



GEOTECHNICAL INVESTIGATION
PROPOSED COSTA MESA FIRE STATION NO. 2
REPLACEMENT
800 BAKER STREET
COSTA MESA, ORANGE COUNTY, CALIFORNIA

Prepared For **PBK ARCHITECTS**
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Project No. 038.0000024475

DIVISION OF BUILDING SAFETY CITY OF COSTA MESA REVIEWED FOR CODE COMPLIANCE BY <u>Jeff Tol</u> DATE <u>06/04/2026</u> The approved plans and attached specifications MUST be kept in the job site at all times. No inspections shall be performed without the approved plans on-site. It is unlawful to make any changes or alterations on same without written permission from the Division of Building and Safety, City of Costa Mesa. The stamping of this plan and attached specifications SHALL NOT be construed to permit or approve the violation of any provisions of any City of Costa Mesa Ordinance, State, or Federal law. These permits shall expire and become null and void if the work is not commenced or maintained per CB105.5/CRC 105.5

July 25, 2024

RECOMMEND FOR APPROVAL
CITY OF COSTA MESA
SCOTT FAZEKAS & ASSOCIATES, INC.

BY  Kyle Tonokawa DATE 06/03/2026
Issuance of a building permit is pending approval of all applicable City Agencies.

This issuance or granting of a permit based on approval of these plans shall not be construed to permit or approve any violation of the applicable codes or ordinances. No permit presumed to give authority to violate or cancel the provisions of such codes shall be valid.



Leighton Consulting, Inc.

July 25, 2024

Project No. 038.0000024475

PBK Architects
8163 Rochester Avenue, Suite 100
Rancho Cucamonga, California 91730

Attention: Mr. Kelley Needham, AIA
Principal

Subject: Geotechnical Investigation
Proposed Costa Mesa Fire Station No. 2 Replacement
800 Baker Street
Costa Mesa, Orange County, California

Leighton Consulting, Inc. (Leighton) is pleased to present this geotechnical investigation report in support of the proposed Costa Mesa Fire Station No. 2 Replacement project to be located at 800 Baker Street in the City of Costa Mesa, Orange County, California. The purpose of our geotechnical investigation has been to evaluate the geologic hazards and geotechnical conditions of the site with respect to the proposed improvements and to provide geotechnical recommendations for design and construction.

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known active faults have been mapped through or trending toward the site. However, as is the case for most of southern California, strong ground shaking has and will occur at this site. The site has been mapped to be outside of a State liquefaction susceptibility zone based on our review of Earthquake Zones of Required Investigations for the Newport Beach Quadrangle (see Figure 5, *Seismic Hazards Map*). The site has also been mapped by the State to have a historically highest groundwater table of approximately 20 feet below ground surface based on our review of the Seismic Hazard Report for the Anaheim and Newport Beach 7.5-Minute Quadrangle. Groundwater was encountered within 2 of our 4 borings at depths ranging from approximately 24 to 25 feet below ground surface.

Based on this investigation, the proposed fire station building may be founded on conventional spread footings bearing solely on a zone of newly excavated and

recompacted fill soils derived from onsite soils, overlying undisturbed native soils. This report addresses these and other geotechnical aspects of the project.

We appreciate this opportunity to be of additional service to PBK Architects. If you have any questions or if we can be of further service, please contact us at your convenience at **866-LEIGHTON**, directly at the phone extensions or e-mail addresses listed below.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Site Location and Description

The proposed fire station is to be constructed within an approximately 0.8-acre property located at 800 Baker Street in the City of Costa Mesa, California (see Figure 1, *Site Location Map*). The proposed fire station site is bounded by Randolph Avenue to the north, Baker Street to the south and multi-family residential properties to the east and west. Based on our review of the current elevation model from Google Earth and published topographic maps, the proposed fire station site is generally flat-lying at an elevation of approximately 37 to 40 feet above mean sea level (msl).

Based on our review of historical aerial imagery dating as far back as 1938 (NETR, 2024), the site was primarily open and undeveloped land until sometime between 1938 and 1952, when development began in and around the site vicinity for the development of single-family residential homes. Removal of the single-family homes and development of the existing fire station appeared to have occurred sometime between 1963 and 1972. The site has undergone only minor site improvements until present date.

