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August 16, 2024

PN 041.22901

Cory Babinski and/or Ali Tibbs
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Subject: Memorandum / Addendum Report #1
River Oaks Ranch Residential Development Project
West Stetson Avenue and Elk Street
APNS 464-300-005 AND 446-300-011
City of Hemet, Riverside County, California

Reference: Geotechnical Exploration Report, Proposed River Oaks Ranch Development, West Stetson Avenue and Elk Street APNs: 464-270-005 and -006 Hemet, California, Leighton & Associates, Inc. (Verdantas Company), dated April 10, 2024.

As requested by City reviewer, this addendum report is to document our clarifications regarding the settlement evaluation for the subject project. More specifically, this report is to further clarify our response to Comments 4A and 4B, as follows:

Comment 4A – Estimation of differential Settlement

From a geologic perspective, it is our opinion that the alluvium is relatively homogenous, especially when considering a horizontal distance of only 30 feet. As indicated in our report, distance between LB-1 and LB-3 is approximately 800 feet with an estimated differential settlement of ~2.2 inches. We additionally performed settlement analysis for LB-2, which is located much closer to LB-1 (~250 feet) and differential settlement was estimated to be 0.68-inch (see attached), which is much less than the 1-inch differential settlement over a distance of 30 feet recommended in our above referenced report.

Comment 4B – Differential Settlement Consideration for Structural Design

The decision to consider both static and dynamic settlements or the largest of the two settlements is typically made bythe A/E since both settlements are not expected to occur at the same time. This decision is based primarily on risk analysis and structural type of building. From a geotechnical perspective, we recommend that the largest of the two settlements be considered for design since static settlement is likely to occur immediately during or shortly after construction. As such, differential dynamic settlement of 1-inch over a distance of 30 feet is considered more critical for post construction and should be used in the structural design.

We appreciate the opportunity to work with you on this project. If you have any questions or if we can be of further service, please contact us at (your convenience

Sincerely,

Verdantas

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Distribution: (1) Addressee (PDF copy via email)

GEOTECHNICAL EXPLORATION REPORT PROPOSED RIVER OAKS RANCH DEVELOPMENT MULTI-FAMILY RESIDENTIAL DEVELOPMENT WEST STESTON AVENUE AND ELK STREET HEMET, CALIFORNIA

Prepared For **HIGHPOINTE COMMUNITIES, INC.**
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Project No. 22901

April 10, 2024

April 10, 2024

Project No. 22901

Highpointe Hemet 1 LLC
530 Technology, Suite 100
Irvine, California 92618

Attention: Mr. James Coleman, Senior Project Manager

Subject: Geotechnical Exploration Report
Proposed River Oaks Ranch Development
West Stetson Avenue and Elk Street
APNs: 464-270-005 and 464-270-006
Hemet, California

Leighton and Associates, Inc. (Leighton) is pleased to present this report documenting the results of our geotechnical exploration of the subject site located in the City of Hemet, California. The purpose of our exploration was to evaluate the general on-site surface/ subsurface soils conditions and develop geotechnical recommendations for design and construction of the proposed development. Based on the results of our exploration and analysis, the site is considered suitable from a geotechnical perspective provided the recommendations herein are properly incorporated into project design and construction. Our recommendations should be further reviewed based on final grading and foundation plans once available.

We appreciate the opportunity to work with you on this project. If you have any questions or if we can be of further service, please contact us at (866) LEIGHTON; or specifically at the telephone extensions or e-mail addresses listed below.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The regional site location is depicted on attached Figure 1, *Site Location Map* (Latitude: 33.72857°; Longitude: -116.98421°). The property consists of 2 parcels totaling approximately 10 acres, designated by Riverside County as APN 464-270-005 and -006, City of Hemet, California. The site fronts West Stetson Avenue on the north and existing developed residential property on the west and south. A north-south trending access road (South Elk Street) bounds the property on the west, connecting with W. Stetson Avenue. Overall topographic relief is relatively flat, with surface elevations (El's) ranging from a low of about 1,555 feet (ft) above mean sea level (msl) near the southwest corner to a high of approximate El 1,567 ft msl on the east. The site is presently undeveloped and exhibits no evidence of any past grading. The site is presently covered by tilled soil with low grassland vegetation.

A Conceptual Site Plan prepared by Danielian Associates, dated March 6, 2024, indicates the development will be on the east side of S. Elk Street, and consists of several multi-family buildings on/or near existing grades accommodating a total of 228 living units. Appurtenant improvements will include paved interior access roads and surface parking, a clubhouse with swimming pool, playgrounds, dog-parks, a surface retention basin for stormwater management, underground utility infrastructure and landscaping. We anticipate buildings to be 2- to 3-story wood-frame structures supported on conventional reinforced concrete spread footings and/or slab-on-grade floors founded near existing/finish grades. Structural loads as high as 90 kips (LRFD) for isolated spread footings and 7.5 kips per lineal feet (klf) for continuous wall footings are anticipated. Vehicular access to the development will be via S. Elk Street and the buildings via a network of paved private access roads.

1.2 Purpose and Scope

The purpose of this exploration was to evaluate the onsite surface and subsurface soils conditions, and provide geotechnical recommendations for design and construction of the proposed development. More specifically, our scope of our work included the following:

- *Literature Review* – Review of published geologic maps and reports, and historical aerial photographs and topographic maps readily available on-line or within our in-house technical library. We also reviewed a geotechnical exploration report prepared for the property directly south of the subject site (GeoTek, 2020). A list of these documents is presented in Section 7.0, *References*.

- *Exploratory Borings* – Advanced three (3) hollow-stem borings (LB-1, LB-3, and LB-7) on the site, on April 11, 2022, extending to depths between 16.5 feet to 51.5 feet below the existing ground surface (bgs). Since the exploratory borings were performed prior to updating the site plans, another hollow-stem boring was performed (LB-2) on a parcel on the west side of S. Elk Street, outside the site boundary. The approximate locations of the borings are shown on Figure 2, *Exploration Location Map*. Logs of the borings are presented in Appendix A, *Exploratory Boring Logs*.

  Bulk and relatively undisturbed drive samples were collected from the borings for geotechnical laboratory testing. The driven samples were obtained using a Modified California Ring sampler conducted in accordance with ASTM Test Method D3550. The samplers were driven for a total penetration of 18 inches using a 140-pound automatic hammer allowed to fall freely from a height of 30 inches. The number of blows per 6 inches of penetration was recorded. Each boring was logged in the field by a member of our technical staff under the direct supervision of a State of California Certified Engineering Geologist (CEG).

  Soil samples were reviewed and described in the field in accordance with the Unified Soil Classification System (USCS). Upon completion, the borings were backfilled with soil cuttings. The results of our laboratory testing are attached in Appendix B, *Laboratory Test Results*.

- *Infiltration Testing* – Two hollow-stem auger borings (LP-1 and LP-2) were advanced to depths of 4 feet bgs for the purpose of field percolation testing, in the location of the proposed retention basin for LP-2 and in the location of a retention basin proposed in the previous plans but not the current plan for LP-1. The borings were converted into temporary test wells by installing 2-inch-diameter slotted (0.020-inch slots) PVC well casing, and placing No. 3 Monterey Sand in the annular space of the well within the test zone. Pre-soaking and percolation testing of the LP-1 and LP-2 test wells was performed on April 14 and 15, 2022, respectively. It should be noted that LP-1 was performed on the west side of S. Elk Street, meaning it is outside the current site boundary. Testing was performed in general accordance with the *Riverside County - Low Impact Development BMP Design Handbook* (Riverside County Flood Control and Water Conservation District, 2011). Test results are presented in Appendix C, *Field Percolation Test Results*. Refer to the discussion of infiltration rate presented in Section 2.6, *Infiltration*.

- *Engineering Evaluation* – Data collected and analyses were performed by a Geotechnical Engineer (GE) and a Certified Engineering Geologist (CEG).

- *Report Preparation* – This report was prepared to document findings and conclusions, and provide design-level geotechnical recommendations addressing the currently proposed development concept.

2.0 GEOTECHNICAL AND GEOLOGIC FINDINGS

2.1 Regional Geology

The site is situated within the southern portion San Jacinto Valley southwest of the San Jacinto River and southeast of the Lakeview Mountains. The San Jacinto Valley is a relatively flat-lying depositional basin surrounded by hills and small mountains. In general, the eastern portion of the valley consists of a down dropped elongated and fault-bounded graben (trough) infilled by alluvium. The graben is formed on the east by the main trace of the San Jacinto Fault and the west by the Casa-Loma fault segment. Collectively these faults form a portion the San Jacinto Fault Zone, a major tectonic structure within the regional landscape. The western valley consists of a broad, gently eastward sloping alluvial mesa (bajada). Published maps show a generalized distribution of geologic units on the site within the surrounding areas, see Figure 3, *Regional Geology Map*.

Deposition of sediments underlying the valley are derived from San Jacinto River and Bautista Creek. Minimum sediment thickness is reportedly on the order of approximately 500 feet within the southwest portion of the valley. Silts and sands of Quaternary-aged terrace deposits, and fanglomerates flank major abandoned drainage channels and the base of the surrounding mountain slopes. The hills and mountains surrounding the valley are composed of metamorphic country rock of Mesozoic age that intrude by Cretaceous-age granitic basement rock.

2.2 Site-Specific Geology

Our borings reveal the site is underlain entirely by Quaternary age alluvial fan deposits, typically composed of silty sands, sandy silts and interlayered poorly-graded sands with a lessor constituent of silts and silty clays. Morton et al. (2006) subdivide these alluvial fan deposits into two distinct units as follows:

- *Young Alluvial Fan Deposits of Bautista Canyon (Map Symbol Qyfb) – (Holocene to late Pleistocene)*: Deposits underlie tilled topsoil throughout the site to depths from 20 to 31 feet bgs. This unit generally consists of unconsolidated to moderately consolidated grayish brown to olive silt sand to sandy silt with interbedded clayey silt layers. These alluvial deposits are expected to possess low expansion potential (EI<51). Based on our laboratory testing for this study and previous testing for adjacent site (GeoTek, 2020), these materials are expected to exhibit slight hydro-collapse potential (up to 2.5 percent) in the upper 8 to 10 feet bgs.

- *Old Alluvial Fan Deposits (Map Symbol Qof) – (late to middle Pleistocene)*: The contact of this unit was identified at depths as shallow as 25 feet bgs. This unit is typically distinguished by a dense reddish brown silty sand to clayey sands with subsequent silty sand units ranging in color from brown to yellowish brown that is medium dense to very dense.

2.3 Groundwater Conditions

No groundwater was encountered within the maximum depth of our exploration (51.5 feet bgs). Approximately 400 feet east of the northwest site corner is an existing municipal groundwater well (Well No. 05S01W21A001S). The well location is noted on attached Figure 1, *Site Location Map*. According to the California Department of Water Resources website, a depth to water recorded in March of 1992 was measured at approximately 223 feet bgs. Exploratory borings and CPT soundings conducted in the site to the south (GeoTek, 2020) encountered no groundwater at depths up to 50 feet. As such, groundwater is not anticipated to pose a constraint to site grading, planned improvements, nor the susceptibility of the site to seismic-induced liquefaction.

Fluctuations in groundwater levels and/or soil moisture beneath the site, or development of temporary perched water conditions, can occur seasonally as a result of storm events, storm water runoff, stormwater infiltration, or landscape irrigation.

2.4 Expansive Soils

As indicated above, the site near surface soils consist of silty sand sandy silts of low plasticity and expected to possess low expansion potential (EI<51).

2.5 Soil Sulfate Content

Based on our previous experience in the site area, we anticipate a negligible concentration of soluble sulfates in onsite soils. Additional corrosion testing should be performed on representative finish grade soils at the completion of rough grading.

A representative bulk sample of near surface soil collected between 0 and 5 feet bgs from boring LB-1 was tested to evaluate corrosion potential. Results of chemical analysis tests are attached in Appendix B, *Laboratory Test Results.* A summary of the test results is presented in Table 1 below.

Table 1. Corrosivity Test Results

Test Parameter	Test Results (LB-1@0=5')	General Classification of Hazard
Water-Soluble Sulfate-SO_4 in Soil (ppm)	107	Negligible sulfate exposure to buried concrete-S0 Exposure Class
Water-Soluble Chloride in Soil (ppm)	60	Non-corrosive to buried concrete (per Caltrans Specifications)
pH	8.42	Mildly alkaline
Minimum Resistivity (saturated ohm-cm)	2946	Moderately corrosive to buried ferrous pipes

2.6 Infiltration

As per discussions with the project civil engineer and in accordance with our authorized scope of work, Leighton performed two field percolation tests (LP-1 and LP-2) at a depth of 4 feet bgs with LP-2 in the vicinity of planned retention basin along the southern site boundary, see Figure 2, *Exploration Location Map*. Soils encountered within test zones at this location consist of silty sands to sandy silt. A well was constructed using 2-inch diameter slotted PVC pipe (0.020 in) with annular space around well pipes infilled with #3 Monterey Sand to a height of 1-foot bgs.

Following pre-soaking and based on the results of preliminary field tests, it was determined that a falling head test procedure was warranted, requiring periodic measurements of water level drop inside the well at intervals during the test period. Calculated from the test results are “measured” rates of percolation, by dividing the rate of discharge (cubic inches per hour) by the infiltration surface area (flow area in square inches). Discharge volumes were calculated by adding the total volume of water drop inside the PVC pipe and within the porosity-factored annulus material. The flow area was based on the average water height within the slotted pipe section of the test well only. At the conclusion of testing well casing was removed and the test holes backfilled with excess soil cuttings.

The measured rates of infiltration yielded by field percolation tests are presented below in Table 2 below, in units of inches per hour (in/hr). Test data are also presented in Appendix C, *Field Percolation Test Results*. The measured rates are defined as “unfactored” in that no safety factor has been applied.

Table 2. Field Percolation Testing Summary

Percolation Test Boring/Well Designation	Percolation Test Method	Approximate Depth of Test Zone Below Ground Surface (feet)	Unfactored* Infiltration Rate (in/hr)
LP-1	Falling Head	0.5 - 4.0	0.61
LP-2	Falling Head	1.0 - 4.0	0.67

Note: Invert of any stormwater infiltration shall be set back at least 15 feet, and outside a 1:1 plane drawn down and out from the bottom of adjacent foundations.

The "measured" infiltration rate yielded by our field percolation test suggests alluvial deposits underlying the site at shallow depths will support use of infiltration BMP's as part of an on-site stormwater system. The rate is the product of a small-scale test performed at a specific location and depth on the site. The actual rate within the same sediments elsewhere on the site, or even within the limits of a proposed BMP, can be more or less than that indicated by our testing. For system design a factor of safety (FS) must be applied to the resultant measured infiltration rates. Use of "factored" rates for design is intended to account for the vertical/lateral variability of soil conditions and promote long-term system performance. Infiltration rate is expected to decline over the lifespan of the system, and between BMP maintenance cycles, as fine particulates accumulate within the infiltration media.

3.0 GEOLOGIC AND SEISMIC HAZARD EVALUATION

3.1 Faulting

The site is not transected directly by any faults classified as active or potentially active by the state of California, nor is the property located within a state-mapped Alquist-Priolo Earthquake Fault Zone. Our review of published geologic maps also did not identify any other faults on, crossing or projecting toward the site. The locations of the nearest known active and potentially active faults with a potential for surface fault rupture are depicted on Figure 4, *Regional Fault Map*.

3.2 Seismicity

Historically, the San Jacinto Fault Zone (SJFZ) has produced earthquakes in the magnitude range of 6.0Mw to 7.6Mw (Moment magnitude). In roughly the last 100 years (1903 through 2020), 9 major quakes in the range of 6.0Mw to 7.6Mw have occurred within a 50-mile radius of the subject site. Each of these large quakes has produced moderate to severe damage to buildings and roads, and several have resulted in fatalities (USGS, 1971). The frequency and relatively short recurrence interval of surface rupture for the SJFZ has resulted in many events during Holocene time with at least 16 documented in the past 3,700 years (Onderdonk et al., 2018).

Common throughout most of Southern California is a potential for strong ground shaking generated by moderate to severe earthquakes. The intensity of ground shaking at a given location depends primarily upon earthquake magnitude, site distance from the source, and site response (soil type) characteristics. Seismic coefficients for the subject site were calculated utilizing an interactive program on current United States Geological Survey (USGS) website using ASCE 7-16 procedures. Based on the results of seismic profiling, the soil sediments underlying the site are classified as Site Class D. As such, the site-specific seismic coefficients are as listed in Table 3 below. Copies of seismic analysis data are attached in Appendix D, *Seismic Design Data and Analysis.*

Table 3. 2019 CBC-Based Seismic Design Parameters

Categorization/Coefficient	Design Value
Site Latitude: 33.72857, Site Longitude: -116.98421	
Site Class: D	
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_S	1.84g
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.72g
Short Period (0.2 sec) Site Coefficient, F_a	1.0
Long Period (1 sec) Site Coefficient, F_v	1.70
Adjusted Spectral Response Acceleration at Short Period (0.2 sec), S_{MS}	1.844g
Adjusted Spectral Response Acceleration at Long Period (1 sec), S_{M1}	1.24g*
Design Spectral Response Acceleration at Short Period (0.2 sec), S_{DS}	1.23g
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	0.82g*
PGA adjusted for Site Class, $PGA_M = F_{PGA}$ *PGA	0.86g
g = Gravity acceleration *Per Exception 2 in Section 11.4.8 of ASCE 7-16, seismic response coefficient CS to be determined by Eq. 12.8-2 for values of T < 1.5Ts and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for TL > T > 1.5Ts or Eq. 12.8-4 for T > TL	

3.3 Other Geologic Hazards

Other site geologic hazards associated with this site are discussed in subsections below.

3.3.1 Liquefaction Potential

According to the Liquefaction Map published on the ESRI ArcGIS website (see Figure 5, *Liquefaction Hazard Map*, the site is defined as having a moderate liquefaction susceptibility. This regional scale mapping represents only a general distribution of the liquefaction potential, and not a definitive indication that liquefaction can or will occur. It is intended to inform practitioners of its potential so that appropriate hazard analyses may be incorporated into a development project. Given an absence of groundwater encountered beneath the site at or above a depth of 50 feet bgs, the potential constraint to the proposed development due to liquefaction and related seismic-induced settlement is considered very low.

3.3.2 Seismically-Induced Settlement

During a strong seismic event, seismically-induced settlement can still occur within loose to medium dense, dry or moist granular soils. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Based on the design earthquake and a Peak Ground Acceleration (PGA) of 0.86g, the estimated dynamic settlement is 5.3 inches based on LB-1 and 3.1 inches based on LB-1 (see appendix D). Due to proposed remedial grading and relatively homogenous deep alluvium, the dynamic settlement is expected to be generally global or occurs over a large area. As such, the differential settlement is not expected to exceed 1-inch in a 30-foot horizontal distance.

3.3.3 Lateral Spreading

As the site has a very low liquefaction potential and it is relatively constrained laterally, the potential for earthquake-induced lateral spreading at the site is considered negligible.

3.3.4 Slope Stability and Seismically Induced Landslides

The site is relatively flat in topographic relief and not designated on County of Riverside hazard maps as occurring within a landslide hazard zone. No slopes or other elevated areas of any significance exist on or adjacent to the property which could be potential source of landslides. Based on the above, the potential for slope instability or seismically induced landslides is considered negligible.

3.4 Earthquake-Induced Flooding

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. The closest water retaining structures to this site are three dams retaining Diamond Valley Lake located approximately 1.8 miles to the southwest. The potential for earthquake induced flooding to affect this site is considered negligible due to the dam's recent construction and seismic design.

3.5 Flooding

The project site is not located within a flood hazard zone as identified by the County of Riverside.

3.6 Land Subsidence

Land subsidence refers to the sinking or gradual downward settlement and compaction of soil deposits, commonly associated with the extraction of deep groundwater and/or petroleum resources from a region. No evidence of historical subsidence is notable in documents reviewed by this firm. The potential impacts to the site from a subsidence hazard are considered low.

4.0 SUMMARY OF FINDINGS AND CONCLUSIONS

A summary of our geologic and geotechnical findings and conclusions are presented below:

- No groundwater was encountered to a depth of at least 51.5 feet below the existing site surface. It is expected to be on the order of several hundred feet, and pose no constraint to site development.
- Shallow field percolation test results in deposits of in-situ alluvium along the southern site margin, indicate use of infiltration BMP elements as part of an on-site stormwater system design is feasible.
- The site is not located within an Alquist-Priolo Earthquake Fault Zone, nor was any evidence of active faulting observed on or projecting toward the site, based on our review of aerial photographs and published hazard maps. Given the close proximity of major faults and historical earthquakes, the occurrence of strong ground shaking at the site is likely during its economic life-span. Surface fault rupture is not considered a site hazard.
- Site soils is expected to possess a low expansion (EI<51) when wetted. These soils are suitable for use as compacted fill and can be readily compacted using a combination of thorough watering and wheel rolling with rubber tire equipment.
- To mitigate the settlement potential of relatively loose surficial soils, we recommend the upper 8 feet of in-situ deposits or minimum of 7 feet below foundation (not to exceed 10 feet bgs) be over-excavated (removal) and recompacted as part of remedial grading. Deeper removals may be required locally depending upon final grading and/or foundation design.
- Existing onsite soils re considered suitable for use as new engineered fill provided they are relatively free of organic material and debris.
- Finish building pads, slope faces and other graded surfaces will be susceptible to erosion if left unprotected. This risk can be reduced through installation of certain control measures including but not limited to a jute net cover, erosion control blankets, straw wattles, or other similar methods of protection.
- Caving and raveling of soils in un-shored excavations should be expected,

5.0 RECOMMENDATIONS

The following geotechnical recommendations are provided for project grading and construction.

5.1 General Earthwork Considerations

All site grading should be performed in accordance with applicable local regulatory codes, and project specifications prepared by applicable design professional. Detailed grading recommendations are attached herein as Appendix E, *General Earthwork and Grading Specifications For Rough Grading*.

5.1.1 Site Preparation

Prior to construction, the site should be cleared of any vegetation, trash, and/or debris within the area of proposed grading. These materials should be removed from the site. Any underground obstructions onsite should be removed. Existing utility lines will need to be removed and/or rerouted where interfering with the proposed construction. Any resulting excavation cavities should be properly backfilled and compacted. All unsuitable earth deposits should be excavated and removed from the footprints of proposed buildings/structures prior to fill placement. Any existing undocumented fill will need to be removed from areas of planned structural improvements.

5.1.2 Remedial Grading

The upper 8 feet of existing surficial soil, or 7 feet below bottom of footings, whichever deeper, should be removed/over-excavated and recompacted prior to foundation construction or placement of any additional fill. The removal limit should be established by a 1:1 (horizontal: vertical) projection from the edge of fill soils supporting settlement-sensitive structures downward and outward to competent material identified by the geotechnical consultant. Removal will also include benching into competent material as the fills rise. Areas adjacent to existing structures/roadways or property limits may require special considerations and monitoring.

5.1.3 Fill Placement and Compaction

Fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned to within 2 percent of optimum moisture content for sandy soils and at least 4 percent above optimum moisture content for clayey soils (not anticipated), and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D 1557.

5.1.4 Shrinkage and Subsidence

The volume change of excavated onsite materials upon compaction is expected to vary with materials, volume of roots and deleterious materials, density, insitu moisture content, location, and compaction effort. The in-place and compacted densities of soil materials vary and accurate overall determination of shrinkage and bulking cannot be made. Therefore, we recommend site grading include, if possible, a balance area or ability to adjust import quantities to accommodate some variation. Based on our experience with similar materials, we anticipate 10 to 16 percent shrinkage in the upper 10 feet of alluvium. Subsidence due solely to scarification, moisture conditioning and recompaction of the exposed bottom of overexcavation, is expected to be on the order of 0.15 foot. This should be added to the above shrinkage value for the recompacted fill zone, to calculate overall subsidence.

5.1.5 Import Soils

Import soils and/or borrow sites, if needed, should be evaluated by the geotechnical consultant prior to import. Import soils should be uncontaminated, granular in nature, free of organic material (loss on ignition less-than 2 percent), have a very low expansion potential (with an Expansion Index less than 12) and have a low corrosion impact to the proposed improvements.

