
*    Version 3.00    *
*                    *
**********************
```

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 12968.001

DATE: 12-18-2020

JOB NAME: Banyan St

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE: 34.1424
   SITE LONGITUDE: 117.5262

SEARCH RADIUS: 60 mi

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
   UNCERTAINTY (M=Median, S=Sigma): M   Number of Sigmas: 0.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
CUCAMONGA	4.4(7.1)	7.0	0.554	X
SAN JACINTO-SAN BERNARDINO	5.9(9.5)	6.7	0.355	IX
SAN JOSE	10.0(16.1)	6.5	0.259	IX
SAN ANDREAS - Southern	10.1(16.2)	7.4	0.350	IX
SAN ANDREAS - San Bernardino	10.1(16.2)	7.3	0.335	IX
SIERRA MADRE	11.7(18.8)	7.0	0.289	IX
SAN ANDREAS - Mojave	11.7(18.9)	7.1	0.269	IX
SAN ANDREAS - 1857 Rupture	11.7(18.9)	7.8	0.378	IX
CLEGHORN	12.3(19.8)	6.5	0.173	VIII
CHINO-CENTRAL AVE. (Elsinore)	15.6(25.1)	6.7	0.176	VIII
NORTH FRONTAL FAULT ZONE (West)	18.6(30.0)	7.0	0.173	VIII
SAN JACINTO-SAN JACINTO VALLEY	18.8(30.2)	6.9	0.148	VIII
WHITTIER	20.9(33.7)	6.8	0.120	VII
ELSINORE-GLEN IVY	21.0(33.8)	6.8	0.120	VII
CLAMSHELL-SAWPIT	21.0(33.8)	6.5	0.105	VII
ELYSIAN PARK THRUST	23.8(38.3)	6.7	0.103	VII

RAYMOND	27.8(44.7)	6.5	0.072	VI
VERDUGO	32.9(53.0)	6.7	0.066	VI
COMPTON THRUST	35.5(57.2)	6.8	0.064	VI
ELSINORE-TEMECULA	36.0(58.0)	6.8	0.062	VI
HELENDALE - S. LOCKHARDT	38.1(61.3)	7.1	0.075	VII
HOLLYWOOD	40.4(65.0)	6.4	0.039	V
NORTH FRONTAL FAULT ZONE (East)	42.1(67.7)	6.7	0.047	VI
NEWPORT-INGLEWOOD (L.A.Basin)	42.1(67.8)	6.9	0.055	VI
NEWPORT-INGLEWOOD (Offshore)	44.1(71.0)	6.9	0.052	VI
SAN JACINTO-ANZA	44.6(71.8)	7.2	0.067	VI
SAN GABRIEL	44.8(72.1)	7.0	0.056	VI
SIERRA MADRE (San Fernando)	45.4(73.0)	6.7	0.042	VI
PINTO MOUNTAIN	46.4(74.6)	7.0	0.053	VI
PALOS VERDES	50.5(81.3)	7.1	0.052	VI
SANTA MONICA	50.9(81.9)	6.6	0.033	V
NORTHRIDGE (E. Oak Ridge)	51.4(82.7)	6.9	0.041	V
LENWOOD-LOCKHART-OLD WOMAN SPRGS	51.9(83.5)	7.3	0.060	VI
JOHNSON VALLEY (Northern)	55.2(88.9)	6.7	0.033	V
SANTA SUSANA	57.2(92.0)	6.6	0.028	V
MALIBU COAST	58.0(93.3)	6.7	0.030	V
LANDERS	59.2(95.3)	7.3	0.051	VI

****

-END OF SEARCH- 37 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE CUCAMONGA FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 4.4 MILES (7.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.5541 g

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

LEIGHTON AND ASSOCIATES, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

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.

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.

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 <u>Processing</u>: 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 <u>Overexcavation</u>: 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 <u>Benching</u>: 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 <u>Evaluation/Acceptance of Fill Areas</u>: 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.

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).

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.