1.2 Proposed Development

Our understanding of the project is based on email correspondence with you and the provided *Proposed Site Plan*, dated January 3, 2024. Preliminary site layouts depict an approximately 9,000-SF fire station building to replace the existing fire station building. The proposed structure will house new dormitories, bathrooms, offices, conference room, exercise room, day room, dining room/kitchen, laundry room, and a 3,725 SF apparatus bay within the central portion of the building. Also planned are two new equipment pads for a fuel tank and an emergency generator towards the southwestern portion of the site, an outdoor patio to the north side of the building. Visitor and crew parking lots, site fencing, sliding security gates, and a new driveway are also proposed. Ancillary improvements are anticipated to include landscaping, a trash enclosure, and associated underground utilities.

The proposed fire station replacement is to be constructed within the overall approximately 0.8-acre site and slated to replace the existing approximately 6,800-square-foot (SF) fire station building. We understand a temporary fire station and

apparatus bay are to be placed towards the northwest portion of the site until the replacement fire station has been completed.

Structural loading of the proposed building foundations has not been provided to us at the time of this report. We assume that the structures will be relatively lightly loaded.

1.3 **Purpose and Scope of Exploration**

The purpose of our exploration has been to: (1) evaluate geotechnical conditions of the site with respect to the proposed improvements, (2) identify significant geotechnical and geologic issues that would impact proposed improvements, and (3) provide preliminary geotechnical recommendations for design and construction of proposed improvements as currently planned. In accordance with our June 5, 2024, proposal, the scope of our exploration included the following:

- **Research:** We reviewed readily available maps, reports, and historical aerial photographs relevant to this project. Pertinent geotechnical documents are referenced at the end of this report text.
- **Field Exploration:** On June 17, 2024, four (4) hollow-stem auger borings (LB-1 through LB-4) were drilled with a truck-mounted drill rig, each logged and sampled to total depths ranging from approximately 10 feet to 51.5 feet below the existing ground surface (bgs). Encountered earth materials were logged in the field by our field representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Relatively undisturbed soil samples were obtained at selected intervals within these borings using both a ring-lined Modified California split-barrel sampler and an unlined, 2-inch outside diameter Standard Penetration Test (SPT) split-spoon sampler. Although the SPT sampler we used had room for a liner, no liner was used as is customary for this type of work in this area. Sampling resistance blow counts were obtained by dropping a 140-pound, automatic-trip hammer through a 30-inch free fall onto a sampling rod anvil. Modified California and SPT samplers were driven 18 inches and the number of blows was recorded for each 6 inches of penetration. Both sampling methods generally followed respective ASTM D3550 and ASTM D1586 procedures. Representative bulk soil samples were also collected at shallow depths.

Infiltration testing was conducted within LB-3 and LB-4, which were converted into temporary wells IT-1 and IT-2, respectively. Testing was conducted at test zones located between 5 to 10 feet bgs for IT-1 and depths of 10 to 15 feet bgs for IT-2 to estimate the infiltration characteristics of the underlying soil at locations and depths tested. These infiltration tests were conducted in general conformance with Orange County Guidelines.

After drilling, sampling, logging, and infiltration testing, all borings were immediately backfilled with soils generated from drilling activities and patched at the surface with cold-mix asphalt to match existing conditions. Logs of our geotechnical borings and infiltration measurements collected in the field are presented in Appendix A, *Geotechnical Exploration Logs*. The approximate boring locations are shown on the accompanying Figure 2, *Exploration Location Map*.

- **Geotechnical Laboratory Testing:** Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field exploration. Our laboratory testing program was designed to evaluate engineering characteristics of onsite soils. A description of test procedures and results are presented in Appendix B, *Geotechnical Laboratory Testing*.
- **Engineering and Geologic Analysis:** Data obtained from our field exploration and geotechnical laboratory testing were evaluated and analyzed to develop geotechnical conclusions and provide recommendations.
- **Report Preparation:** Results of our geologic hazards review and geotechnical exploration have been summarized in this report, presenting our findings, conclusions, and preliminary geotechnical design recommendations.

This report does not address the potential for encountering hazardous materials in site soils or within groundwater. Important information about limitations of geotechnical reports in general, is presented in Appendix E, *GBA's Important Information About This Geotechnical-Engineering Report*.