5.1.6 Utility Trenches

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the Standard Specifications for Public Works Construction, (“Greenbook”), 2021 Edition (or most recent). Fill material above the pipe zone should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D 1557) by mechanical means only. Site soils may generally be suitable as trench backfill provided these soils are screened of rocks over 1½ inches in diameter and organic matter. If imported sand is used as backfill, the upper 3 feet in building and pavement areas should be compacted to 95 percent. The upper 6 inches of backfill in all pavement areas should be compacted to at least 95 percent relative compaction.

Where granular backfill is used in utility trenches adjacent moisture sensitive subgrades and foundation soils, we recommend that a cut-off “plug” of impermeable material be placed in these trenches at the perimeter of buildings, and at pavement edges adjacent to irrigated landscaped areas. A “plug” can consist of a 5-foot long section of clayey soils with more than 35-percent passing the No. 200 sieve, or a Controlled Low Strength Material (CLSM) consisting of one sack of Portland-cement plus one sack of bentonite per cubic-yard of sand. CLSM should generally conform to Section 201-6 of the Standard Specifications for Public Works Construction, (“Greenbook”), 2018 Edition. This is intended to reduce the likelihood of water permeating trenches from landscaped areas, then seeping along permeable trench backfill into the building and pavement subgrades, resulting in wetting of moisture sensitive subgrade earth materials under buildings and pavements.

Excavation of utility trenches should be performed in accordance with the project plans, specifications and the California Construction Safety Orders (current Edition). The contractor should be responsible for providing a "competent person" as defined in Article 6 of the California Construction Safety Orders. Contractors should be advised that sandy soils (such as fills generated from the onsite alluvium) could make excavations particularly unsafe if all safety precautions are not properly implemented. In addition, excavations at or near the toe of slopes and/or parallel to slopes may be highly unstable due to the increased driving force and load on the trench wall. Spoil piles from the excavation(s) and construction equipment should be kept away from the sides of the trenches. Leighton does not consult in the area of safety engineering.

5.1.7 Drainage

All drainage should be directed away from foundations at a slope not less than 5-percent and for a minimum distance of 10 feet from the face of the wall per the 2019 CBC. Drainage should be directed away from buildings, slopes, and pavements by means of approved permanent/temporary drainage devices. Adequate storm drainage of any proposed pad should be provided to avoid wetting of foundation soils. Irrigation adjacent to buildings should be avoided when possible. As an option, sealed-bottom planter boxes and/or drought resistant vegetation should be used within 5-feet of buildings.

5.1.8 Slope Design and Construction

Based on our understanding and for planning purposes, all fill and cut slopes will be designed and constructed at 2:1 (horizontal:vertical and expected to be less than 10 feet in height. These slopes are considered grossly stable for static and pseudostatic conditions. For planning purposes, cut slopes should be constructed as replacement fill slopes due to the highly erosive nature of site soils. Future grading plans should be subject to further review and evaluation.

The outer portion of fill slopes should be either overbuilt by 2 feet (minimum) and trimmed back to the finished slope configuration or compacted in vertical increments of 5 feet (maximum) by a weighted sheepsfoot roller as the fill is placed. The slope face should then be track-walked by dozers of appropriate weight to achieve the final slope configuration and compaction to the slope face.

Slope faces are inherently subject to erosion, particularly if exposed to wind, rainfall and irrigation. Landscaping and slope maintenance should be conducted as soon as possible in order to increase long-term surficial stability. Berms should be provided at the top of fill slopes. Drainage should be directed such that surface runoff on the slope face is minimized.

5.2 Foundation Design

Based on our analysis, and upon implementation of remedial grading measured recommendations herein, the use of shallow isolated and/or continuous wall footings will be suitable to support the proposed residential structures.

5.2.1 Bearing and Lateral Pressures

The proposed foundations and slabs should be designed in accordance with the structural consultants' design, the minimum recommendations presented herein, and the 2019 CBC. In utilizing the minimum geotechnical foundation recommendations, the structural consultant should design the foundation system to acceptable deflection criteria as determined by the architect. Foundation footings may be designed with the following geotechnical design parameters:

- ***Bearing Capacity***: A net allowable bearing capacity of 2,750 pounds per square foot (psf), or a modulus of subgrade reaction of 200 pci may be used for design of footings founded entirely into compacted fill. The footings should extend a minimum of 12 inches below lowest adjacent grade. A minimum base width of 18 inches for continuous footings and a minimum bearing area of 3 square feet (1.75 ft by 1.75 ft) for pad foundations should be used. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind).
- ***Passive Pressures***: The passive earth pressure may be computed as an equivalent fluid having a density of 300 psf per foot of depth, to a maximum earth pressure of 3,000 pounds per square foot. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

The footing width, depth, reinforcement, slab reinforcement, and the slab-on-grade thickness should be designed by the structural consultant based on recommendations and soil characteristics indicated herein and the most recently adopted edition of the CBC.

5.2.2 Settlement

The project civil engineer, structural engineer, and architect should consider the potential effects of both static settlement and dynamic settlement presented below.

- ***Static Settlement***: Most of the static settlement of onsite soils is expected to be immediate or within 30 days following fill placement/foundations. A differential static settlement of 0.5 inch over a 30-foot span may be considered for design purposes.

- **Dynamic Settlement**: Based on our analysis, we estimate that total dynamic settlement is expected to be approximately 5.3 inches at the location of LB-1 and approximately 3.1 inches at the location of LB-3. Due to relatively uniform alluvium conditions and a large distance between the boreholes, this settlement is expected to be global and total differential settlement is expected to be minimal or less than 1-inch over a 30-foot horizontal span.

5.2.3 Vapor Retarder

It has been a standard of care to install a moisture retarder underneath all slabs where moisture condensation is undesirable. Moisture vapor retarders may retard but not totally eliminate moisture vapor movement from the underlying soils up through the slabs. Moisture vapor transmission may be additionally reduced by use of concrete additives. Leighton does not practice in the field of moisture vapor transmission evaluation/mitigation. Therefore, we recommend that a qualified person/firm be engaged/consulted to evaluate the general and specific moisture vapor transmission pathways and any impacts to proposed construction elements. This person/firm should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structure as deemed appropriate.

However, based on our experience, the standard of practice in Southern California has evolved over the last 15 to 20 years into a construction of a vapor retarder system that generally consisted of a membrane (such as 10-mil thick or greater), underlain by a capillary break consisting of 4 inches of clean ½-inch-minimum gravel or 2-inch sand layer (SE>30). The structural engineer/architect or concrete contractor often require a sand layer be placed over the membrane (typically 2-inch thick layer) to help in curing and reduction of curling of concrete. If such sand layer is placed on top of the membrane, the contractor should not allow the sand to become wet prior to concrete placement (e.g., sand should not be placed if rain is expected).

In conclusion, the construction of the vapor barrier/retarder system is dependent on several variables which cannot all be evaluated and/or tested from a geotechnical standpoint. As such, the design of this system should be a design team/owner decision taking into consideration finish flooring materials and manufacture's installation requirements of proposed membrane. Moreover, we recommend that the design team also follow ACI Committee 302 publication for "Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials" (ACI 302.2R-06) which includes a flow chart that assists in determining if a vapor barrier /retarder is required and where it is to be placed.

5.3 Temporary Excavation and Shoring Design

All temporary excavations for utility trenches, retaining walls, and foundations should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter. Site soils should be considered as Type C Soil per OSHA guidelines.

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 cut, 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 structure.

5.4 Preliminary Pavement Design Parameters

Our laboratory testing of a bulk soil sample collected from Boring LB-1, at a depth of 0 to 5 fee bgs, yielded an R-value of 61. Pavement section recommendations based on this test are presented in Table 4 below. The recommendations are intended for planning purposes only and should not supersede minimum City or County requirements. For final pavement design, appropriate traffic indices should be selected by the project civil engineer or traffic engineering consultant. Additional testing should be performed to verity design once samples of actual soil subgrade are in place.

Table 4. Pavement Section Design

Street Type	Loading Conditions TI	*AC Pavement Section Thickness*	
		Asphaltic-Concrete (AC) Thickness (inch)	Aggregate Base (AB) Thickness (inch)
Alleys/Local Streets	5	3.0	4.0
Collector Street/ Trucks	6	3.0	4.0
Perimeter Roadways	7	3.5	4.0

The upper 6 inches of subgrade soil should be properly compacted to at least 95 percent relative compaction (ASTM D1557) and should be moisture-conditioned to near optimum and kept in this condition until the pavement section is constructed. Proof-rolling subgrade to identify localized areas of yielding subgrade (if any) should be performed prior to placement of aggregate base and under the observation of the geotechnical consultant.

Minimum relative compaction requirements for aggregate base should be 95 percent of the maximum laboratory density as determined by ASTM D1557. Base rock should conform to the "Standard Specifications for Public Works Construction" (green book) current edition or Caltrans Class 2 aggregate base having a minimum R-value of 78. Asphaltic concrete should be placed on compacted aggregate base and compacted to minimum 95% relative compaction.

The pavement sections provided in this section are intended as minimum values. Should thinner or highly variable as-built pavement sections result from construction, increased maintenance and repair may be needed.

5.5 Retaining Walls

Retaining wall earth pressures are a function of the amount of wall yielding horizontally under load. If the wall can yield enough to mobilize full shear strength of backfill soils, then the wall can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance. Retaining walls backfilled with non-expansive soils should be designed using the following equivalent fluid pressures:

Table 5. Retaining Wall Design Earth Pressures (Static, Drained)

Loading Conditions	Equivalent Fluid Density (pcf)	
	Level Backfill	2:1 Backfill
Active	33	50
At-Rest	50	80
Passive*	300	150 (2:1, sloping down)

*This assumes level condition in front of the wall will remain for the duration of the project, not to exceed 3,000 psf at depth. If sloping down (2:1) grades exist in front of walls, then they should be designed using passive values reduced to ½ of level backfill passive resistance values.

Unrestrained (yielding) cantilever walls should be designed for the active equivalent-fluid weight value provided above for very low expansive soils that are free draining. In the design of walls restrained from movement at the top (non-yielding) such as basement or elevator pit/utility vaults, the at-rest equivalent fluid weight value should be used. Total depth of retained earth for design of cantilever walls should be measured as the vertical distance below the ground surface measured at the wall face for stem design, or measured at the heel of the footing for overturning and sliding calculations. Should a sloping backfill other than a 2:1

(horizontal:vertical) be constructed above the wall (or a backfill is loaded by an adjacent surcharge load), the equivalent fluid weight values provided above should be re-evaluated on an individual case basis by us. Non-standard wall designs should also be reviewed by us prior to construction to check that the proper soil parameters have been incorporated into the wall design.

For non-restrained walls, an incremental seismic earth pressures of 14H psf, where H is the retaining wall stem height in feet, should be applied for design in addition to static earth and surcharge pressures (triangular). Per 2019 CBC, seismic pressures may be ignored for walls retaining less than 6 feet of soils. Surcharge loads such as adjacent structures, and/or traffic loading should be considered in design of retaining walls. Loads applied within a 1:1 projection down from the surcharging structure on the stem of the wall should also be considered in wall design. In general, 0.30 of uniform vertical surcharge-loads should be applied as a horizontal pressure on cantilever (active) retaining walls, while half of uniform vertical surcharge-loads should be applied as a horizontal pressure on braced (at-rest) retaining walls (assuming sand soils backfill). Hydrostatic pressure should also be incorporated into to the above equivalent fluid pressures, where applicable.

All retaining walls should be provided with appropriate drainage. The outlet pipe should be sloped to drain to a suitable outlet. Typical wall drainage design is illustrated in Appendix E, *Retaining Wall Backfill and Subdrain Detail*. Wall backfill should be non-expansive (EI $\leq$ 21) sands compacted by mechanical methods to a minimum of 90 percent relative compaction (ASTM D 1557). Clayey site soils should not be used as wall backfill. Walls should not be backfilled until wall concrete attains the 28-day compressive strength and/or as determined by the Structural Engineer that the wall is structurally capable of supporting backfill. Lightweight compaction equipment should be used, unless otherwise approved by the Structural Engineer.

5.6 Foundation Setback from Slopes

We recommend a minimum horizontal setback distance from the face of slopes for all structural footings (retaining and decorative walls, flatwork, building footings, pools, etc.). This distance is measured from the outside bottom edge of the footing horizontally to the slope face (or the face of a retaining wall) and should be a minimum of H/2, where H is the slope height (in feet). The setback requirements should follow Chapter 18 of the 2019 CBC and the recommended setbacks provided in table below. The most stringent of both setback should apply to this project.

Table 6. Footing Setbacks

Slope Height	Recommended Footing Setback
<5 feet	5 feet minimum
5 to 15 feet	7 feet minimum
>15 feet	H/2, where H is the slope height, not to exceed 10 feet to 2:1 slope face

*Per county minimum or as calculated

The soils within the structural setback area generally possess poor lateral stability and improvements (such as retaining walls, pools, sidewalks, fences, pavements, decorative flatwork, etc.) constructed within this setback area will be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a pier and grade-beam foundation system to support the improvement. The deepened footing should meet the setback described above. Modifications of slope inclinations near foundations may increase the setback and should be reviewed by the design team prior to completion of design or implementation.

5.7 Concrete Flatwork

Exterior concrete slabs-on-grade should have a minimum thickness of 4 inches. Common Type II cement should be adequate for concrete flatwork not exposed to recycled water. Type V cement and a water:cement ratio of 0.45 should be used for concrete exposed to recycled water.

Concrete flatwork should be placed on compacted fill. If this material has been disturbed or become dry or desiccated, the subgrade soil to a depth of 12 inches should be moisture conditioned to near optimum moisture content and recompacted to a minimum of 90 percent relative compaction. Moisture content should be checked 48 hours prior to placing concrete.

As discussed in conjunction with floor slabs, minor cracking of concrete after curing due to expansion, drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water-to-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.

The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking. Inclusion of joints at frequent intervals and reinforcement will help control the locations of cracking, and improve aesthetics. Control joints should be spaced at regular intervals no greater than 6 feet on-center and have appropriate joints and saw cuts in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. If cracking occurs, repairs may be needed to mitigate a trip hazard (should it develop) and/or improve the appearance.

Landscape areas must be separated from pavements with concrete curbs and/or edge drains. Excessive over-irrigation will have an adverse effect on adjacent pavements. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from paving will result in premature pavement distress.

6.0 GEOTECHNICAL CONSTRUCTION SERVICES

The long term integrity and performance of foundation and earthwork improvements for residential development projects is closely attributable to an adequate construction review process. Geotechnical review is of paramount importance as a part of this process. To verify that project grading and foundation plans conform to the recommendations of this report, we recommend Leighton professionals be retained to review these plan(s) once available.

Direct observation and testing by the geotechnical professional during remedial grading and foundation construction allows for an assessment of exposed soil conditions and verification of the geotechnical conclusions and recommendations presented herein. Our presence also affords opportunity to provide alterative recommendations where/if warranted to address unanticipated conditions in the field. We therefore recommend that Leighton be retained during rough and precise grading to provide these services. Our geotechnical observation and testing services are typically required by the city for the following:

- After completion of site demolition and clearing;
- During ground preparation, excavations, overexcavation of soils;
- During compaction of all fill materials;
- Following foundation excavation, prior to placement of any forms, steel or concrete;
- During slab-on-grade, driveway and flatwork subgrade preparation,
- During street, curb-gutter base placement and asphalt paving;
- During utility trench backfilling and compaction; and
- When any unusual conditions are encountered.

7.0 LIMITATIONS

Leighton and Associates, Inc.'s work was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in California at this time. No other warranty, express or implied, is made as to the conclusions and professional opinions included in this report.

This report is issued with the understanding that it is the responsibility of the owner or a duly authorized agent acting on behalf of the owner, to ensure that information and recommendations contained herein are brought to the attention of the necessary design consultants for this project and incorporated into plans and specifications.

The conclusions and preliminary recommendations in this report are based in part upon data that were obtained from a necessarily limited number of observations, site visits, excavations, samples and tests. Such information can be obtained only with respect to the specific locations explored, and therefore may not completely define all subsurface conditions throughout the site. The nature of many sites is that differing geotechnical and/or geological conditions can occur within small distances and under varying climatic conditions. Furthermore, changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report should be considered preliminary if unanticipated conditions are encountered and additional explorations, testing and analyses may be necessary to develop alternative recommendations.

Any persons using this report for bidding or construction purposes should perform such independent investigations as they deem necessary to satisfy themselves as to the surface and/or subsurface conditions to be encountered and the procedures to be used in the performance of work on the subject site. For additional information about geotechnical engineering studies and this reports and its applicability, provided by the Geoprofessional Business Association (GBA), the client is referred to Appendix F, *GBA Important Information About This Geotechnical Engineering Report.*

8.0 REFERENCES

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Project: 22901	Eng/Geol: SIS/JLH	
Scale: 1 " = 2,000 '	Date: April 2024	
Base Map: ESRI ArcGIS Online 2024		

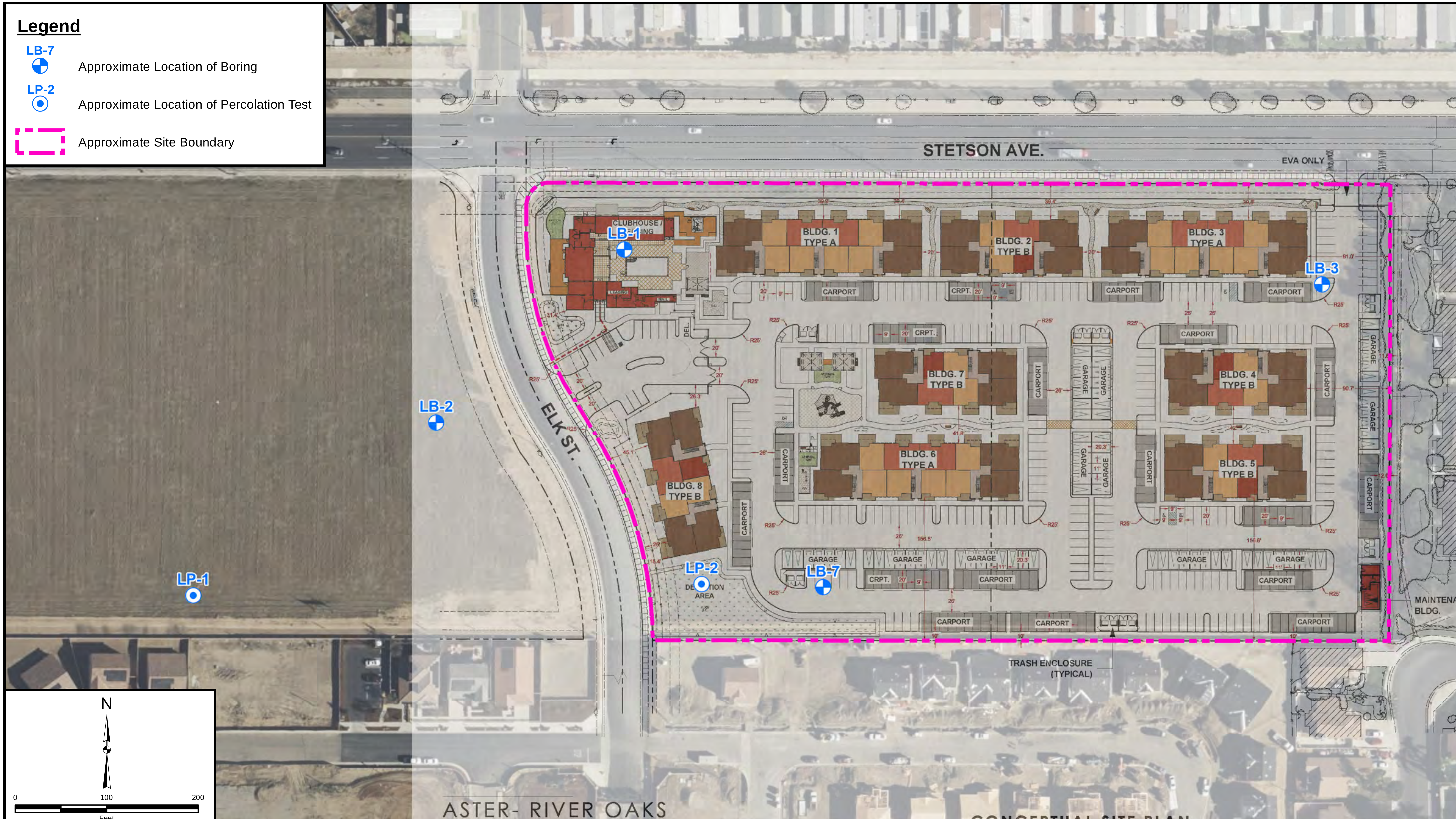
SITE LOCATION MAP

River Oaks Ranch
Residential Development Project
City of Hemet, Riverside County, California

FIGURE 1

Leighton

Map Saved as Z:\Project Files\GA-HZ\Highpointe\22901 - HPC Aster-River Oaks Rch Suppl Consultin\GIS\Maps\22901_F01_SLM_2024-04-08.mxd on 4/8/2024 11:26:04 AM Author: KVM (mmurphy)



Project: 22901	Eng/Geol: SIS/JLH
Scale:1 " = 100 '	Date: April 2024

Reference: © 2024 Microsoft Corporation © 2024 Maxar ©CNES (2024) Distribution Airbus DS Conceptual Site Plan by Danielian Associates, 3/6/2024.

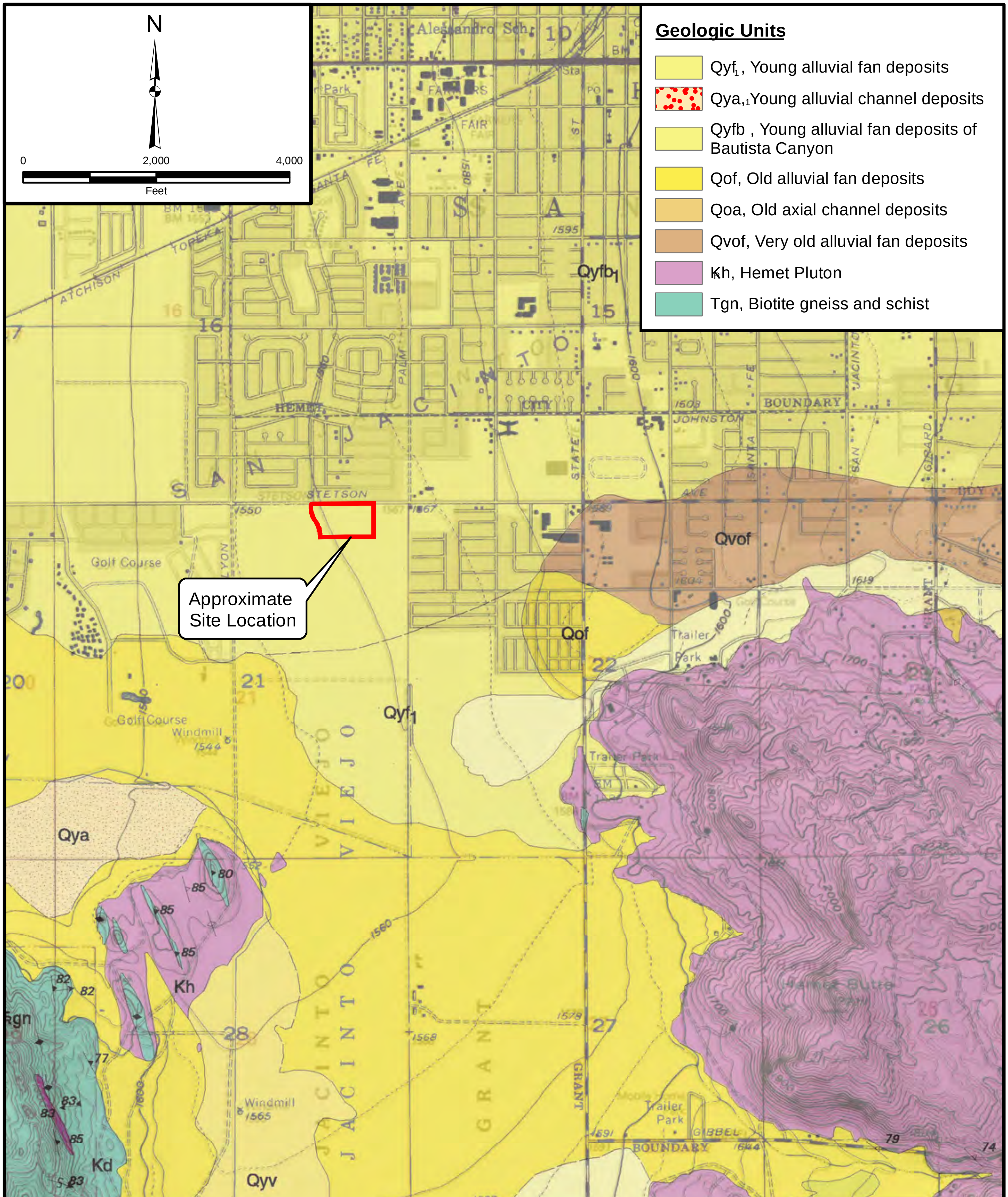
EXPLORATION MAP

River Oaks Ranch
Residential Development Project
City of Hemet, Riverside County, California

FIGURE 2

Leighton

Map Saved as Z:\Project Files\GA-HZ\Highpointe\22901 - HPC Aster-River Oaks Rch Suppl Consultin\GIS\Maps\22901_F02_EM_2024-04-08a.mxd on 4/8/2024 11:07:46 AM Author: KVM (mmurphy)



Project: 13369.002	Eng/Geol: JLH
Scale: 1 " = 2,000 '	Date: April 2024

Basemap: USGS Topo Map Service from Esri, 2021
Reference: Geologic Compilation of Hemet 7.5' Quadrangle, Riverside County, California, Morton and Matti, 2005

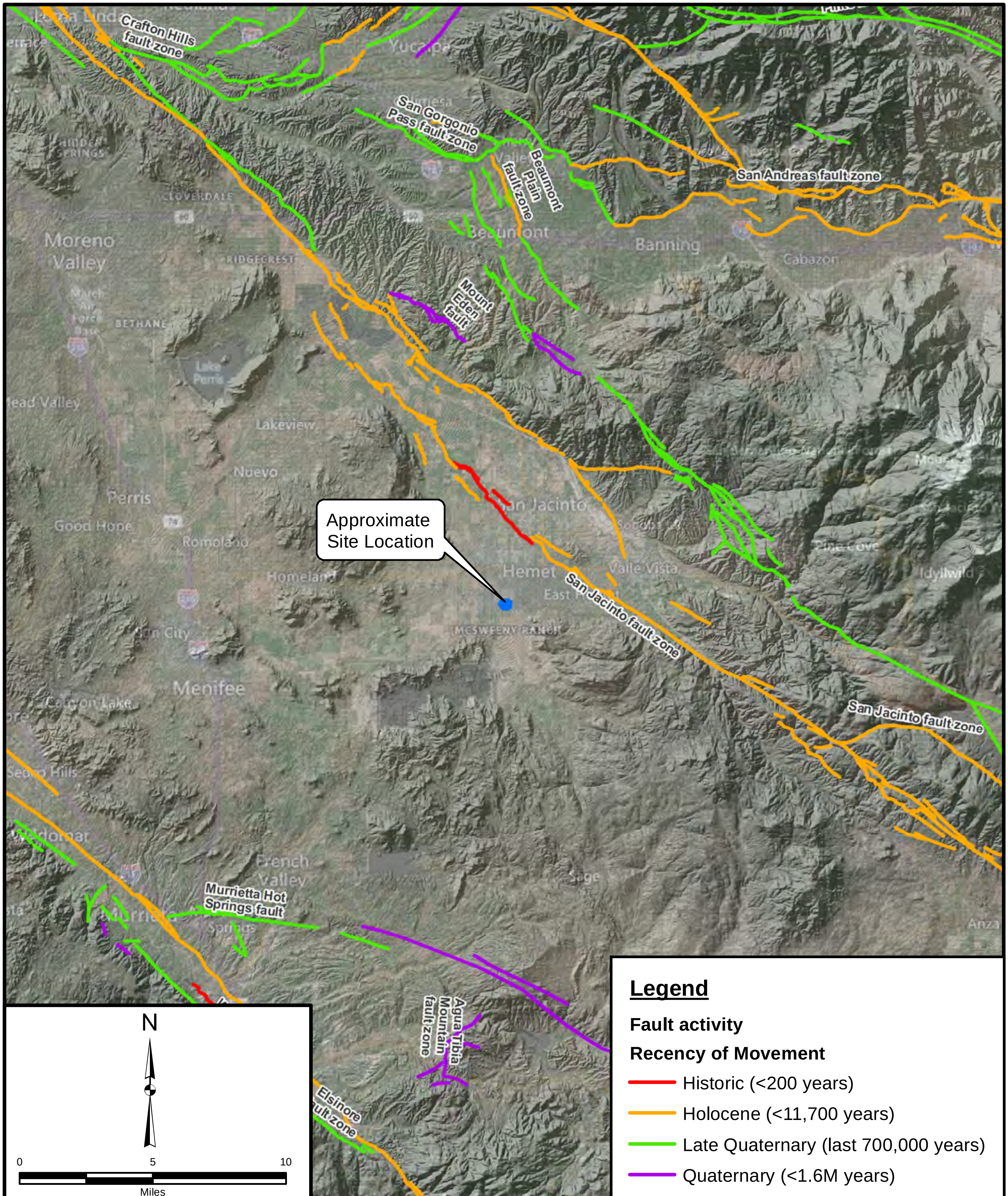
REGIONAL GEOLOGY MAP

River Oaks Ranch
Residential Development Project
City of Hemet, Riverside County, California

FIGURE 3

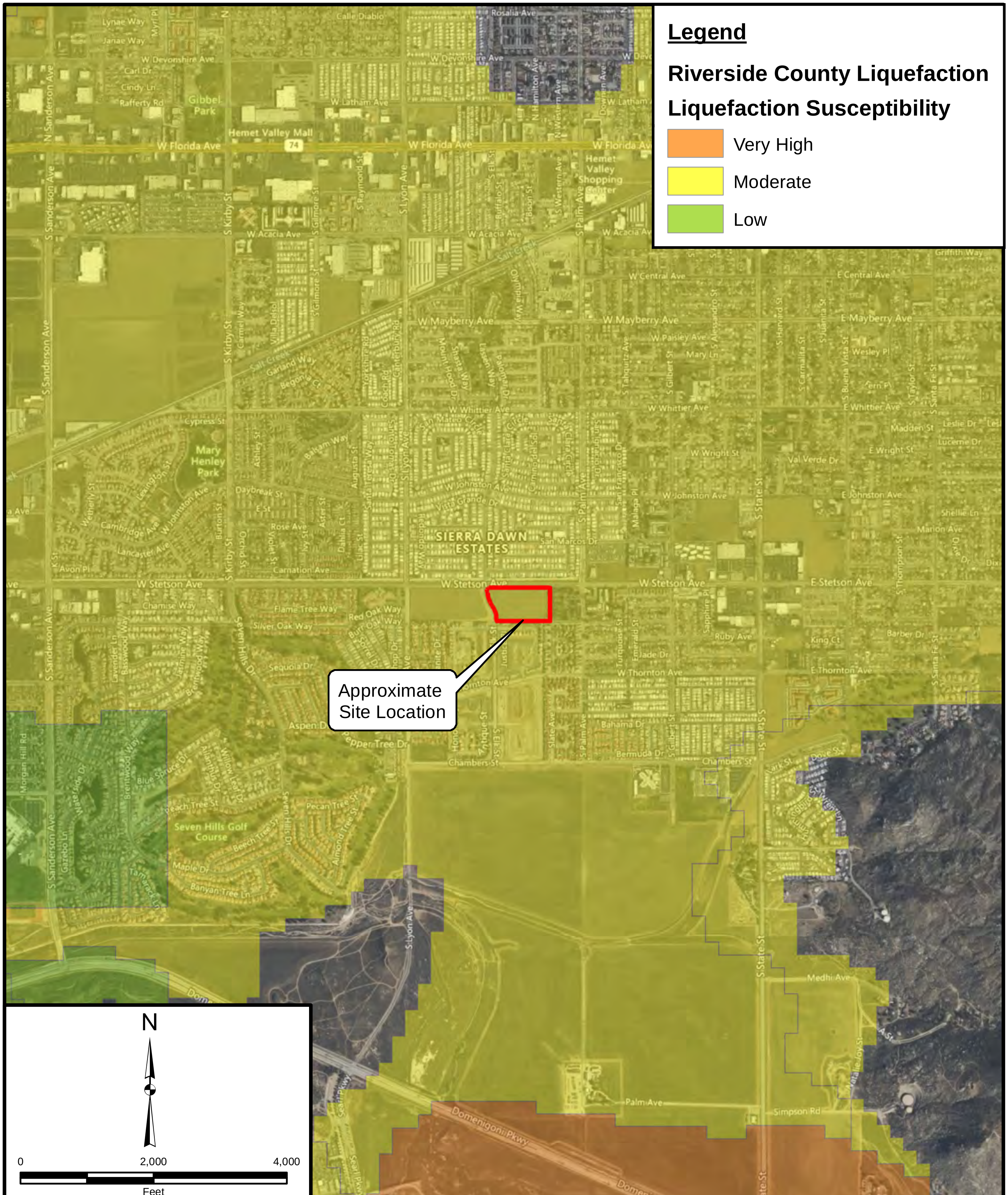
Leighton

Map Saved as Z:\Project Files\GA-HZ\Highpointe\22901 - HPC Aster-River Oaks Rch Suppl Consultin\GIS\Maps\22901_F03_RGM_2024-04-08.mxd on 4/8/2024 1:03:30 PM Author: KVM (mmurphy)



Project: 22901	Eng/Geol: SIS/JLH	REGIONAL FAULT MAP River Oaks Ranch Residential Development Project City of Hemet, Riverside County, California	FIGURE 4
Scale: 1 " = 5 miles	Date: April 2024		Leighton
Base Map: ESRI ArcGIS Online 2024 Reference: maps.conservation.ca.gov			

Map Saved as Z:\Project Files\GA-HZ\Highpointe\22901 - HPC Aster-River Oaks Rch Suppl Consultin\GIS\Maps\22901-004_RFM_2024-04-08.mxd on 12/15/2021 2:56:57 PM Author: KVM (mmurphy)



Project: 13369.002	Eng/Geol: JLH
Scale: 1 " = 2,000 '	Date: April 2024

Base Map: ESRI ArcGIS Online 2024
Reference: Riverside County Liquefaction Map, Riverside TLMA

LIQUEFACTION HAZARD MAP

River Oaks Ranch
Residential Development Project
City of Hemet, Riverside County, California

FIGURE 5

Leighton

Map Saved as Z:\Project Files\GA-HZ\Highpointe\22901 - HPC Aster-River Oaks Rch Suppl Consultin\GIS\Maps\22901_F05_LQHM_2024-04-08.mxd on 4/8/2024 1:04:07 PM Author: KVM (mmurphy)

APPENDIX A

EXPLORATORY BORING LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	13369.003	**Date Drilled**	4-11-22
Project	Highpointe River Oaks Ranch	**Logged By**	LFO
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	LFO

SOIL DESCRIPTION

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. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	**Quaternary Young Alluvial Fan Deposits (Qyf1)** @0': Tilled topsoil w/ dry grass Silty SAND, grayish brown, slightly moist, predominantly fine sand	CR, EI, MD, RV SA
	5			R-1	5 8 10	98.2	5.8	ML	@5': Sandy SILT, gray brown, slightly moist, predominantly fine sand, thinly laminated, medium dense	
	10			R-2	5 5 8	98.2	5.4		@10': SILT, light olive brown, moist, micaceous, predominantly fine sand, medium dense (CO = -0.75)	CO
	15			R-3	6 7 7	96.6	16.3		@15': SILT, olive gray, moist, micaceous, predominantly fine sand, trace clay, low plasticity, massive, medium dense (CO = 0.20)	CO
	20			R-4	4 7 7	108.2	7.8	S(ML)	@20': Sandy SILT, gray brown, moist, micaceous, predominatly fine sand, thinly laminated, medium dense	
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

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

GEOTECHNICAL BORING LOG LB-1

Project No.	13369.003	**Date Drilled**	4-11-22
Project	Highpointe River Oaks Ranch	**Logged By**	LFO
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	LFO

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									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. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			R-5	12 12 22	121.1	10.5	S(ML)	@30': Sandy SILT, gray brown, moist, micaceous, predominatly fine sand, thinly laminated, medium dense	
								SC	**<u>Quaternary Old Alluvial Fan Deposits (Qof)</u>** @31.4': Clayey SAND, red brown, moist, micaceous, predominantly fine sand, trace coarse sand, low plasticity	
	35									
	40			R-6	18 30 39	121.7	8.7	SM	@40': Silty SAND, yellow brown, moist, micaceous, predominantly fine sand, trace medium sand of granitic origin, dense	
	45									
	50			R-7	25 38 36				@50': Silty SAND, medium brown with dark brown mottling, moist, micaceous, predominantly fine sand, trace coarse sand, trace clay, dense	
								ML	@51.3': Sandy SILT, gray brown, slightly moist, micaceous, predominantly fine sand	
									Total Depth: 51.5 feet bgs **No groundwater encountered during drilling.** **Borehole backfilled with soil cuttings.**	
	55									
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

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

GEOTECHNICAL BORING LOG LB-2

Project No.	13369.003	**Date Drilled**	4-11-22
Project	Highpointe River Oaks Ranch	**Logged By**	LFO
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	LFO

SOIL DESCRIPTION

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. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SM	**Quaternary Young Alluvial Fan Deposits (Qyf1)** @0': Tilled top soil w/ dry vegetation Silty SAND, gray brown, slightly moist, micaceous, predominantly fine sand	
				R-1	2 4 5	92.6	3.3	SP-SM	@2.5': Poorly-graded SAND w/ Silt, gray brown, slightly moist, micaceous, predominatly fine sand, trace medium sand, loose	
								SM	@3.8': Silty SAND, gray brown, moist, micaceous, predominantly fine sand, massive	
	5			R-2	5 4 7	95.8	7.6		@5': Silty SAND, gray brown, moist, micaceous, predominantly fine sand, trace coarse sand, abundant carbonate stringers, laminated, loose	
				R-3	10 8 11	98.9	5.5	CL-ML	@7.5': Silty CLAY, olive gray, moist, micaceous, predominantly fine sand, decreased abundance of carbonate stringers, massive, medium dense (CO = -1.02)	CO
	10			R-4				SM	@10': Silty SAND, gray brown, moist, micaceous, predominantly fine sand, some carbonate stringers, massive	
	15			R-5	8 8 10	97.5	11.7	ML	@15': SILT, olive gray, moist, micaceous, moist, predominantly fine sand, trace medium sand, trace clay, low plasticity, laminated, medium dense (CO = -0.39)	CO
	20			R-6	12 9 11			SM	@20': Silty SAND, brown, moist, micaceous, moist, predominantly fine sand, trace medium sand, trace clay, low plasticity, laminated, medium dense	
	25			R-7				SM	**Quaternary Old Alluvial Fan Deposits (Qof)** @25': Silty SAND, dark red brown, moist, predominantly fine sand, trace medium to coarse sand, trace clay, low plasticity, massive	
									Total Depth: 26.5 feet bgs **No groundwater encountered during drilling.** **Borehole backfilled with soil cuttings.**	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-3

Project No.	13369.003	**Date Drilled**	4-11-22
Project	Highpointe River Oaks Ranch	**Logged By**	LFO
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	LFO

SOIL DESCRIPTION

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. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SM	**Quaternary Young Alluvial Fan Deposits (Qyf1)** @0': Tilled soil w/ dry vegetation Silty SAND, olive brown, slightly moist, micaceous, predominantly fine sand, organic material	MD, SA
				R-1	4 5 8	96.9	6.4	SP-SM	@2.5': Silty SAND, olive brown, moist, micaceous, predominantly fine sand, trace carbonate stringers, medium dense	
	5			R-2	6 11 15	100.4	11.3	SM	@5': Silty SAND, olive brown, moist, micaceous, predominantly fine sand, trace carbonate stringers, laminated, medium dense	
				R-3	7 10 11				@7.5': Silty SAND, grayish brown, moist, micaceous, predominantly fine sand, trace carbonate stringers, laminated, medium dense	
								SP	@8.3': Poorly-graded SAND, grayish brown, slightly moist, micaceous, predominantly fine to medium sand	
	10			R-4	7 9 12	103.0	2.6	SM	@10': Silty SAND, grayish brown, moist, micaceous, predominantly fine sand, trace medium sand, trace Fe-stained veins, medium dense	
	15			R-5	10 10 10	100.1	6.5		@15': Silty SAND, olive gray, moist, micaceous, predominantly fine sand, trace medium sand, trace Fe-stained veins, medium dense (CO = -0.39)	CO
								SC	@16.3': Clayey SAND, grayish brown, moist, micaceous, predominantly fine sand, low plasticity, laminated	
	20			R-6	5 11 14	114.4	4.8	ML-SM	**Quaternary Old Alluvial Fan Deposits (Qof)** @20': SILT w/ Sand, reddish brown, moist, micaceous, predominantly fine to medium sand, trace coarse sand, trace clay, medium dense	
	25			R-7	15 25 36				@25': SILT w/ Sand, reddish brown, moist, micaceous, predominantly medium sand, trace coarse sand, trace clay, very dense	
									Total Depth: 26.5 feet bgs **No groundwater encountered during drilling.** **Borehole backfilled with soil cuttings.**	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-7

Project No.	13369.003	**Date Drilled**	4-11-22
Project	Highpointe River Oaks Ranch	**Logged By**	LFO
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	LFO

SOIL DESCRIPTION

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. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SM	**Quaternary Young Alluvial Fan Deposits (Qyf1)** @0': Tilled top soil w/ dry vegetation Silty SAND, gray brown, dry to slightly moist, predominatly fine sand, some organic material	
	5			R-1	5 5 12	104.7	6.6		@5': Silty SAND, gray brown ,moist, micaceous, predominatly fine sand, trace medium sand, medium dense	
	10			R-2	8 9 9	109.4	1.5	SW-SM	@10': Well-graded SAND w/ Silt, gray brown, predominantly fine to medium sand, trace coarse sand, trace gravel, medium dense (CO = -0.74) @11.3': Grades to fine sand	CO
	15			R-3	7 7 9	95.0	12.7	CL-ML	@15': Silty CLAY, olive, moist, micaceous, low plasticity, medium dense (CO = -0.82)	CO
									T.D. 16.5 feet bgs **No groundwater encountered during drilling.** **Borehole backfilled with soil cuttings.**	
	20									
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LP-1

Project No.	13369.003	**Date Drilled**	4-11-22
Project	Highpointe River Oaks Ranch	**Logged By**	LFO
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	LFO

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	**<u>Quaternary Young Alluvial Fan Deposits (Qyf1)</u>** @0': Tilled soil w/ dry vegetation Silty SAND, gray brown, slightly moist, fine sand	
				R-1	2 7 7				@2.5': Silty SAND, gray brown slightly moist, micaceous, fine sand, abundant carbonate stringers and blebs, medium dense	
	5								**T.D. 4.5 feet bgs** **No groundwater encountered during drilling.** **Borehole converted to percolation well with 5 feet of 0.020-inch slotted PVC pipe and submurged with Monterey No. 3 sand.** **Borehole backfilled with soil cuttings after well removal.**	
	10									
	15									
	20									
	25									
	30									

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. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LP-2

Project No.	13369.003	**Date Drilled**	4-11-22
Project	Highpointe River Oaks Ranch	**Logged By**	LFO
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	LFO

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									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. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0			B-1				SM	<u>**Quaternary Young Alluvial Fan Deposits (Qyf1)**</u> @0': Tilled soil w/ dry vegetation Silty SAND, gray brown, slightly moist to dry, fine sand	
				R-1	7 7 9				@2.5': Silty SAND, gray brown slightly moist, micaceous, predominantly fine to medium sand, trace coarse sand, medium dense @3.8': Carbonate stringers appear	
	5								**T.D. 4.37 feet bgs** **No groundwater encountered during drilling.** **Borehole converted to percolation well with 5 feet of 0.020-inch slotted PVC pipe and submurged with Monterey No. 3 sand.** **Borehole backfilled with soil cuttings after well removal.**	
	10									
	15									
	20									
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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APPENDIX B

LABORATORY TEST RESULTS

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: HP Hemet | Tested By : G. Berdy | Date: 04/20/22

Project No. : 13369.003 | Checked By: A. Santos | Date: 05/02/22

Boring No.	LB-1	LB-4		
Sample No.	B-1	B-1		
Sample Depth (ft)	0-5	0-5		
Soil Identification:	Grayish brown (SM)	Olive brown s(ML)		
Wet Weight of Soil + Container (g)	0.00	0.00		
Dry Weight of Soil + Container (g)	0.00	0.00		
Weight of Container (g)	1.00	1.00		
Moisture Content (%)	0.00	0.00		
Weight of Soaked Soil (g)	100.18	100.63		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	17	9		
Crucible No.	1	3		
Furnace Temperature (°C)	860	860		
Time In / Time Out	7:15/8:00	7:00/7:45		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	25.7365	24.5143		
Wt. of Crucible (g)	25.7339	24.5126		
Wt. of Residue (g) (A)	0.0026	0.0017		
PPM of Sulfate (A) x 41150	106.99	69.95		
PPM of Sulfate, Dry Weight Basis	**107**	**70**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15	15		
ml of AgNO3 Soln. Used in Titration (C)	0.5	0.3		
PPM of Chloride (C -0.2) * 100 * 30 / B	60	20		
PPM of Chloride, Dry Wt. Basis	**60**	**20**		

pH TEST, DOT California Test 643

pH Value	**8.42**	**8.06**		
Temperature °C	20.9	20.3		

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	HP Hemet		Tested By :	G. Berdy	Date:	04/25/22
Project No. :	13369.003		Checked By:	A. Santos	Date:	05/02/22
Boring No.:	LB-1		Depth (ft.) :	0-5		
Sample No. :	B-1					

Soil Identification:* Grayish brown (SM)

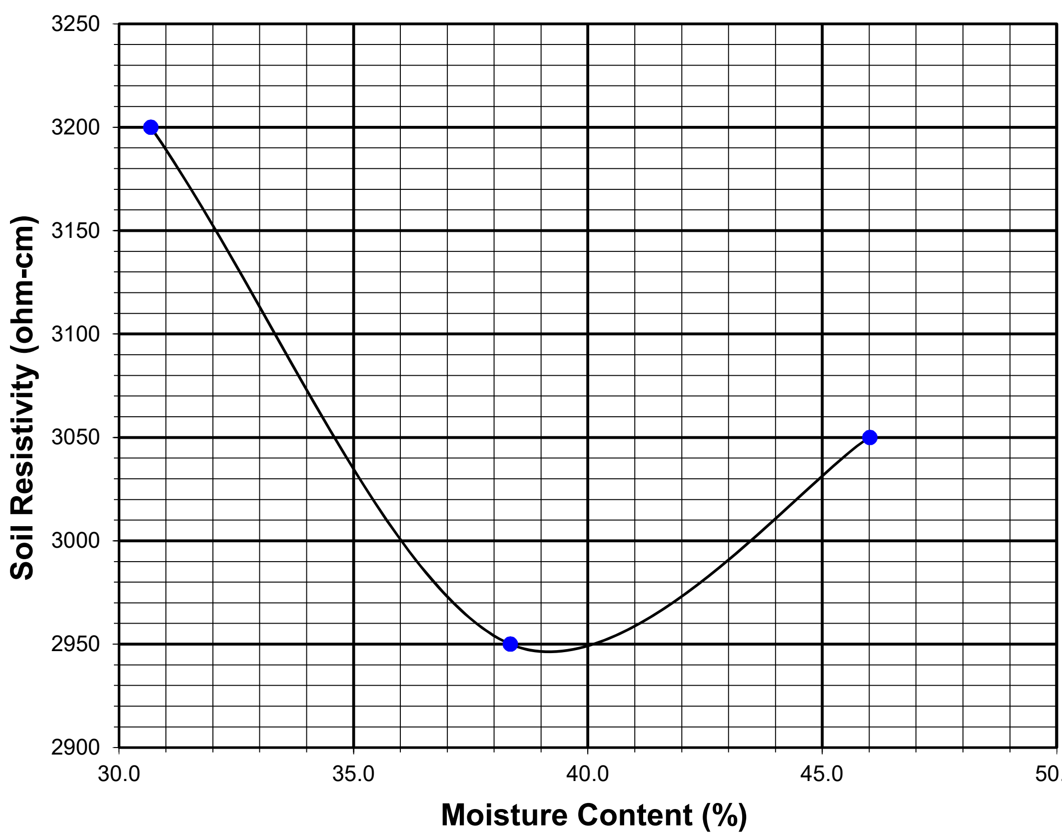
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	40	30.67	3200	3200
2	50	38.34	2950	2950
3	60	46.01	3050	3050
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.40
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	
2946	**39.2**	**107**	**60**	**8.42**	**20.9**



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	HP Hemet	Tested By :	J. Domingo Date: 05/02/22
Project No. :	13369.003	Checked By:	A. Santos Date: 05/02/22
Boring No.:	LB-4	Depth (ft.) :	0-5
Sample No. :	B-1		

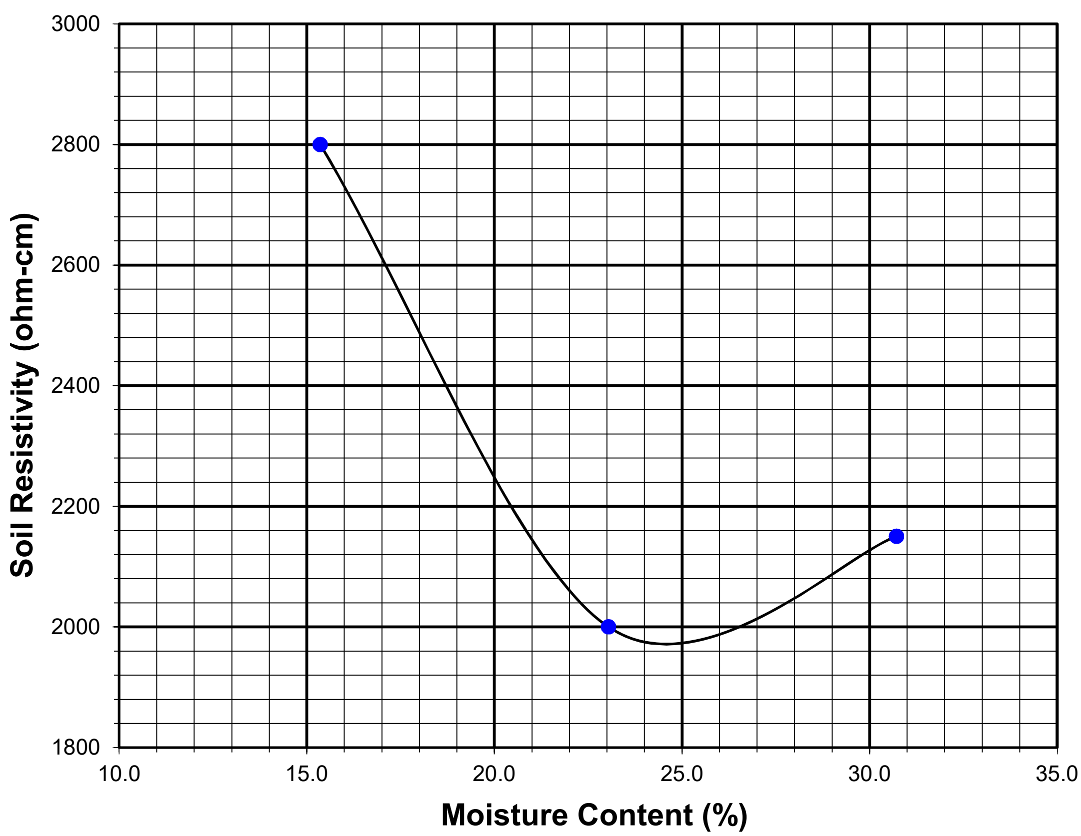
Soil Identification:* Olive brown s(ML)

*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	20	15.36	2800	2800
2	30	23.04	2000	2000
3	40	30.72	2150	2150
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.20
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	
1930	**24.5**	**70**	**20**	**8.06**	**20.3**



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EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	HP Hemet	Tested By:	G. Berdy	Date:	04/20/22
Project No.:	13369.003	Checked By:	A. Santos	Date:	05/02/22
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Grayish brown silty sand (SM)				

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

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0120
Wt. Comp. Soil + Mold	(g)	591.40	456.70
Wt. of Mold	(g)	163.50	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	862.50	620.20
Dry Wt. of Soil + Cont.	(g)	804.50	562.66
Wt. of Container	(g)	0.00	163.50
Moisture Content	(%)	7.21	14.42
Wet Density	(pcf)	129.1	136.1
Dry Density	(pcf)	120.4	119.0
Void Ratio		0.400	0.417
Total Porosity		0.286	0.294
Pore Volume	(cc)	59.2	61.6
Degree of Saturation (%) [S_{meas}]		**48.6**	93.3

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.)
04/20/22	10:18	1.0	0	0.6275
04/20/22	10:28	1.0	10	0.6275
Add Distilled Water to the Specimen				
04/20/22	23:17	1.0	769	0.6380
04/21/22	5:19	1.0	1131	0.6395
04/21/22	6:32	1.0	1204	0.6395

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**12**

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	HP Hemet	Tested By:	J. Gonzalez	Date:	04/19/22
Project No.:	13369.003	Checked By:	A. Santos	Date:	04/29/22
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Grayish brown silty sand (SM)				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03330 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3788	3882	3937	3848		
Weight of Mold (g)	1826	1826	1826	1826		
Net Weight of Soil (g)	1962	2056	2111	2022		
Wet Weight of Soil + Cont. (g)	489.4	451.1	497.6	553.5		
Dry Weight of Soil + Cont. (g)	465.8	421.1	454.3	494.2		
Weight of Container (g)	37.6	39.4	40.2	39.4		
Moisture Content (%)	5.51	7.86	10.46	13.04		
Wet Density (pcf)	129.9	136.1	139.8	133.9		
Dry Density (pcf)	123.1	126.2	126.5	118.4		

Maximum Dry Density (pcf) **127.1** **Optimum Moisture Content (%)** **9.4**

PROCEDURE USED

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

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

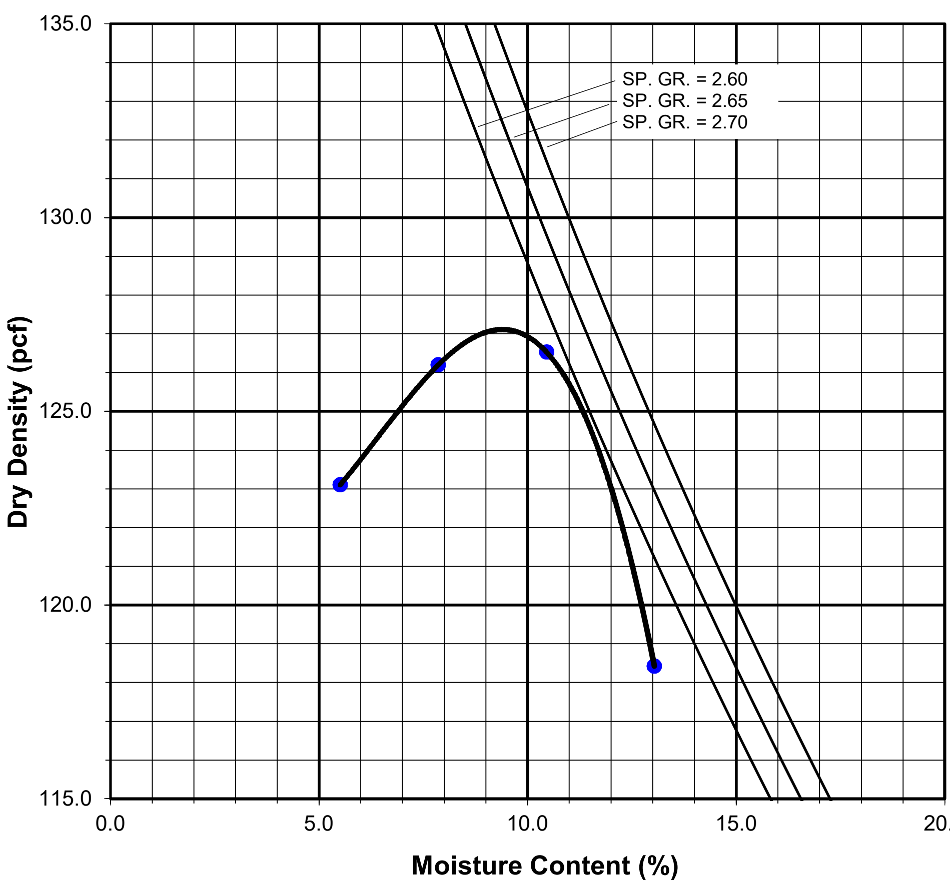
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	HP Hemet	Tested By:	J. Gonzalez	Date:	04/19/22
Project No.:	13369.003	Checked By:	A. Santos	Date:	04/29/22
Boring No.:	LB-3	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown silty sand (SM)				

Preparation Method: [X] Moist [ ] Dry   [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft^3) 0.03330   *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3893	3967	3927			
Weight of Mold (g)	1826	1826	1826			
Net Weight of Soil (g)	2067	2141	2101			
Wet Weight of Soil + Cont. (g)	492.6	482.9	473.3			
Dry Weight of Soil + Cont. (g)	465.6	446.8	429.1			
Weight of Container (g)	37.7	38.7	39.7			
Moisture Content (%)	6.31	8.85	11.35			
Wet Density (pcf)	136.8	141.7	139.1			
Dry Density (pcf)	128.7	130.2	124.9			

Maximum Dry Density (pcf) **130.5**   **Optimum Moisture Content (%)** **8.5**

PROCEDURE USED

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

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

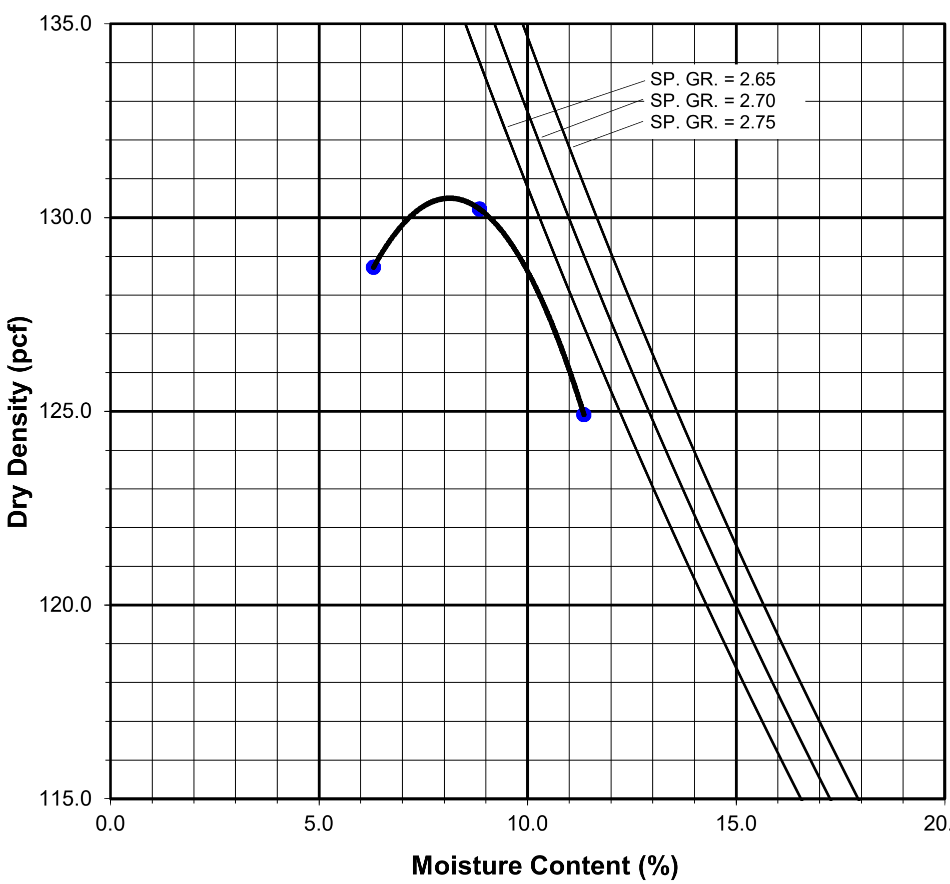
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Project Name: HP Hemet
Project No.: 13369.003

Summary of Pocket Penetrometer Test Results

Tested by: S. Felter Date: 04/28/22
Prepared by: G. Bathala Date: 04/29/22

Boring No.	Sample No.	Depth (ft.)	Readings	Remarks
LB-1	R-1	5	2.75	
	R-2	10	3.25/2.25	
	R-3	15	2.25/2.50	
	R-4	20	>4.50	
	R-5	30	>4.50	
	R-6	40	>4.50	
LB-2	R-1	2.5	3.25	
	R-2	5	>4.50	
	R-3	7.5	2.75/4.50	
	R-5	15	3.00/4.50	
LB-3	R-1	2.5	>4.50	
	R-2	5	>4.50	
	R-4	10	4.00	
	R-5	15	4.00/4.50	
	R-6	20	>4.50	
LB-4	R-1	5	>4.50	
	R-2	10	>4.50	
	R-3	15	>4.50	
LB-5	R-1	2.5	>4.50	
	R-2	5	>4.50	
	R-3	7.5	>4.50	
	R-5	15	>4.50	
LB-6	R-1	2.5		
	R-2	5		
	R-3	7.5	>4.50	
	R-4	10	>4.50	
	R-5	15	>4.50	
LB-7	R-1	5	>4.50	
	R-2	10	2.50/4.25	
	R-3	15	>4.50	

R-VALUE TEST RESULTS

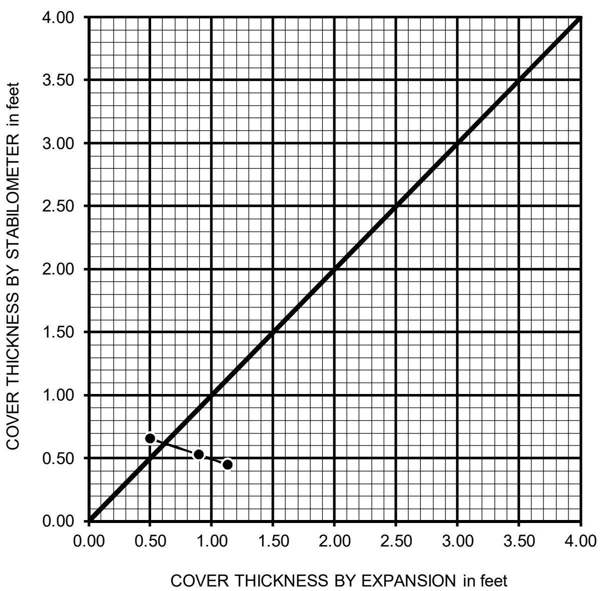
DOT CA Test 301

PROJECT NAME:	HP Hemet	PROJECT NUMBER:	13369.003
BORING NUMBER:	LB-1	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Grayish brown silty sand (SM)	DATE COMPLETED:	4/27/2022

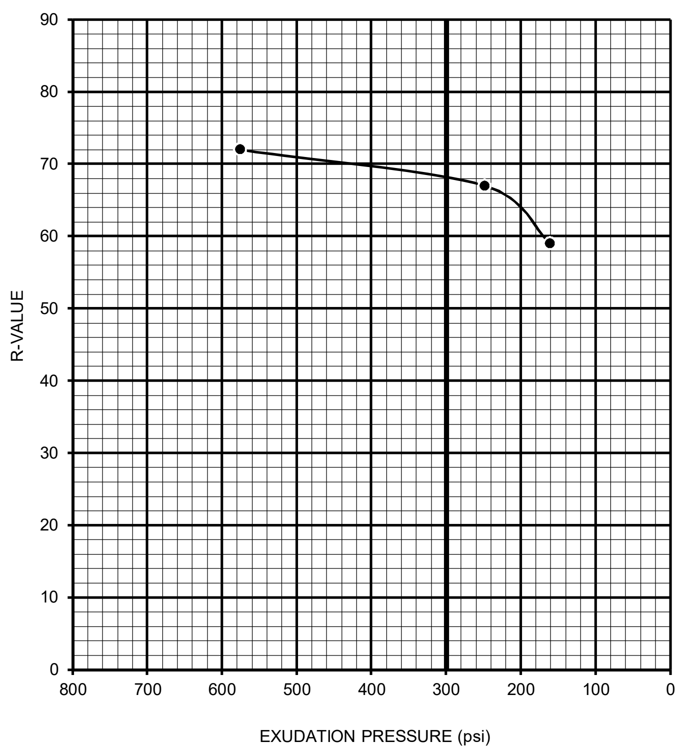
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	10.3	11.1	12.0
HEIGHT OF SAMPLE, Inches	2.47	2.49	2.51
DRY DENSITY, pcf	123.1	121.0	121.2
COMPACTOR PRESSURE, psi	350	300	225
EXUDATION PRESSURE, psi	576	249	161
EXPANSION, Inches x 10exp-4	34	27	15
STABILITY Ph 2,000 lbs (160 psi)	26	31	38
TURNS DISPLACEMENT	4.91	5.10	5.57
R-VALUE UNCORRECTED	72	67	59
R-VALUE CORRECTED	72	67	59

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.45	0.53	0.66
EXPANSION PRESSURE THICKNESS, ft.	1.13	0.90	0.50

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	61
R-VALUE BY EXUDATION:	68
EQUILIBRIUM R-VALUE:	61

Leighton

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

ASTM D 6913

Project Name:	HP Hemet	Tested By:	J. Domingo	Date:	04/21/22
Project No.:	13369.003	Checked By:	A. Santos	Date:	05/02/22
Boring No.:	LB-1	Depth (feet):	0-5		
Sample No.:	B-1				
Soil Identification:	Grayish brown silty sand (SM)				

		Moisture Content of Total Air - Dry Soil	
Container No.:	935	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	652.1	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	107.6	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	544.5	Moisture Content (%)	0.0

After Wet Sieve	Container No.	935
	Wt. of Dry Soil + Container (g)	453.2
	Wt. of Container (g)	107.6
	Dry Wt. of Soil Retained on # 200 Sieve (g)	345.6

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5		
#4	4.75	0.0	100.0
#8	2.36	1.0	99.8
#16	1.18	2.7	99.5
#30	0.600	20.1	96.3
#50	0.300	34.8	93.6
#100	0.150	198.6	63.5
#200	0.075	316.4	41.9
PAN			

GRAVEL: **0 %**

SAND: **58 %**

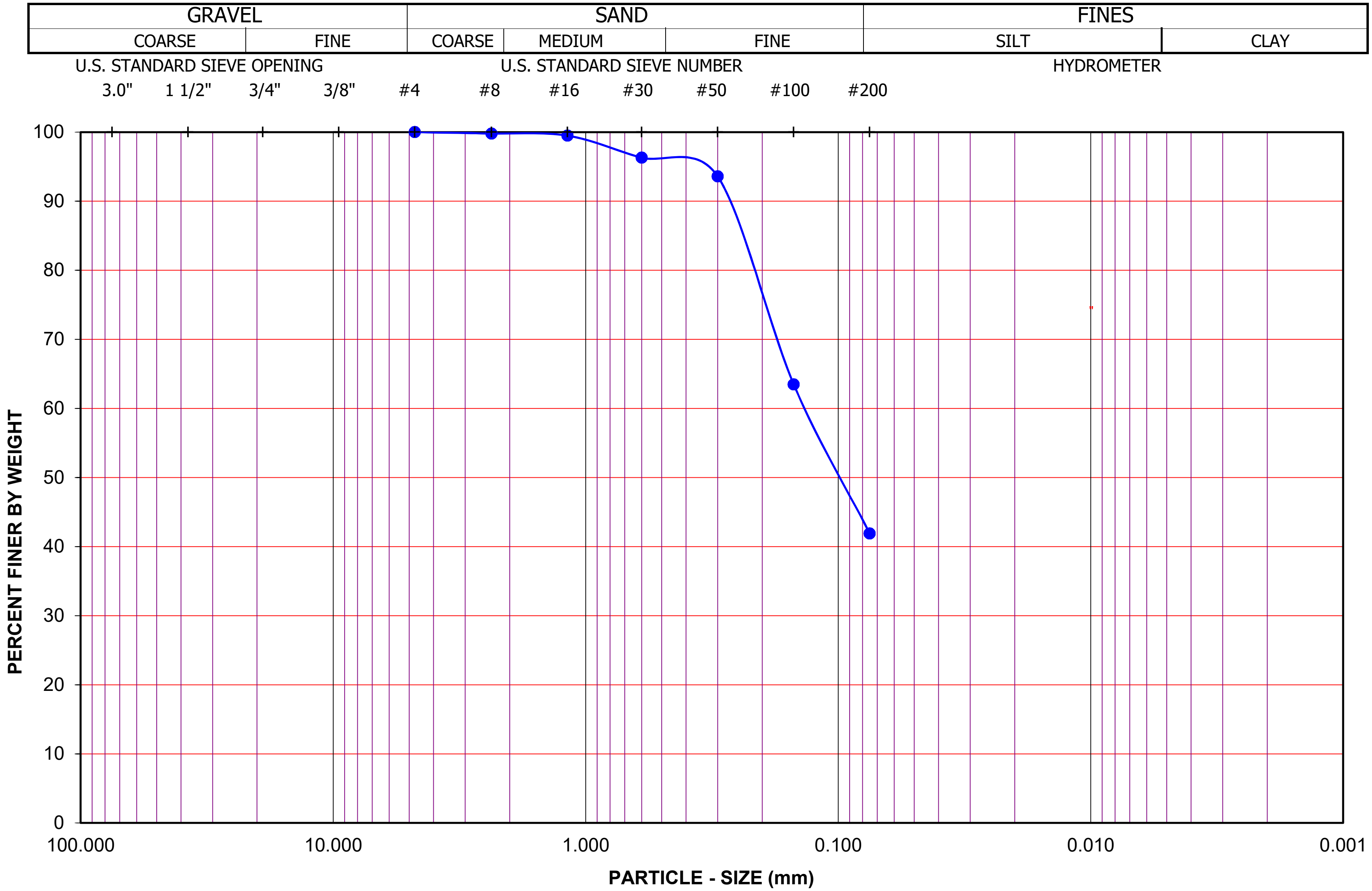
FINES: **42 %**

GROUP SYMBOL: **SM**

Cu = D60/D10 = ______

Cc = (D30)²/(D60*D10) = ______

Remarks: ______



Project Name: HP Hemet

Project No.: 13369.003

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: LB-1 Sample No.: B-1

Depth (feet): 0-5 Soil Type : SM

Soil Identification: Grayish brown silty sand (SM)

GR:SA:FI : (%) 0 : 58 : 42

May-22

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

Project Name: HP Hemet
Project No.: 13369.003
Boring No.: LB-3
Sample No.: B-1
Soil Identification: Olive brown silty sand (SM)

Tested By: J. Domingo Date: 04/21/22
Checked By: A. Santos Date: 05/02/22
Depth (feet): 0-5

		Moisture Content of Total Air - Dry Soil	
Container No.:	929	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	677.7	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	107.5	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	570.2	Moisture Content (%)	0.0

After Wet Sieve	Container No.	929
	Wt. of Dry Soil + Container (g)	430.1
	Wt. of Container (g)	107.5
	Dry Wt. of Soil Retained on # 200 Sieve (g)	322.6

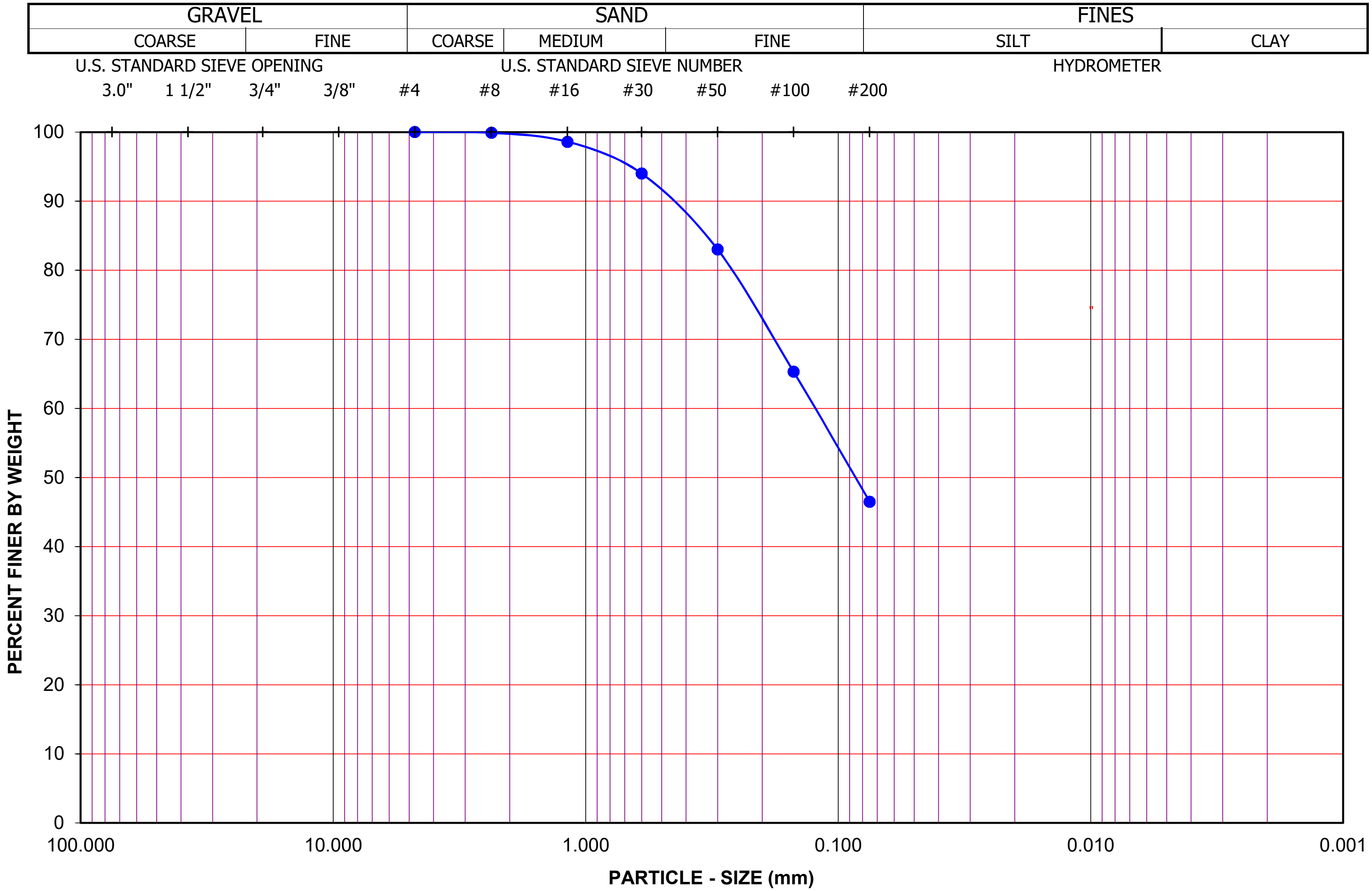
U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5		
#4	4.75	0.0	100.0
#8	2.36	0.5	99.9
#16	1.18	7.9	98.6
#30	0.600	34.0	94.0
#50	0.300	97.0	83.0
#100	0.150	198.1	65.3
#200	0.075	305.3	46.5
PAN			

GRAVEL: **0 %**
SAND: **53 %**
FINES: **47 %**
GROUP SYMBOL: **SM**

Cu = D60/D10 = ______

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

Remarks:



Project Name: HP Hemet

Project No.: 13369.003

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

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

Depth (feet): 0-5 Soil Type : SM

Soil Identification: Olive brown silty sand (SM)

GR:SA:FI : (%) 0 : 53 : 47

May-22

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

Leighton

Project Name:	HP Hemet	Tested By:	G. Bathala	Date:	04/20/22
Project No.:	13369.003	Checked By:	A. Santos	Date:	05/02/22
Boring No.:	LB-1	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.)	10.0		
Sample Description:	Light olive brown silt (ML)				

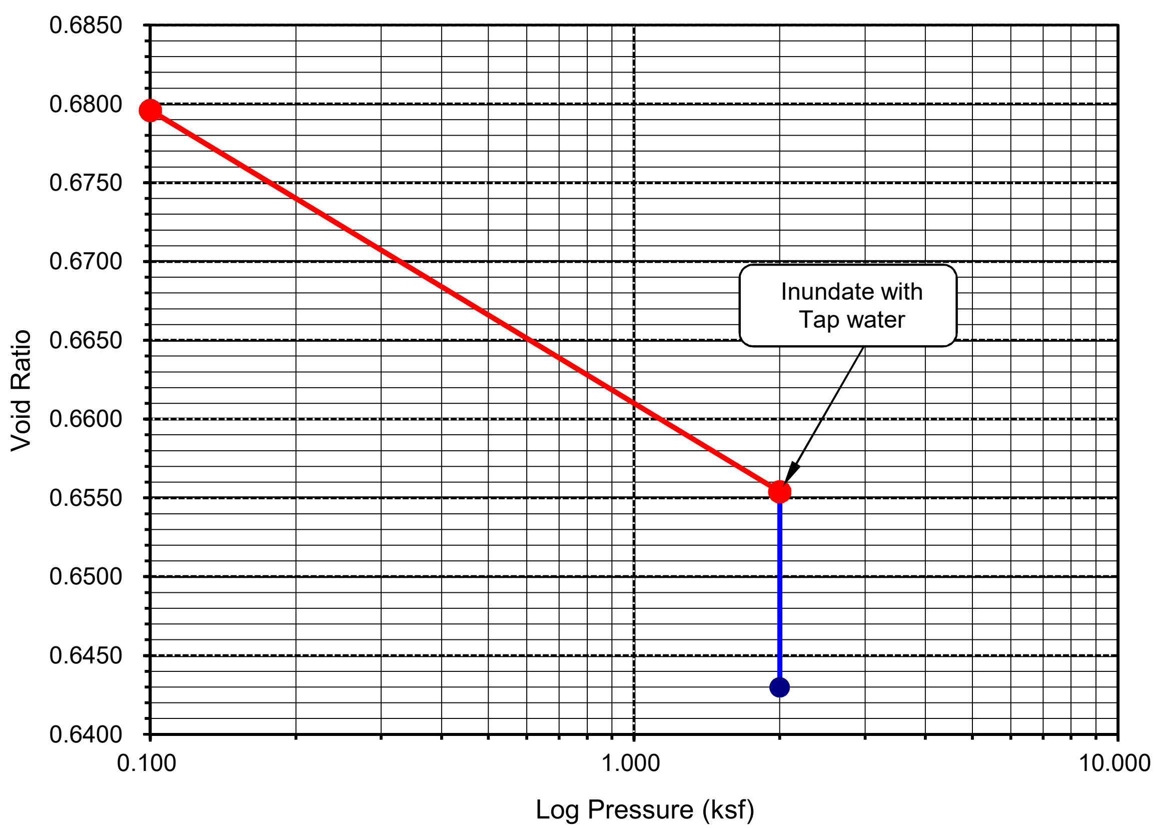
Initial Dry Density (pcf):	100.3
Initial Moisture (%):	5.39
Initial Length (in.):	1.0000
Initial Dial Reading:	0.3083
Diameter(in):	2.415

Final Dry Density (pcf):	102.6
Final Moisture (%) :	20.3
Initial Void Ratio:	0.6799
Specific Gravity(assumed):	2.70
Initial Saturation (%)	21.4

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3081	0.9998	0.00	-0.02	0.6796	-0.02
2.000	0.2917	0.9834	0.20	-1.66	0.6554	-1.46
H2O	0.2843	0.9760	0.20	-2.40	0.6430	-2.20

Percent Swell (+) / Settlement (-) After Inundation = -0.75

Void Ratio - Log Pressure Curve



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

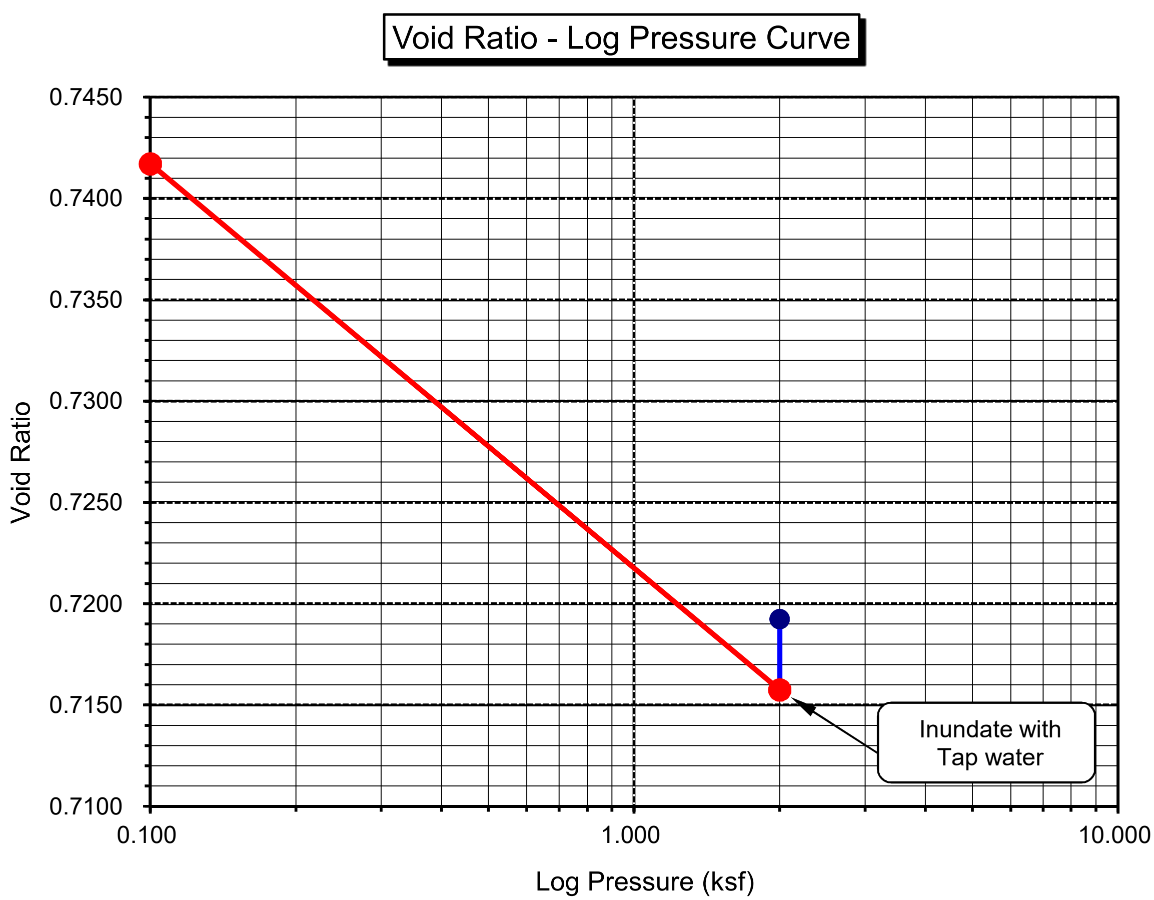
Project Name:	HP Hemet		Tested By:	G. Bathala	Date:	04/20/22
Project No.:	13369.003		Checked By:	A. Santos	Date:	05/02/22
Boring No.:	LB-1		Sample Type:	Ring		
Sample No.:	R-3		Depth (ft.)	15.0		
Sample Description:	Olive gray silt (ML)					

Initial Dry Density (pcf):	96.7
Initial Moisture (%):	16.29
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2749
Diameter(in):	2.415

Final Dry Density (pcf):	98.0
Final Moisture (%) :	32.3
Initial Void Ratio:	0.7428
Specific Gravity(assumed):	2.70
Initial Saturation (%)	59.2

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2743	0.9994	0.00	-0.06	0.7417	-0.06
2.000	0.2561	0.9812	0.33	-1.88	0.7157	-1.55
H2O	0.2581	0.9832	0.33	-1.68	0.7192	-1.35

Percent Swell (+) / Settlement (-) After Inundation = 0.20



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

Project Name:	HP Hemet	Tested By:	G. Bathala	Date:	04/20/22
Project No.:	13369.003	Checked By:	A. Santos	Date:	05/02/22
Boring No.:	LB-2	Sample Type:	Ring		
Sample No.:	R-3	Depth (ft.)	7.5		

Sample Description: Olive gray silty clay (CL-ML)

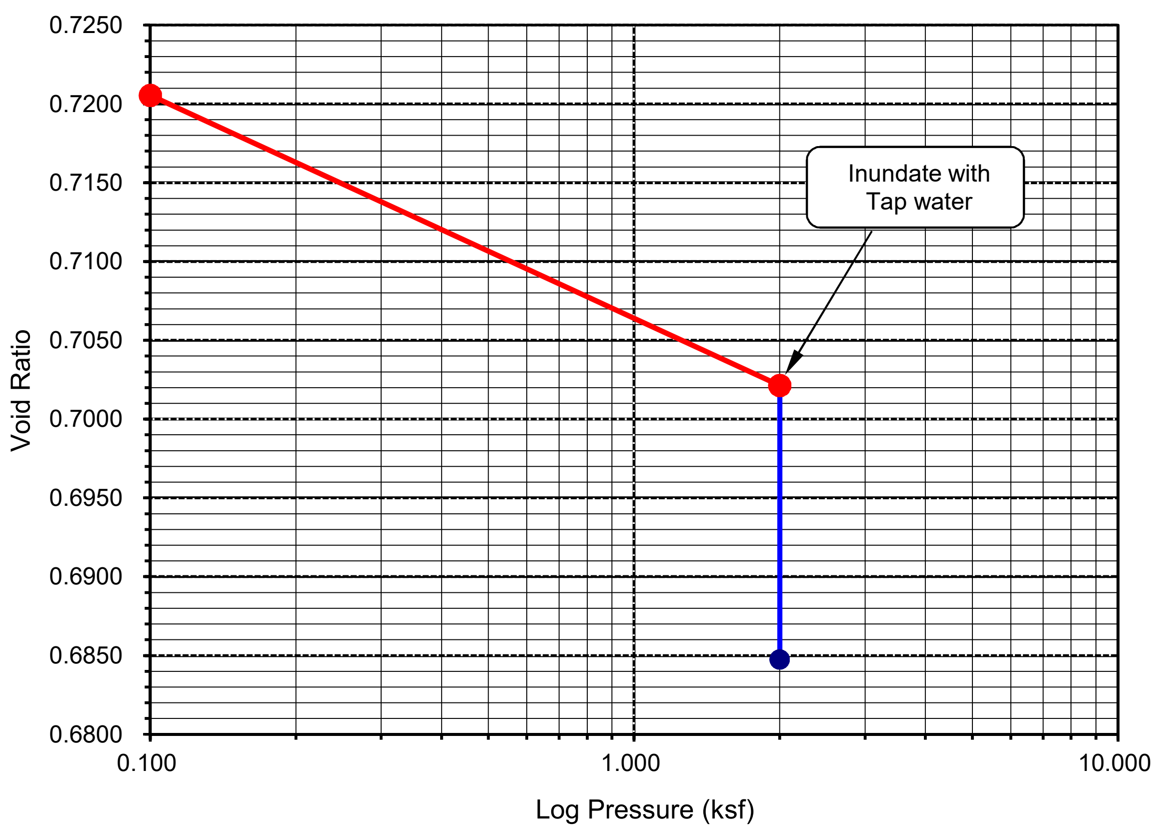
Initial Dry Density (pcf):	98.0
Initial Moisture (%):	5.46
Initial Length (in.):	1.0000
Initial Dial Reading:	0.3313
Diameter(in):	2.415

Final Dry Density (pcf):	100.1
Final Moisture (%) :	23.2
Initial Void Ratio:	0.7207
Specific Gravity(assumed):	2.70
Initial Saturation (%)	20.5

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3312	0.9999	0.00	-0.01	0.7205	-0.01
2.000	0.3158	0.9845	0.47	-1.55	0.7021	-1.08
H2O	0.3057	0.9744	0.47	-2.56	0.6847	-2.09

Percent Swell (+) / Settlement (-) After Inundation = -1.02

Void Ratio - Log Pressure Curve



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

Leighton

Project Name:	HP Hemet	Tested By:	G. Bathala	Date:	04/22/22
Project No.:	13369.003	Checked By:	A. Santos	Date:	05/02/22
Boring No.:	LB-2	Sample Type:	Ring		
Sample No.:	R-5	Depth (ft.)	15.0		
Sample Description:	Olive gray silt (ML)				

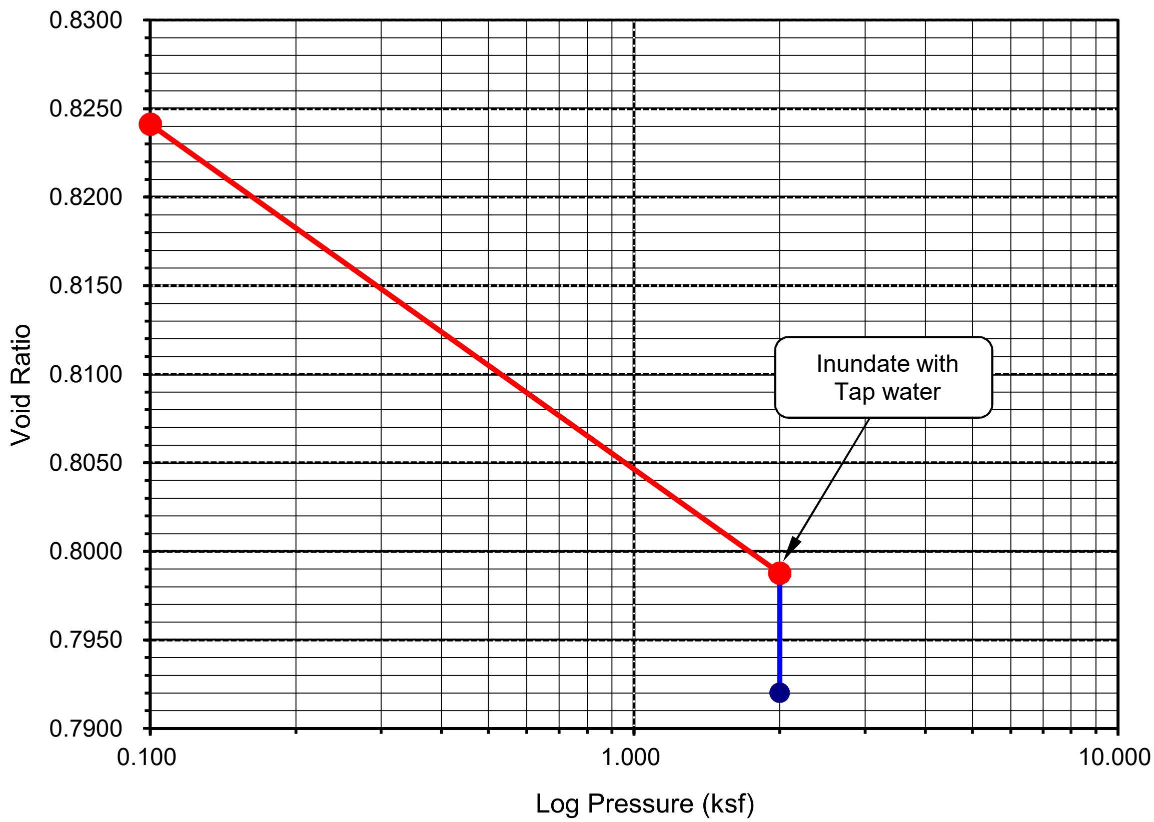
Initial Dry Density (pcf):	92.4
Initial Moisture (%):	11.71
Initial Length (in.):	1.0000
Initial Dial Reading:	0.3114
Diameter(in):	2.415

Final Dry Density (pcf):	94.1
Final Moisture (%) :	28.1
Initial Void Ratio:	0.8241
Specific Gravity(assumed):	2.70
Initial Saturation (%)	38.4

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3114	1.0000	0.00	0.00	0.8241	0.00
2.000	0.2928	0.9814	0.47	-1.86	0.7988	-1.39
H2O	0.2891	0.9777	0.47	-2.23	0.7920	-1.76

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

Void Ratio - Log Pressure Curve



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

Project Name: HP Hemet
Project No.: 13369.003
Boring No.: LB-3
Sample No.: R-5
Sample Description: Olive gray silty clay (CL-ML)

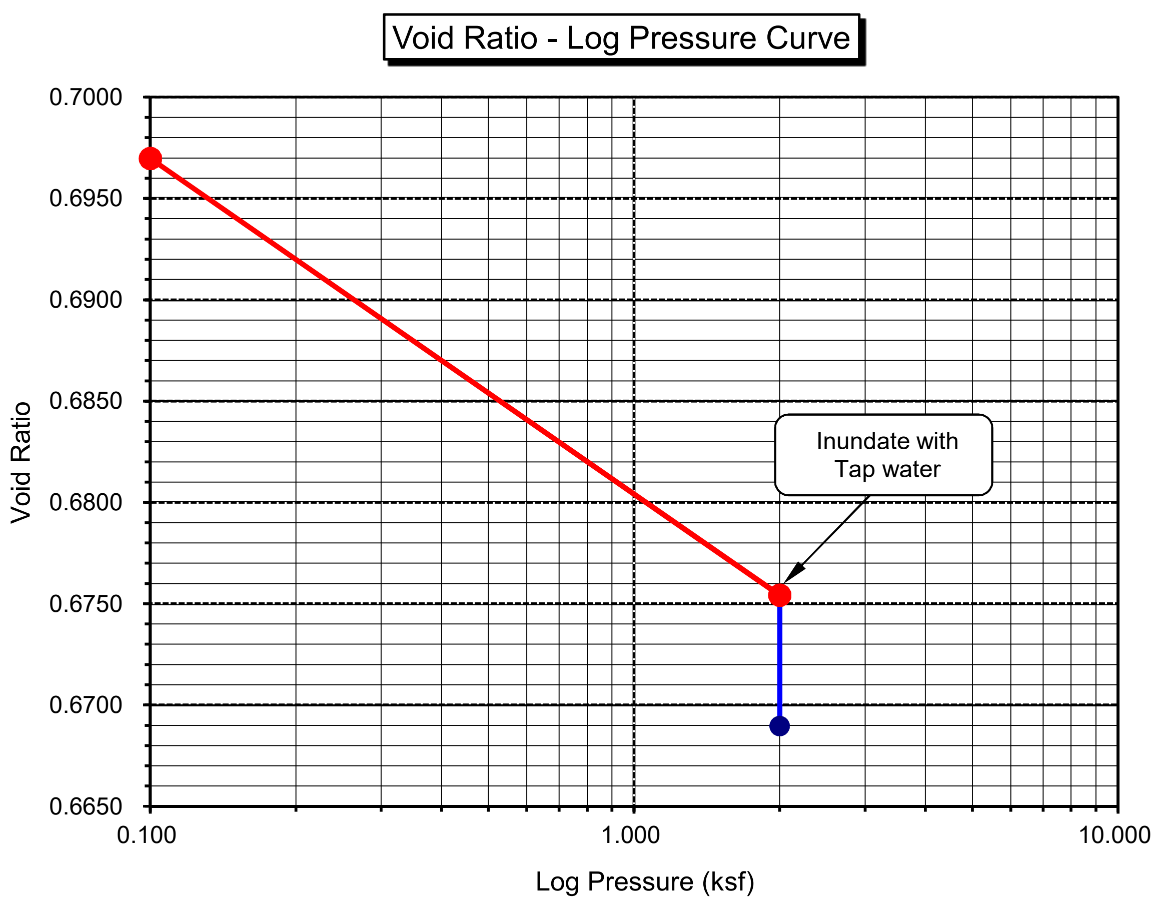
Tested By: G. Bathala Date: 04/22/22
Checked By: A. Santos Date: 05/02/22
Sample Type: Ring
Depth (ft.) 15.0

Initial Dry Density (pcf):	99.3
Initial Moisture (%):	6.51
Initial Length (in.):	1.0000
Initial Dial Reading:	0.3058
Diameter(in):	2.415

Final Dry Density (pcf):	101.0
Final Moisture (%) :	25.5
Initial Void Ratio:	0.6975
Specific Gravity(assumed):	2.70
Initial Saturation (%)	25.2

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3055	0.9997	0.00	-0.03	0.6970	-0.03
2.000	0.2908	0.9850	0.20	-1.50	0.6754	-1.30
H2O	0.2870	0.9812	0.20	-1.88	0.6690	-1.68

Percent Swell (+) / Settlement (-) After Inundation = -0.39



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

Project Name:	HP Hemet	Tested By:	G. Bathala
Date:	04/25/22		
Project No.:	13369.003	Checked By:	A. Santos
Date:	05/02/22		
Boring No.:	LB-7	Sample Type:	Ring
Sample No.:	R-2	Depth (ft.)	10.0
Sample Description:	Olive well-graded sand with silt (SW-SM)		

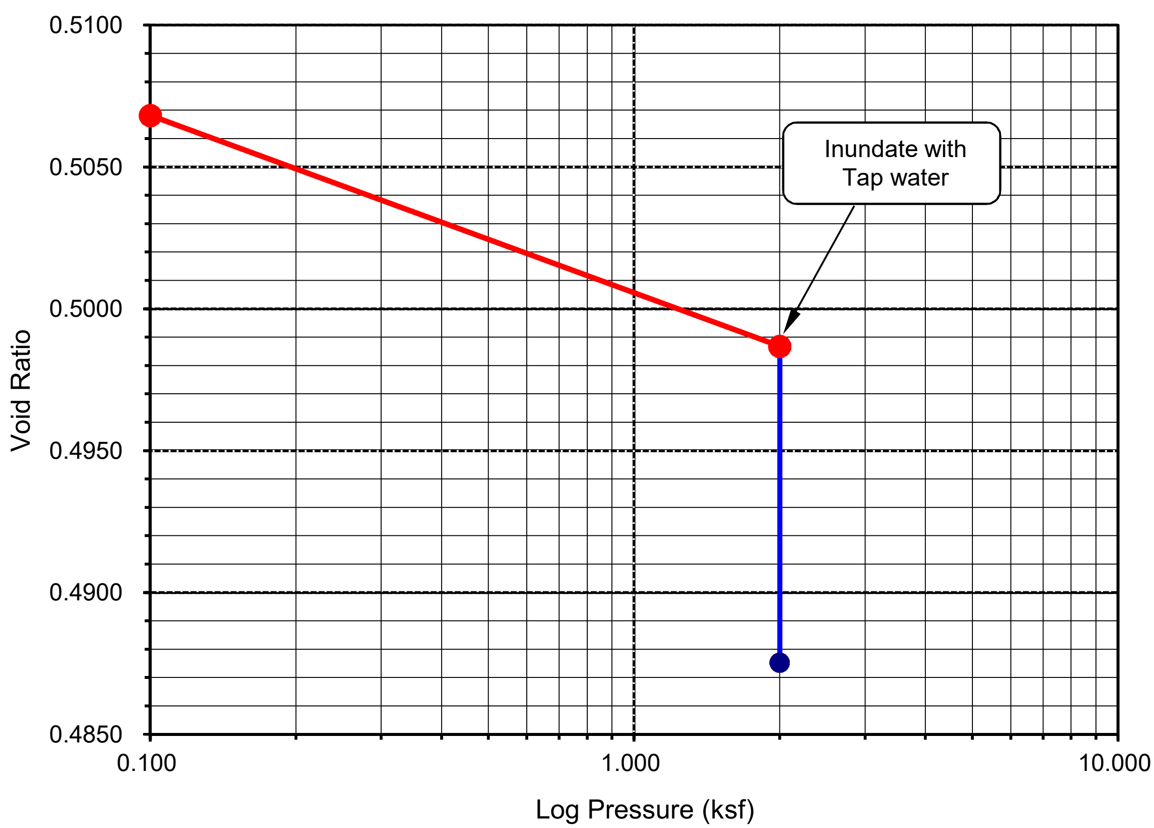
Initial Dry Density (pcf):	111.9
Initial Moisture (%):	1.47
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2809
Diameter(in):	2.415

Final Dry Density (pcf):	113.3
Final Moisture (%) :	14.6
Initial Void Ratio:	0.5070
Specific Gravity(assumed):	2.70
Initial Saturation (%)	7.8

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2808	0.9999	0.00	-0.01	0.5068	-0.01
2.000	0.2711	0.9902	0.43	-0.98	0.4987	-0.55
H2O	0.2637	0.9828	0.43	-1.72	0.4875	-1.29

Percent Swell (+) / Settlement (-) After Inundation = -0.74

Void Ratio - Log Pressure Curve



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

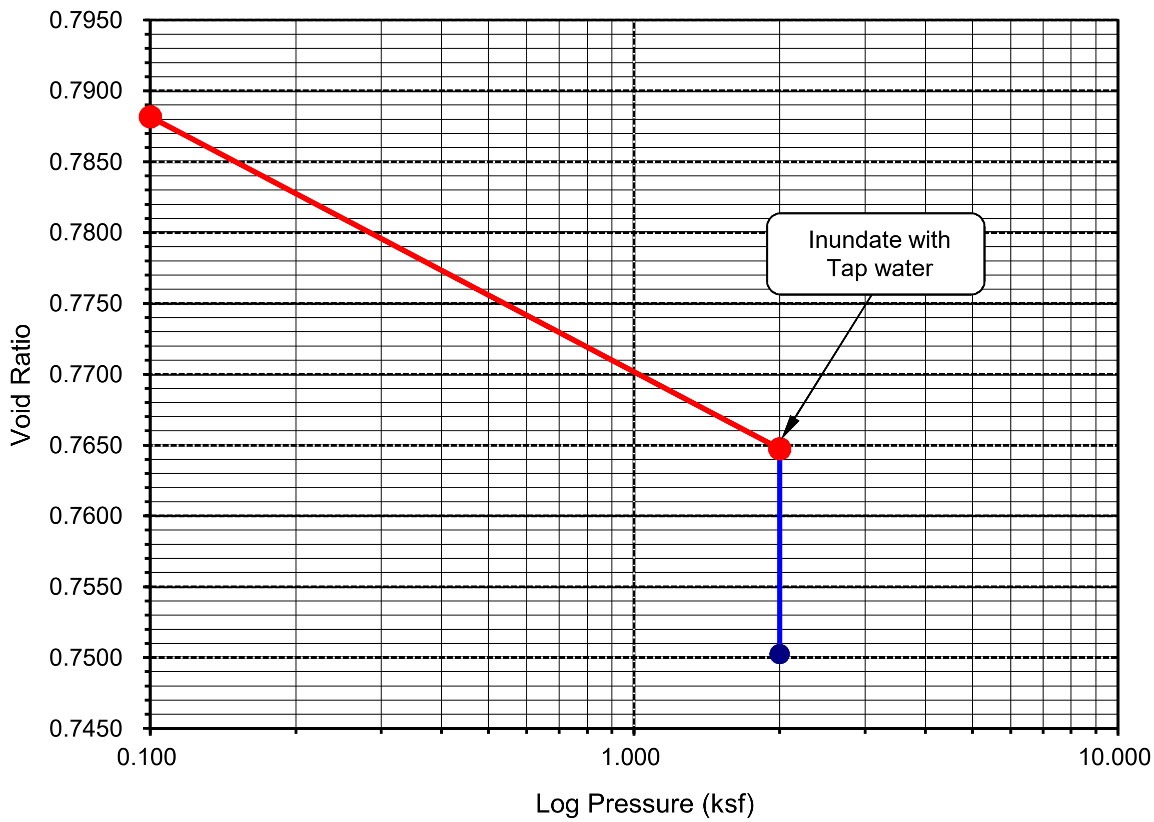
Project Name:	HP Hemet	Tested By:	G. Bathala	Date:	04/25/22
Project No.:	13369.003	Checked By:	A. Santos	Date:	05/02/22
Boring No.:	LB-7	Sample Type:	Ring		
Sample No.:	R-3	Depth (ft.)	15.0		
Sample Description:	Olive gray silty clay (CL-ML)				

Initial Dry Density (pcf):	94.3	Final Dry Density (pcf):	96.3
Initial Moisture (%):	12.71	Final Moisture (%) :	24.7
Initial Length (in.):	1.0000	Initial Void Ratio:	0.7883
Initial Dial Reading:	0.3237	Specific Gravity(assumed):	2.70
Diameter(in):	2.415	Initial Saturation (%)	43.5

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3236	0.9999	0.00	-0.01	0.7882	-0.01
2.000	0.3094	0.9857	0.11	-1.43	0.7647	-1.32
H2O	0.3013	0.9776	0.11	-2.24	0.7503	-2.13

Percent Swell (+) / Settlement (-) After Inundation = -0.82

Void Ratio - Log Pressure Curve



APPENDIX C

FIELD PERCOLATION TEST RESULTS

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Boring Percolation Test Data Sheet

Project Number:	13369.003	**Test Hole Number:**	LP-1
Project Name:	HP River Oaks Ranch	**Date Excavated:**	4/12/2022
Earth Description:	Alluvium	**Date Tested:**	4/15/2022
Liquid Description:	Tap water	**Depth of boring (ft):**	4.5
Tested By:	LFO	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	9:03 AM	**Length of slotted of casing (ft):**	5
Start Time for Standard:	9:03 AM	**Depth to Initial Water Depth (ft):**	1
Standard Time Interval	25 mins	**Porosity of Annulus Material, *n* :**	0.35
Between Readings, mins:	10	**Bentonite Plug at Bottom:**	No

Field Percolation Data - Falling Head Test

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
P1	9:03	25	0.62	46.6	13.6	0.61
	9:28		1.75	33.0		
P2	9:30	25	0.63	46.4	13.0	0.58
	9:55		1.71	33.5		
1	10:00	10	0.62	46.6	7.9	0.83
	10:10		1.28	38.6		
2	10:12	10	0.68	45.8	6.5	0.68
	10:22		1.22	39.4		
3	10:25	10	0.66	46.1	6.4	0.66
	10:35		1.19	39.7		
4	10:37	10	0.65	46.2	6.0	0.62
	10:47		1.15	40.2		
5	10:49	10	0.62	46.6	6.0	0.62
	10:59		1.12	40.6		
6	11:01	10	0.62	46.6	5.8	0.59
	11:11		1.10	40.8		

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Measured Infiltration Rate, I (Average of Last 3 Readings) = $\boxed{\textbf{0.61}}$ **in./hr.**

Boring Percolation Test Data Sheet

Project Number:	13369.003	**Test Hole Number:**	LP-2
Project Name:	HP River Oaks Ranch	**Date Excavated:**	4/12/2022
Earth Description:	Alluvium	**Date Tested:**	4/15/2022
Liquid Description:	Tap water	**Depth of boring (ft):**	4.375
Tested By:	LFO	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	8:53 AM	**Length of slotted of casing (ft):**	5
Start Time for Standard:	8:53 AM	**Depth to Initial Water Depth (ft):**	2
Standard Time Interval	25 mins	**Porosity of Annulus Material, *n* :**	0.35
Between Readings, mins:	10	**Bentonite Plug at Bottom:**	No

Field Percolation Data - Falling Head Test

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
P1	8:53	25	2.00	28.6	9.6	0.70
	9:18		2.80	19.0		
P2	9:20	25	2.00	28.5	9.4	0.68
	9:45		2.78	19.1		
1	11:29	10	2.00	28.6	5.8	0.98
	11:39		2.48	22.8		
2	11:41	10	2.00	28.6	5.3	0.89
	11:51		2.44	23.3		
3	11:53	10	1.97	28.9	4.4	0.73
	12:03		2.34	24.5		
4	12:05	10	1.96	29.0	4.4	0.72
	12:15		2.33	24.6		
5	12:17	10	1.93	29.4	4.0	0.63
	12:27		2.26	25.4		
6	12:29	10	2.00	28.6	4.0	0.65
	12:39		2.33	24.6		

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Measured Infiltration Rate, I (Average of Last 3 Readings) = $\boxed{0.67}$ in./hr.

APPENDIX D

SEISMIC DESIGN DATA AND ANALYSIS

Leighton
a verdantas company

River Oaks Ranch

Latitude, Longitude: 33.72857, -116.98421

Date	5/4/2022, 9:29:06 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.844	MCE_R ground motion. (for 0.2 second period)
S_1	0.725	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.844	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.23	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	0.779	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.857	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.912	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.123	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.844	Factored deterministic acceleration value. (0.2 second)
S1RT	0.755	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.853	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.725	Factored deterministic acceleration value. (1.0 second)
PGAd	0.779	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.901	Mapped value of the risk coefficient at short periods
C_{R1}	0.885	Mapped value of the risk coefficient at a period of 1 s

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. 2008 (v3.3.3)

Latitude

Decimal degrees

33.72857

Longitude

Decimal degrees, negative values for western longitudes

-116.98421

Site Class

259 m/s (Site class D)

Spectral Period

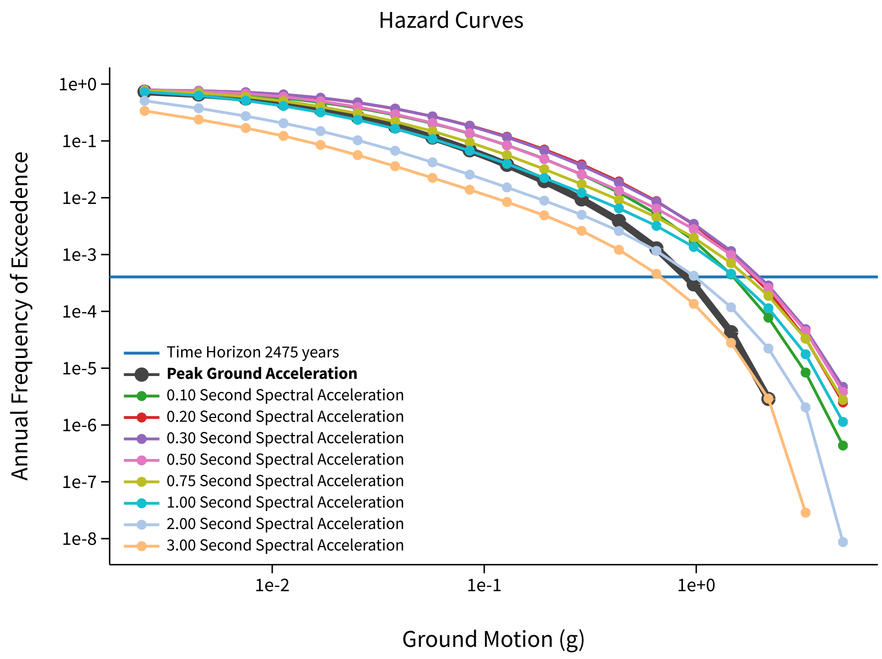
Peak Ground Acceleration

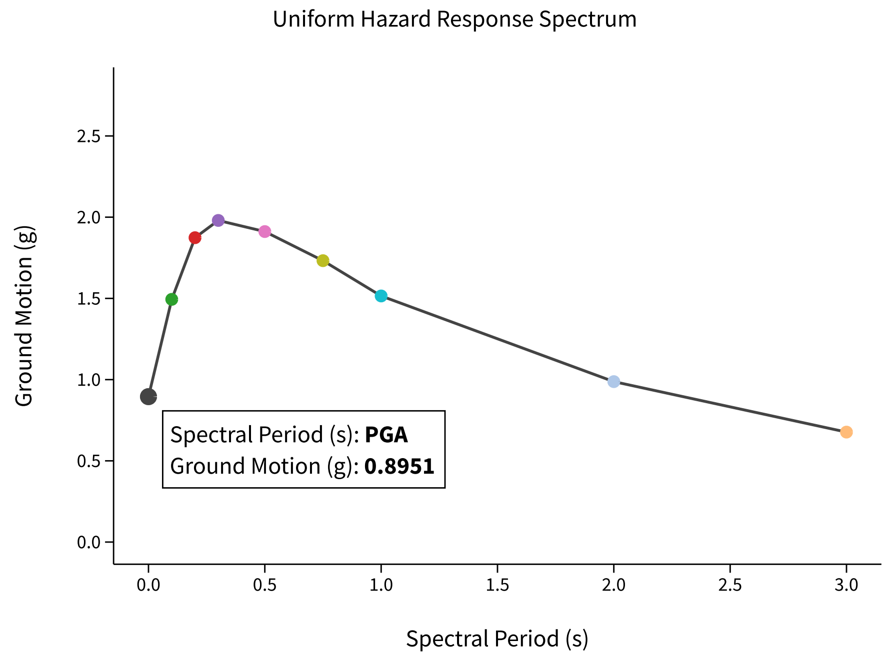
Time Horizon

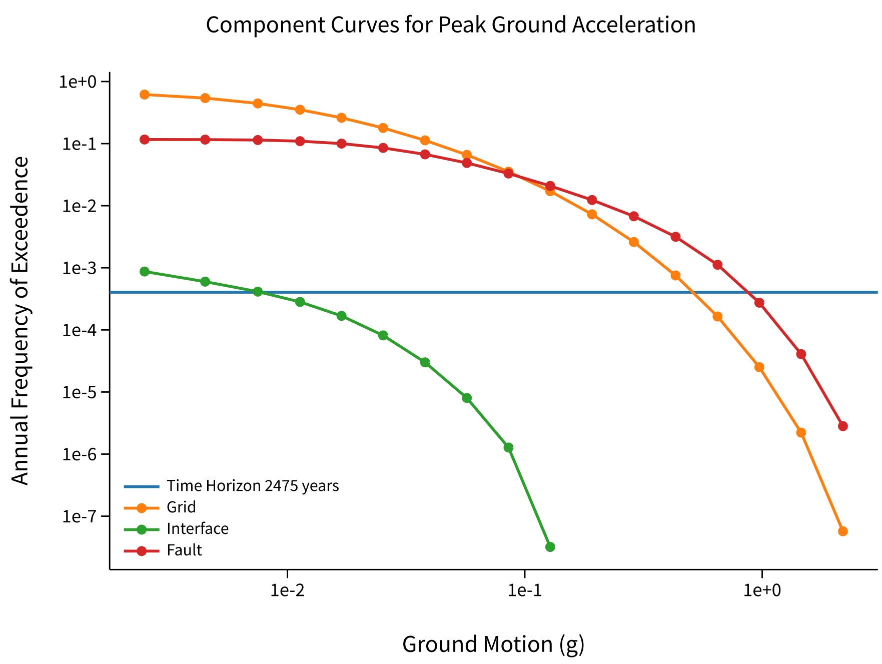
Return period in years

2475

Hazard Curve





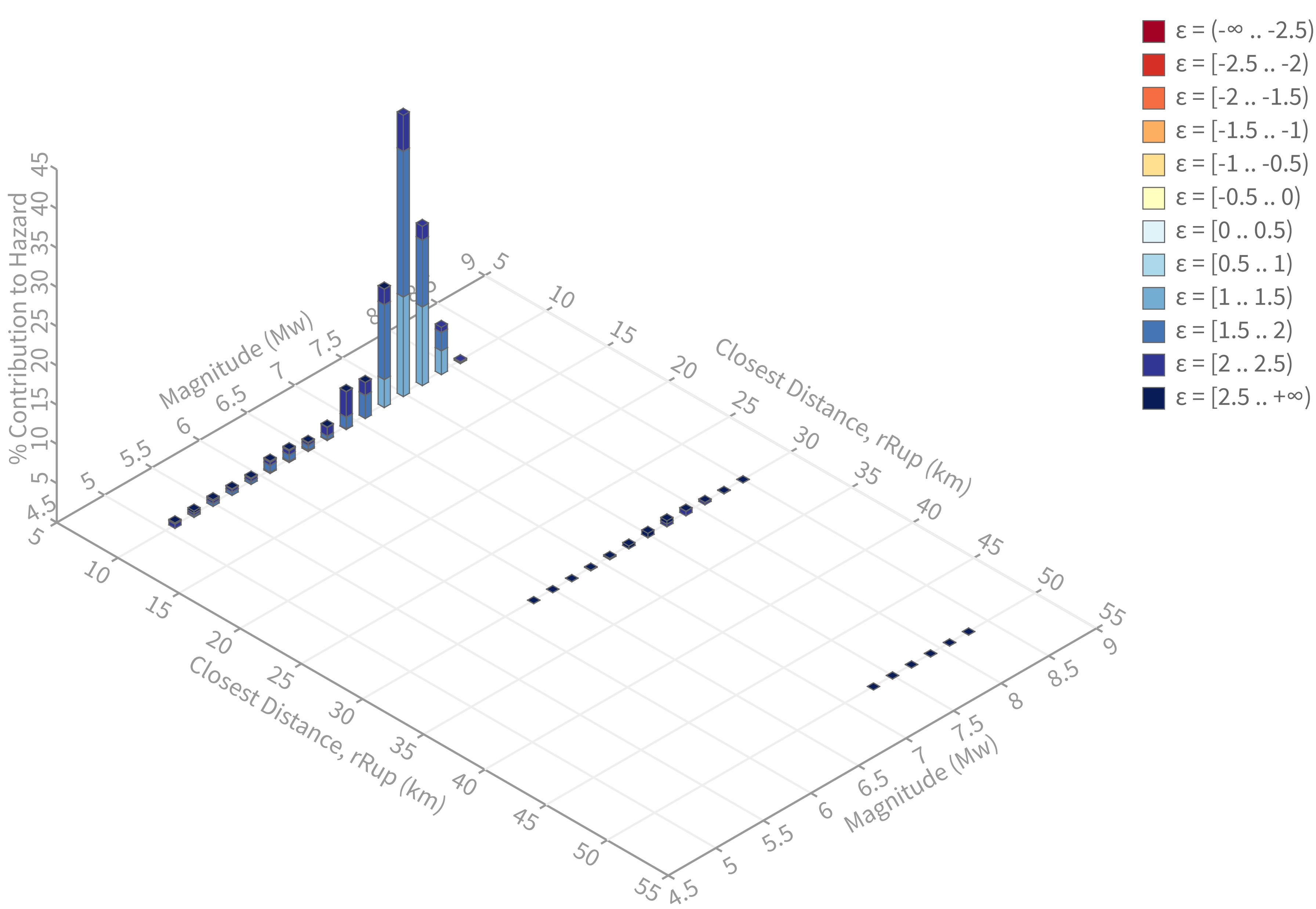


View Raw Data

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.8951321 g

Recovered targets

Return period: 3456.384 yrs
Exceedance rate: 0.0002893197 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 7.34
r: 6.25 km
ε_0: 1.71 σ

Mode (largest m-r bin)

m: 7.5
r: 4.66 km
ε_0: 1.58 σ
Contribution: 35.87 %

Mode (largest m-r-ε_0 bin)

m: 7.5
r: 4.97 km
ε_0: 1.58 σ
Contribution: 18.58 %

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	%
aFault_aPriori_D2.1	Fault							48.55
San Jacinto : A+C		4.21	7.49	1.54	116.962°W	33.762°N	29.13	28.02
San Jacinto : SBV+SJV+A+C		5.46	7.76	1.58	116.914°W	33.736°N	82.28	8.38
San Jacinto : SJV		7.99	6.96	2.08	116.903°W	33.756°N	68.11	6.11
San Jacinto : SBV+SJV		7.99	7.32	1.94	116.903°W	33.756°N	68.11	4.62
aFault_MoBal	Fault							33.50
San Jacinto : A+C		4.21	7.49	1.54	116.962°W	33.762°N	29.13	10.40
San Jacinto : SBV+SJV+A+C		5.46	7.76	1.58	116.914°W	33.736°N	82.28	3.31
San Jacinto : SJV+A+C		5.46	7.64	1.62	116.914°W	33.736°N	82.28	3.09
San Jacinto : SBV+SJV+A		5.46	7.63	1.62	116.914°W	33.736°N	82.28	3.07
San Jacinto : SJV+A		5.46	7.46	1.67	116.914°W	33.736°N	82.28	2.78
San Jacinto : SBV+SJV		7.99	7.32	1.95	116.903°W	33.756°N	68.11	1.54
San Jacinto : SJV		7.99	6.96	2.08	116.903°W	33.756°N	68.11	1.44
aFault_unseg	Fault							8.97
San Jacinto (SB to C)		5.75	7.33	1.74	116.914°W	33.736°N	82.28	8.57
CAmap.21.ch.in (opt)	Grid							3.08
PointSourceFinite: -116.984, 33.760		6.15	5.82	1.85	116.984°W	33.760°N	0.00	1.58
CAmap.24.ch.in (opt)	Grid							3.08
PointSourceFinite: -116.984, 33.760		6.15	5.82	1.85	116.984°W	33.760°N	0.00	1.58
CAmap.24.gr.in (opt)	Grid							1.37
CAmap.21.gr.in (opt)	Grid							1.37

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Schmertmann Method

Date April 11, 2024
Identification Highpointe 13369.003

Input

Units	E	E or SI
Shape	SQ	SQ, CI, CO, or RE
B =	6	ft
L =	6	ft
D =	1.5	ft
P =	90	k
Dw =	20	ft
gamma =	120	lb/ft^3
t =	10	yr

Results

q =	2725	lb/ft^2
delta =	0.80	in

Depth to Soil Layer Top (ft)	Bottom (ft)	Es (lb/ft^2)	zf (ft)	I epsilon	strain (%)	delta (in)
0.0	1.5					
1.5	2.5	250000	0.5	0.203	0.2789	0.0335
2.5	3.5	250000	1.5	0.409	0.5617	0.0674
3.5	4.5	250000	2.5	0.614	0.8445	0.1013
4.5	5.5	250000	3.5	0.678	0.9316	0.1118
5.5	6.5	250000	4.5	0.598	0.8220	0.0986
6.5	7.5	250000	5.5	0.518	0.7124	0.0855
7.5	8.5	250000	6.5	0.438	0.6028	0.0723
8.5	9.5	250000	7.5	0.359	0.4932	0.0592
9.5	10.5	160000	8.5	0.279	0.5993	0.0719
10.5	11.5	150000	9.5	0.199	0.4566	0.0548
11.5	12.5	150000	10.5	0.120	0.2740	0.0329
12.5	13.5	150000	11.5	0.040	0.0913	0.0110
13.5	14.5	90500	12.5	0.000	0.0000	0.0000
14.5	15.5	95000	13.5	0.000	0.0000	0.0000
15.5	16.5	95000	14.5	0.000	0.0000	0.0000
16.5	17.5	95000	15.5	0.000	0.0000	0.0000
17.5	18.5	95000	16.5	0.000	0.0000	0.0000
18.5	19.5	95000	17.5	0.000	0.0000	0.0000
19.5	20.5	95000	18.5	0.000	0.0000	0.0000
20.5	21.5	95000	19.5	0.000	0.0000	0.0000
21.5	22.5	95000	20.5	0.000	0.0000	0.0000
22.5	23.5	95000	21.5	0.000	0.0000	0.0000
23.5	24.5	95000	22.5	0.000	0.0000	0.0000
24.5	25.5	95000	23.5	0.000	0.0000	0.0000
25.5	26.5	95000	24.5	0.000	0.0000	0.0000
26.5	27.5	95000	25.5	0.000	0.0000	0.0000
27.5	28.5	176100	26.5	0.000	0.0000	0.0000
28.5	29.5	176100	27.5	0.000	0.0000	0.0000
29.5	30.5	176100	28.5	0.000	0.0000	0.0000
30.5	31.5	176100	29.5	0.000	0.0000	0.0000
31.5	32.5	176100	30.5	0.000	0.0000	0.0000
32.5	33.5	176100	31.5	0.000	0.0000	0.0000
33.5	34.5	176100	32.5	0.000	0.0000	0.0000
34.5	35.5	176100	33.5	0.000	0.0000	0.0000
35.5	36.5	176100	34.5	0.000	0.0000	0.0000
36.5	37.5	176100	35.5	0.000	0.0000	0.0000
37.5	38.5	176100	36.5	0.000	0.0000	0.0000
38.5	39.5	176100	37.5	0.000	0.0000	0.0000
39.5	40.5	176100	38.5	0.000	0.0000	0.0000

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Schmertmann Method

Date	April 11, 2024
Identification	Highpointe 13369.003

Input

Units	E	E or SI
Shape	CO	SQ, CI, CO, or RE
B =	3	ft
L =	30	ft
D =	1.5	ft
P =	7.5	k/ft
Dw =	20	ft
gamma =	120	lb/ft^3
t =	10	yr

Results

q =	2725	lb/ft^2
delta =	1.02	in

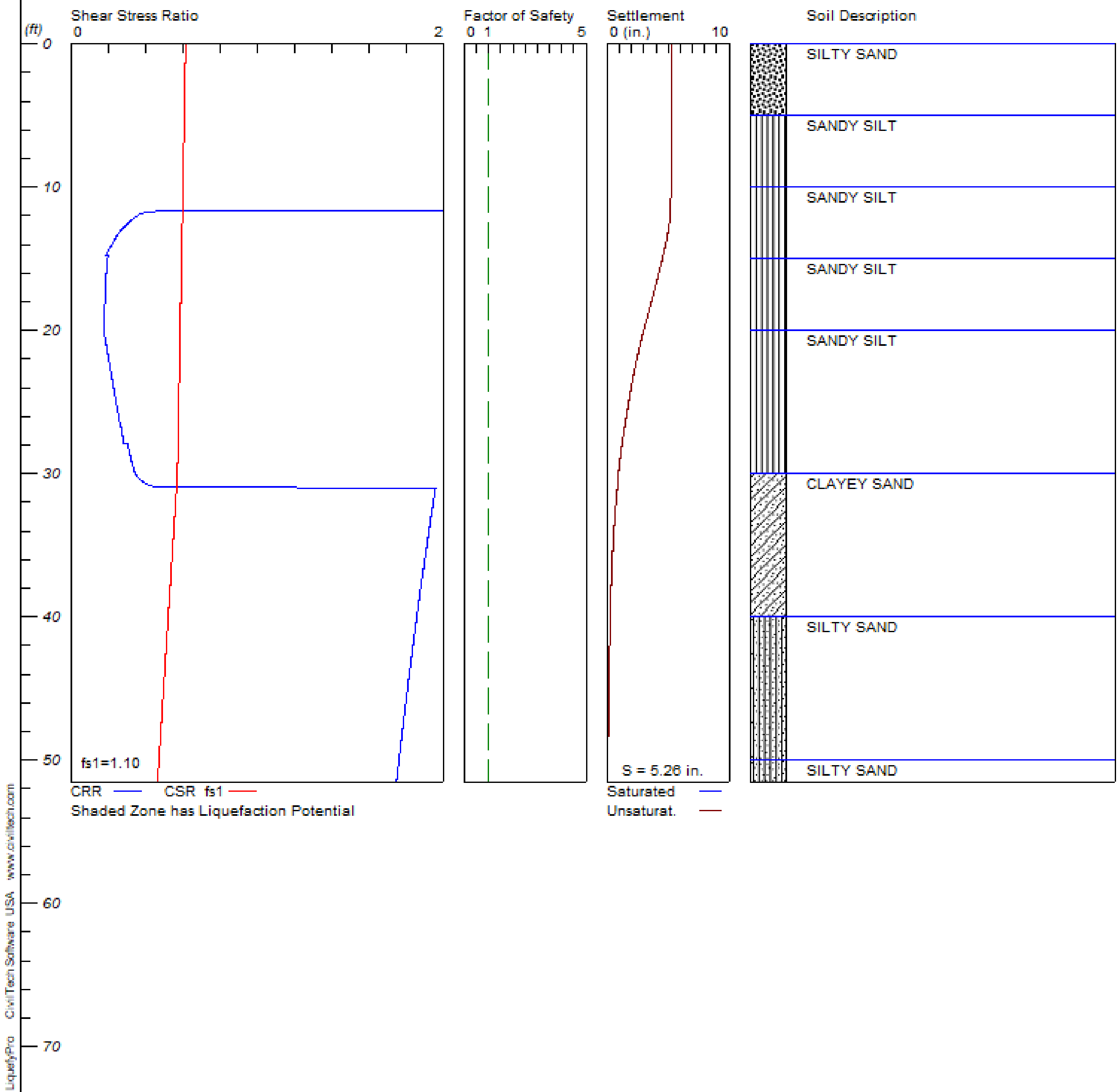
Depth to Soil Layer Top (ft)	Bottom (ft)	Es (lb/ft^2)	zf (ft)	I epsilon	strain (%)	delta (in)
0.0	1.5					
1.5	2.5	250000	0.5	0.375	0.3767	0.0452
2.5	3.5	250000	1.5	0.726	0.7286	0.0874
3.5	4.5	250000	2.5	1.077	1.0806	0.1297
4.5	5.5	250000	3.5	1.181	1.1856	0.1423
5.5	6.5	250000	4.5	1.042	1.0461	0.1255
6.5	7.5	250000	5.5	0.903	0.9066	0.1088
7.5	8.5	250000	6.5	0.764	0.7671	0.0921
8.5	9.5	250000	7.5	0.625	0.6276	0.0753
9.5	10.5	160000	8.5	0.486	0.7628	0.0915
10.5	11.5	150000	9.5	0.347	0.5812	0.0697
11.5	12.5	150000	10.5	0.208	0.3487	0.0418
12.5	13.5	150000	11.5	0.069	0.1162	0.0139
13.5	14.5	90500	12.5	0.000	0.0000	0.0000
14.5	15.5	95000	13.5	0.000	0.0000	0.0000
15.5	16.5	95000	14.5	0.000	0.0000	0.0000
16.5	17.5	95000	15.5	0.000	0.0000	0.0000
17.5	18.5	95000	16.5	0.000	0.0000	0.0000
18.5	19.5	95000	17.5	0.000	0.0000	0.0000
19.5	20.5	95000	18.5	0.000	0.0000	0.0000
20.5	21.5	95000	19.5	0.000	0.0000	0.0000
21.5	22.5	95000	20.5	0.000	0.0000	0.0000
22.5	23.5	95000	21.5	0.000	0.0000	0.0000
23.5	24.5	95000	22.5	0.000	0.0000	0.0000
24.5	25.5	95000	23.5	0.000	0.0000	0.0000
25.5	26.5	95000	24.5	0.000	0.0000	0.0000
26.5	27.5	95000	25.5	0.000	0.0000	0.0000
27.5	28.5	176100	26.5	0.000	0.0000	0.0000
28.5	29.5	176100	27.5	0.000	0.0000	0.0000
29.5	30.5	176100	28.5	0.000	0.0000	0.0000
30.5	31.5	176100	29.5	0.000	0.0000	0.0000
31.5	32.5	176100	30.5	0.000	0.0000	0.0000
32.5	33.5	176100	31.5	0.000	0.0000	0.0000
33.5	34.5	176100	32.5	0.000	0.0000	0.0000
34.5	35.5	176100	33.5	0.000	0.0000	0.0000
35.5	36.5	176100	34.5	0.000	0.0000	0.0000
36.5	37.5	176100	35.5	0.000	0.0000	0.0000
37.5	38.5	176100	36.5	0.000	0.0000	0.0000
38.5	39.5	176100	37.5	0.000	0.0000	0.0000
39.5	40.5	176100	38.5	0.000	0.0000	0.0000

LIQUEFACTION ANALYSIS

Highpointe River Oaks Ranch

Hole No.=LB-1   Water Depth=100 ft

Magnitude=7.5
Acceleration=0.86g



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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: P:\INFOCUS PROJECTS\13001-13500\13369 Highpointe Comm - Curtis Park\002 Geo\Analyses\Dynamic Settlement\LB-1.liq
Title: Highpointe River Oaks Ranch
Subtitle: 13369.003

Surface Elev.=
Hole No.=LB-1
Depth of Hole= 51.50 ft
Water Table during Earthquake= 100.00 ft
Water Table during In-Situ Testing= 100.00 ft
Max. Acceleration= 0.86 g
Earthquake Magnitude= 7.50

Input Data:
Surface Elev.=
Hole No.=LB-1
Depth of Hole=51.50 ft
Water Table during Earthquake= 100.00 ft
Water Table during In-Situ Testing= 100.00 ft
Max. Acceleration=0.86 g
Earthquake Magnitude=7.50
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1.1
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	20.00	127.00	NoLiq
9.00	20.00	120.00	NoLiq
10.00	20.00	120.00	60.00
15.00	8.40	120.00	60.00
20.00	8.40	120.00	60.00
30.00	20.40	129.00	NoLiq
40.00	41.40	127.00	40.00
50.00	44.40	127.00	40.00

Output Results:

Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=5.26 in.
Total Settlement of Saturated and Unsaturated Sands=5.26 in.
Differential Settlement=2.628 to 3.469 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.61	5.00	0.00	5.26	5.26
5.00	2.00	0.61	5.00	0.00	5.26	5.26
10.00	2.00	0.60	5.00	0.00	5.23	5.23
15.00	0.19	0.59	5.00	0.00	4.52	4.52
20.00	0.17	0.59	5.00	0.00	2.97	2.97
25.00	0.24	0.58	5.00	0.00	1.75	1.75
30.00	0.34	0.57	5.00	0.00	0.91	0.91
35.00	1.91	0.55	5.00	0.00	0.45	0.45
40.00	1.86	0.52	5.00	0.00	0.22	0.22
45.00	1.81	0.50	5.00	0.00	0.13	0.13
50.00	1.76	0.47	5.00	0.00	0.03	0.03

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

---

1 atm (atmosphere) = 1 tsf (ton/ft2)

CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands

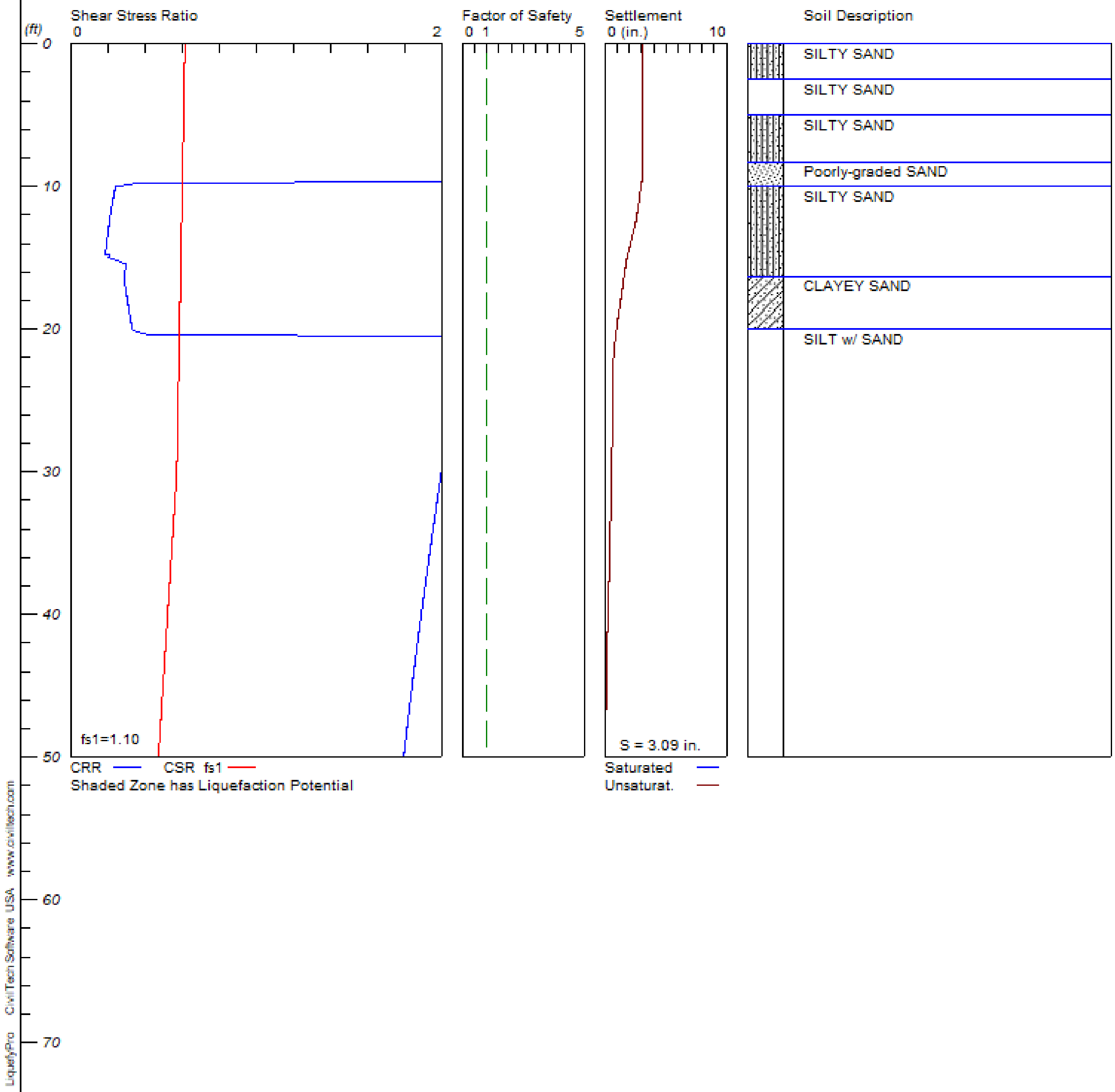
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

Highpointe River Oaks Ranch

Hole No.=LB-3   Water Depth=100 ft

Magnitude=7.5
Acceleration=0.86g



Leighton 13369.003 **Plate A-1**

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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: P:\INFOCUS PROJECTS\13001-13500\13369 Highpointe Comm - Curtis Park\002 Geo\Analyses\Dynamic Settlement\LB-3.liq
Title: Highpointe River Oaks Ranch
Subtitle: 13369.003

Surface Elev.=
Hole No.=LB-3
Depth of Hole= 50.00 ft
Water Table during Earthquake= 100.00 ft
Water Table during In-Situ Testing= 100.00 ft
Max. Acceleration= 0.86 g
Earthquake Magnitude= 7.50

Input Data:
Surface Elev.=
Hole No.=LB-3
Depth of Hole=50.00 ft
Water Table during Earthquake= 100.00 ft
Water Table during In-Situ Testing= 100.00 ft
Max. Acceleration=0.86 g
Earthquake Magnitude=7.50
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio,   Ce = 1.25
7. Borehole Diameter,   Cb= 1
8. Sampling Method,   Cs= 1
9. User request factor of safety (apply to CSR) ,   User= 1.1
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	8.00	103.10	NoLiq
5.00	17.00	111.70	NoLiq
7.50	14.00	111.70	NoLiq
8.00	14.00	111.70	NoLiq
8.50	14.00	111.70	NoLiq
9.00	14.00	105.60	NoLiq
10.00	14.00	105.60	10.00
15.00	13.00	106.60	10.00
16.50	13.00	106.60	NoLiq
20.00	16.00	119.80	60.00
25.00	40.00	119.80	60.00

Output Results:

Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=3.09 in.
Total Settlement of Saturated and Unsaturated Sands=3.09 in.
Differential Settlement=1.544 to 2.039 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.61	5.00	0.00	3.09	3.09
5.00	2.00	0.61	5.00	0.00	3.09	3.09
10.00	0.24	0.60	5.00	0.00	2.95	2.95
15.00	0.20	0.59	5.00	0.00	1.80	1.80
20.00	0.33	0.59	5.00	0.00	0.92	0.92
25.00	2.00	0.58	5.00	0.00	0.61	0.61
30.00	2.00	0.57	5.00	0.00	0.51	0.51
35.00	1.94	0.55	5.00	0.00	0.39	0.39
40.00	1.89	0.52	5.00	0.00	0.23	0.23
45.00	1.84	0.50	5.00	0.00	0.09	0.09
50.00	1.80	0.47	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

_

1 atm (atmosphere) = 1 tsf (ton/ft2)
CRRm Cyclic resistance ratio from soils
CSRsf Cyclic stress ratio induced by a given earthquake (with user request factor of safety)

F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

A P P E N D I X E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

APPENDIX E
LEIGHTON AND ASSOCIATES, INC.
GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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Section **Appendix J Page**

Attachments - Standard Details

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 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

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).

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). 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.

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 OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed 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 relative compaction 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.

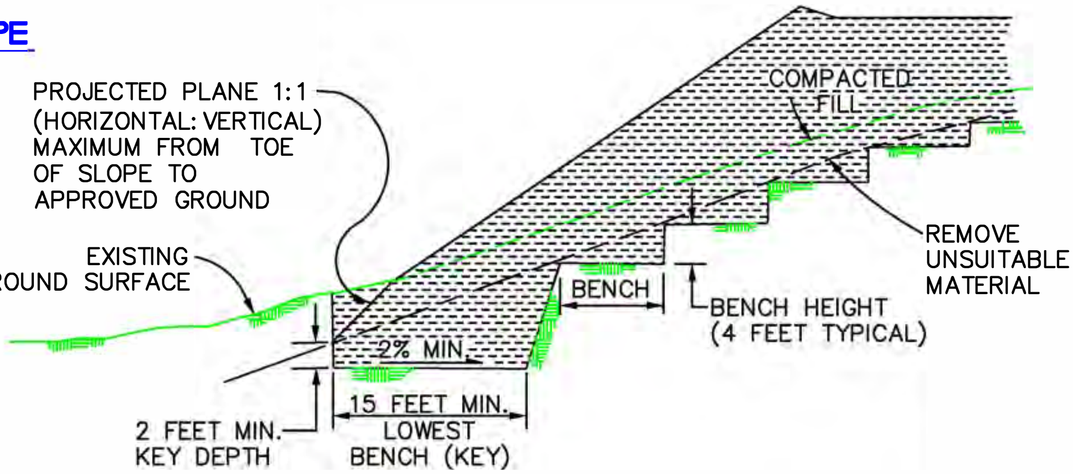
7.3 Lift Thickness

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.

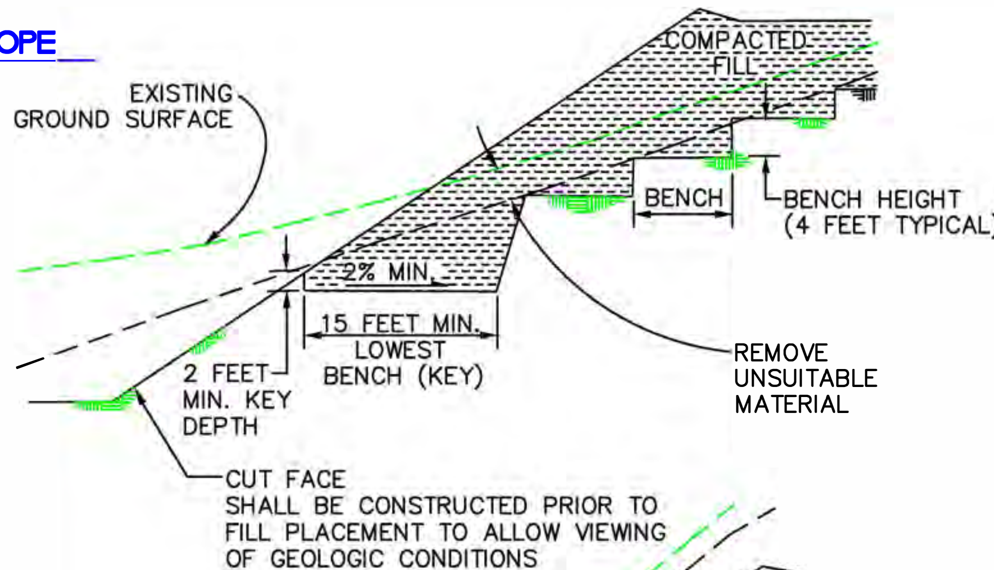
7.4 Observation and Testing

The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

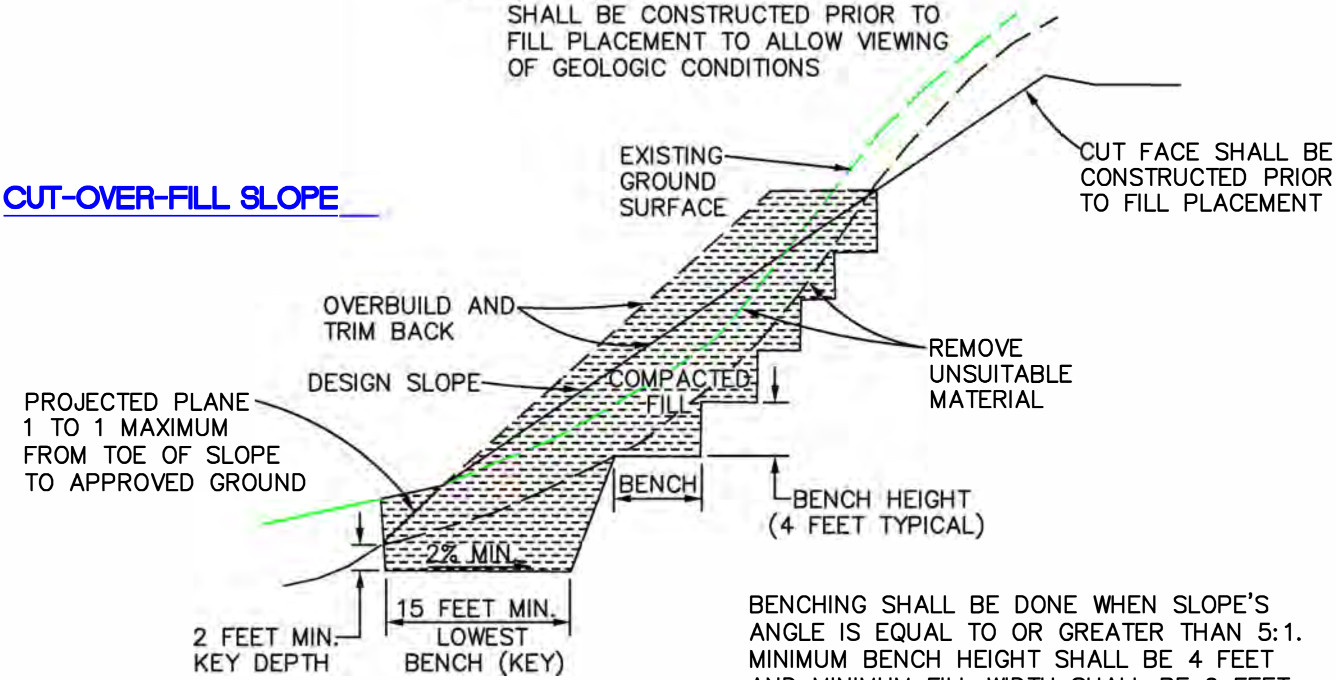
FILL SLOPE



FILL-OVER-CUT SLOPE

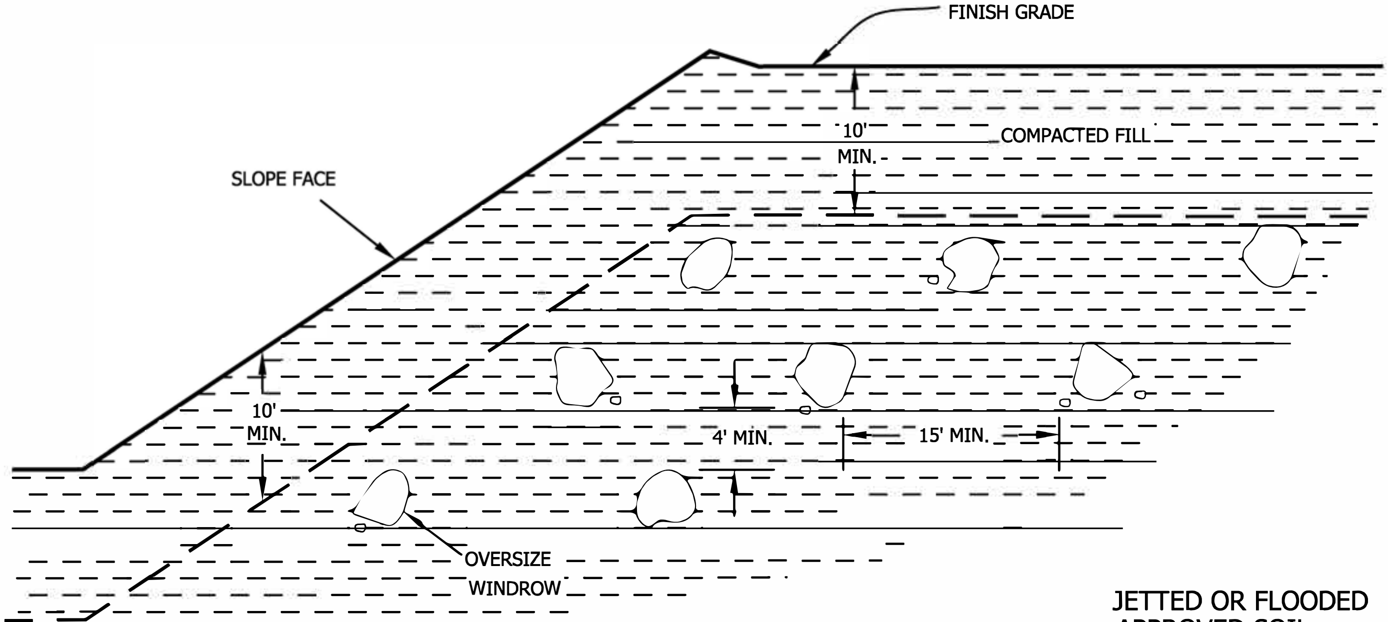


CUT-OVER-FILL SLOPE

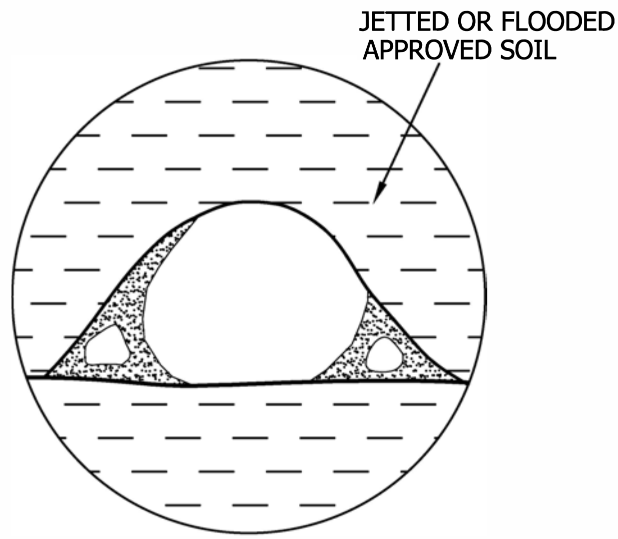


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton

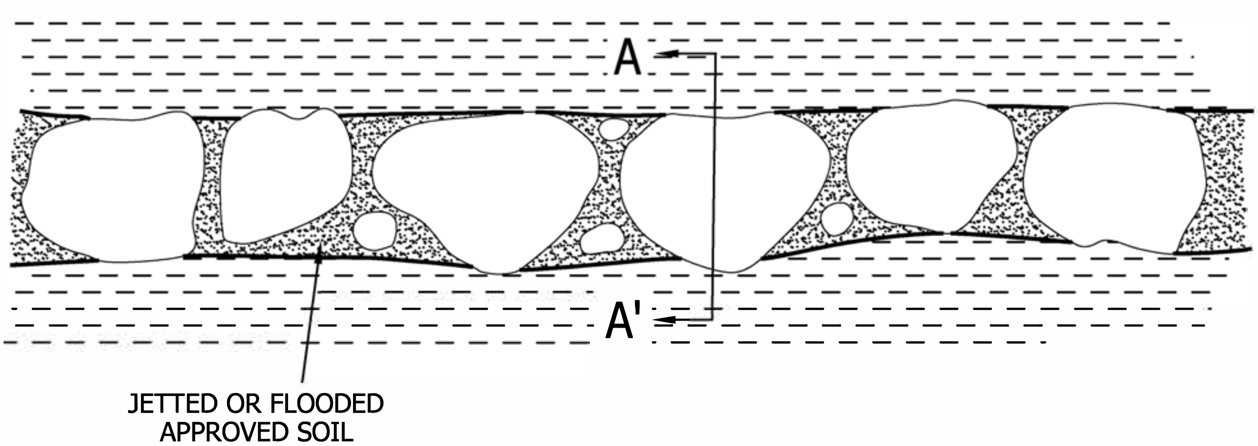


- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.



SECTION A-A'

PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B	Leighton

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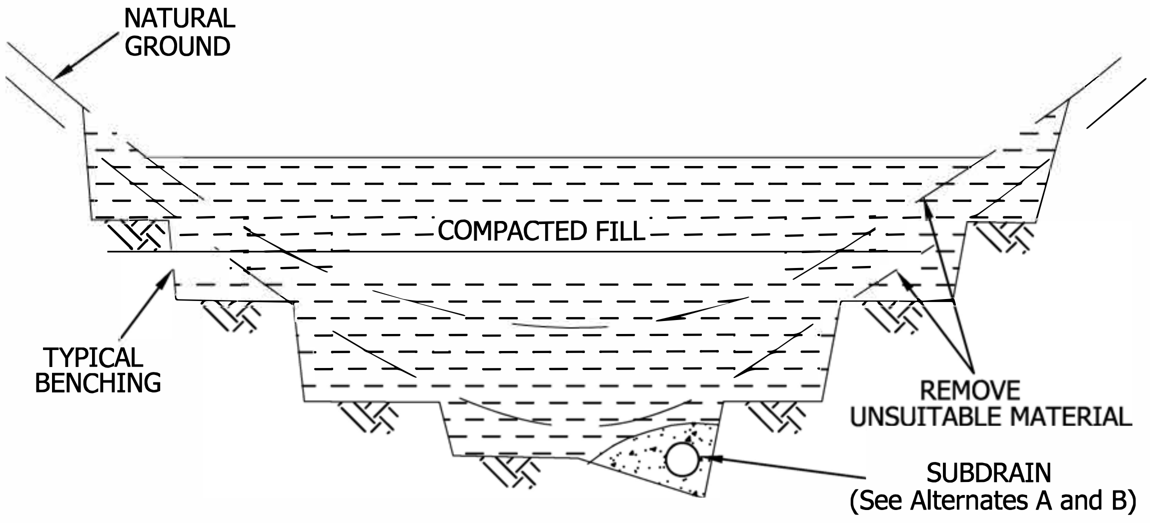
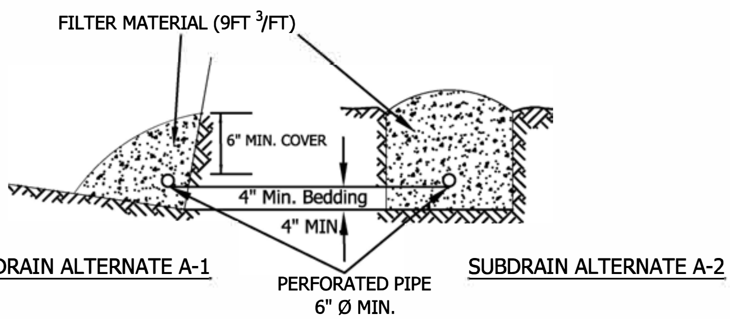


SUBDRAIN ALTERNATE A

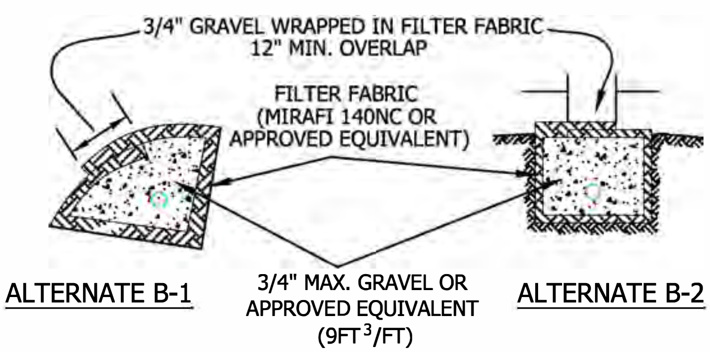


FILTER MATERIAL

FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE. CLASS 2 GRADING AS FOLLOWS:

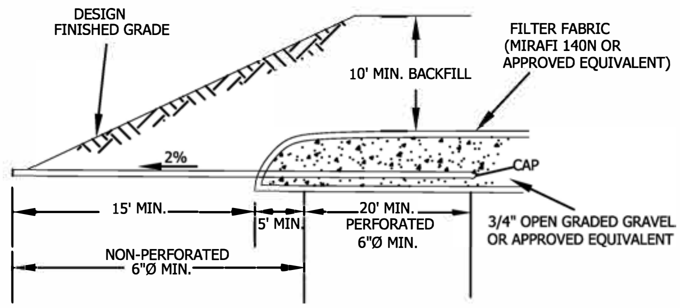
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B



○ PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

DETAIL OF CANYON SUBDRAIN TERMINAL



CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	Leighton

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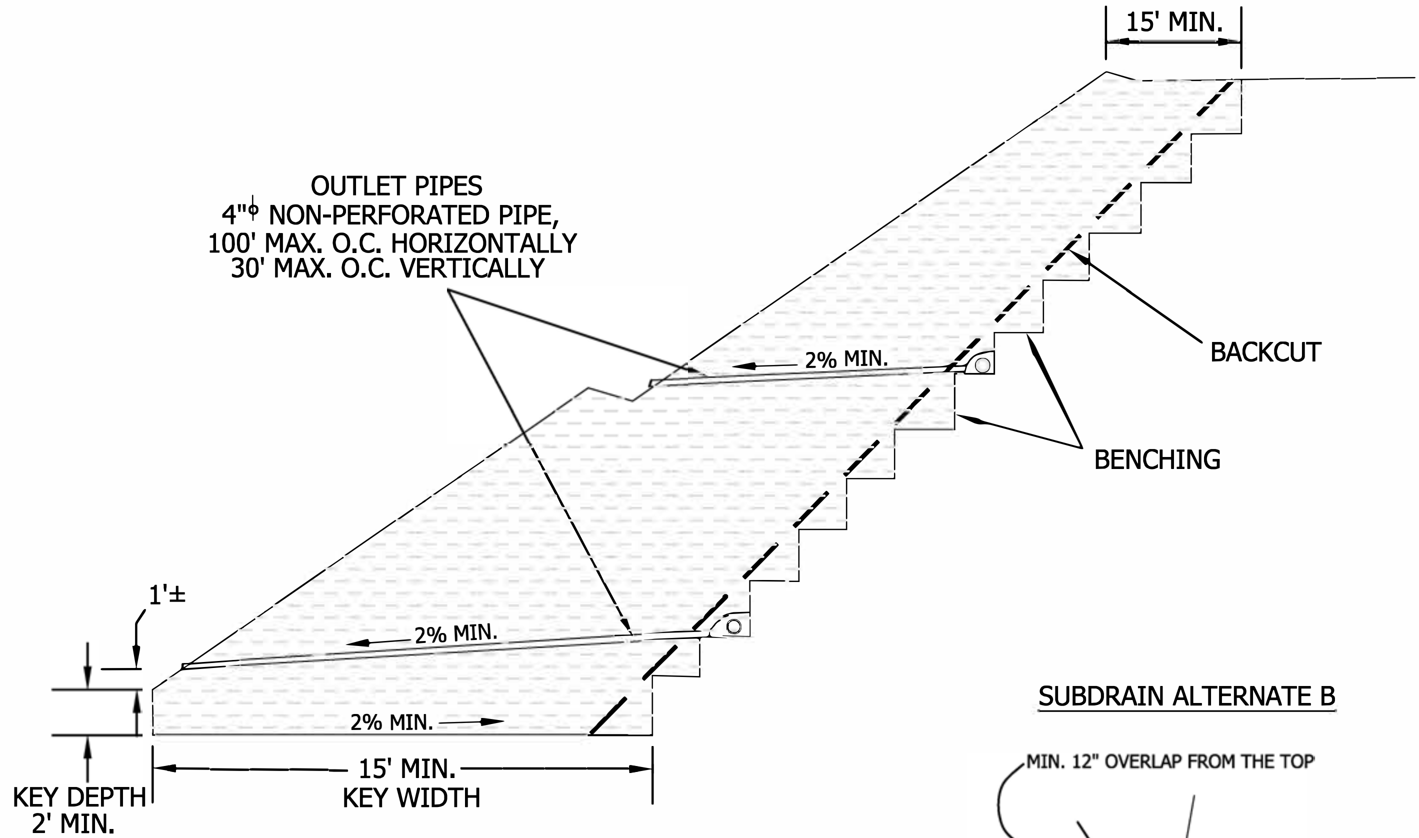
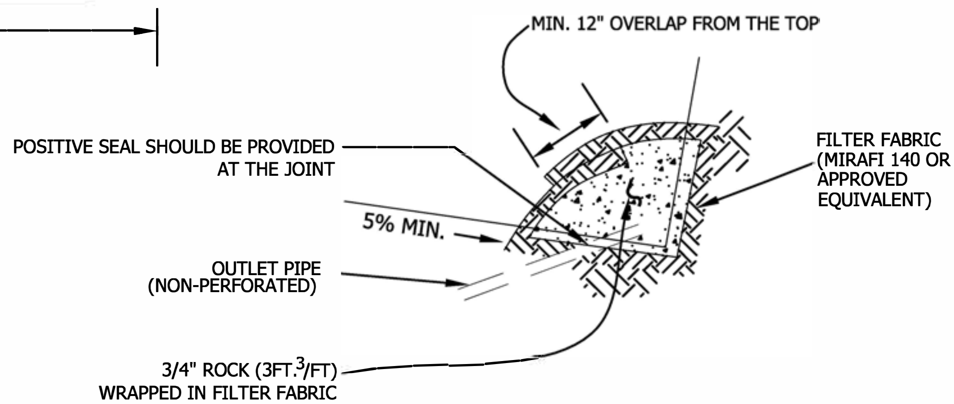
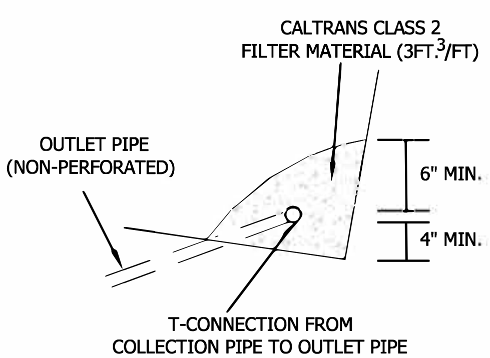


SUBDRAIN ALTERNATE B



SUBDRAIN ALTERNATE A



- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2% towards the outlet.
- SUBDRAIN PIPE - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.
- All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS D	Leighton

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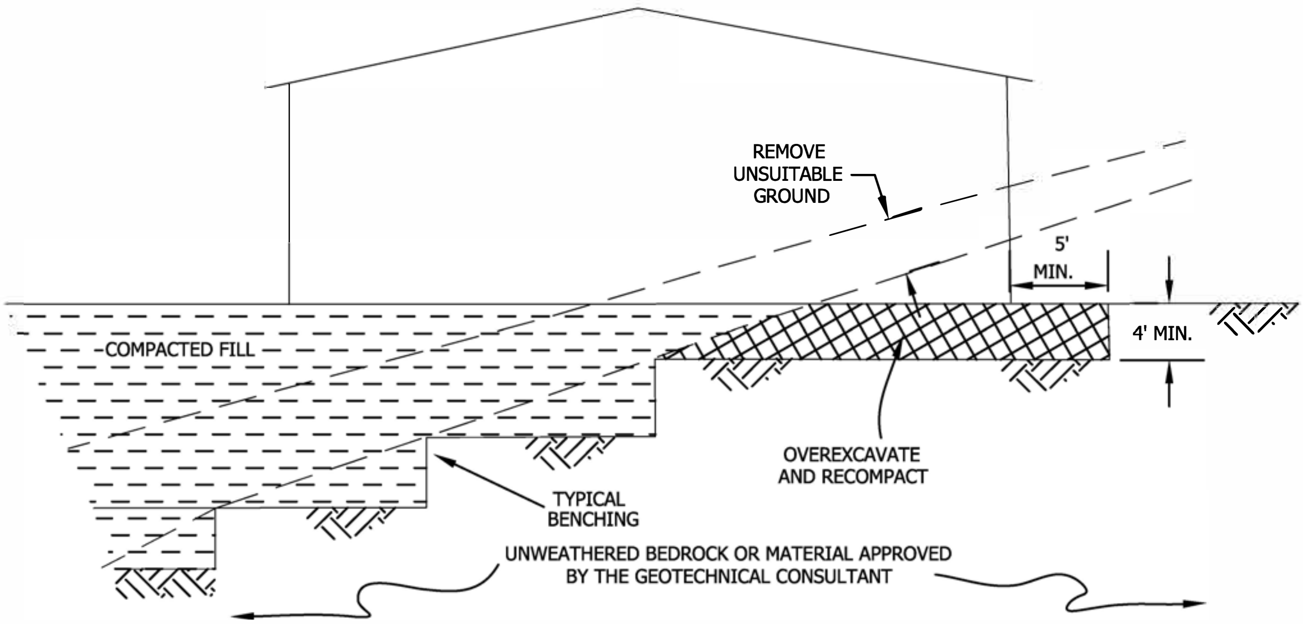
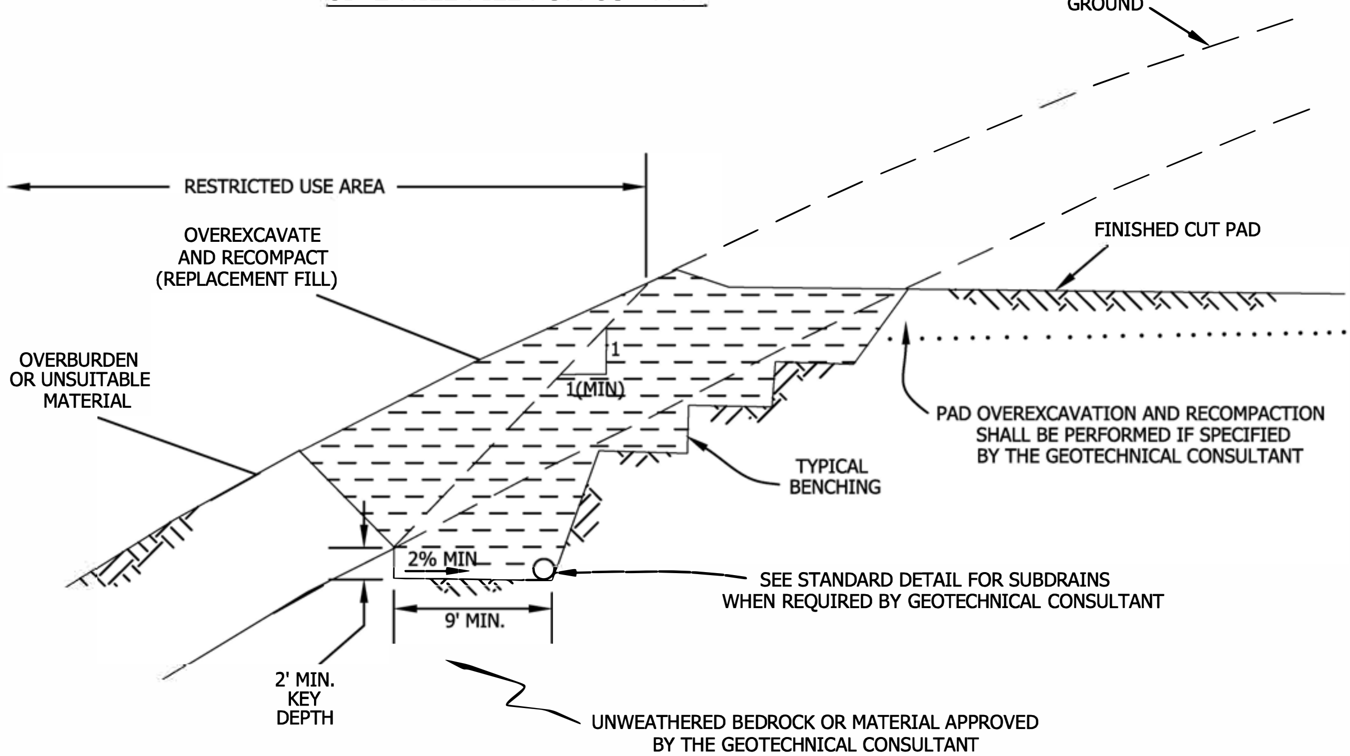
CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS E	Leighton

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APPENDIX F

GBA 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, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them 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 herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences 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.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

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

- for a different client;
- for a different project or purpose;
- 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, the reliability of a geotechnical-engineering report can be 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 you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site 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.*

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

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement 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 through project completion to obtain 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 <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed 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 continuing 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 available 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-phase observations.

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 information 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. 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. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. 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 <u>Not</u> 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 provide 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 obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

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, the engineer's services were not designed, conducted, or intended to prevent 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

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