2.0 FINDINGS

2.1 Regional Geologic Setting

The project is located in the Peninsular Ranges geomorphic province of southern California, on the Newport Mesa of the Los Angeles Basin in western Orange County (Yerkes et al., 1965). The Newport Mesa is located east of the current channel of the Santa Ana River and west and north of Newport Bay. This mesa is composed of Pleistocene age tidal flat deposits that have been uplifted by the Quaternary development of the northwest-plunging San Joaquin Hills anticline. The dextral Newport-Inglewood-Rose Canyon fault zone trends along this anticlinal axis and its potentially active segments are located as close as approximately 1.7-mile southwest of Costa Mesa Fire Station No. 2. The San Joaquin Hills thrust fault is located approximately 0.8-miles north of the site.

Figure 4, Regional Fault and Historical Seismicity Map illustrates regional active and potentially active fault traces with respect to the site location.

2.2 Subsurface Soil Conditions

Based on results of our research and subsurface exploration, site soils encountered to the depths explored (51.5 feet bgs) consisted of the following:

- **Undocumented Fill (Afu):** We are unaware of any documentation of previous fill placement for this site, so we have characterized all fill onsite as undocumented. Undocumented artificial fill was encountered in our borings to be generally about 5 to 7.5 feet thick. Undocumented fill encountered in our borings consisted primarily of very stiff, reddish brown sandy clay (CL).
- **Old Paralic Deposits overlain by Alluvial-Fan Deposits (Qopf):** The site has been regionally mapped to be underlain by old paralic deposits that have been overlain by alluvial fan deposits. These soils were encountered in our borings below undocumented fill and consisted generally of medium stiff to very stiff sandy silt and clay with sand to explored depths. Occasional interbedded sand and silty sand were encountered. Moisture contents from the underlying alluvial soils within the upper 10 feet ranged from values of 10 to 21 percent moisture and in situ dry densities ranged from 103 to 119 pcf.

More detailed descriptions of subsurface soils encountered are presented on our boring logs in Appendix A.

2.3 **Groundwater**

Groundwater was encountered in 2 of our 4 borings (LB-1 and LB-2) drilled on June 17, 2024, at depths ranging from approximately 24 to 25 feet below the existing ground surface (bgs). According to the Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangles (CGS, 1997), the historically highest groundwater level at the site was on the order of 20 feet below ground surface.

Based on our borings and historical groundwater data, groundwater is not expected to be a significant constraint for development nor is anticipated to be encountered during construction activities for the proposed fire station building.

2.4 **Faulting and Seismicity**

Southern California is a seismically active area. As such, the site will be subject to seismic hazards from numerous sources in the area. The severity of potential seismic hazards is related to site-specific geology, distances from seismic sources, and the magnitude of earthquake events. Principal seismic hazards evaluated on a site-specific basis included: potential for surface rupture along active or potentially active fault traces, magnitude of seismic shaking, and the susceptibility to ground failure (liquefaction, lurching, and seismically induced landslides). Site-specific faulting and seismicity considerations are discussed in the following two subsections:

2.4.1 Surface Fault Rupture: Fault classification criteria adopted by the California Geological Survey, formerly the California Division of Mines and Geology, defines Earthquake Fault Zones along active or potentially active faults. The California Alquist-Priolo Earthquake Fault Zoning Act of 1972 classification system is used in this report, as follows:

- **Active:** An active fault is one that has ruptured within the Holocene epoch (the last 11,700 years).
- **Potentially Active:** A fault that has ruptured during the last 1.8 million years (Quaternary period) but has not been proven by direct evidence to have not moved within the Holocene epoch is considered to be potentially active.

- **Inactive:** A fault that has not moved during both Pleistocene and Holocene epochs (that is, no movement within the last 1.8 million years) is considered to be inactive.

This project lies outside of any State designated Earthquake Fault Zones (CGS, 1998). Based on our review of available in-house literature, and as depicted on Figure 4, *Regional Faults and Historical Seismicity Map*, there are no currently known active fault traces that traverse or trend towards this site. The Third California Earthquake Rupture Forecast (UCERF3) has estimated locations of potential fault rupture outside or trending outside the project site.

The closest known active or potentially active fault is the San Joaquin Hills blind thrust located approximately 0.8 mile north of the project site. Other known regional active or potentially active faults that could produce the most significant ground shaking at the site include those related to the Newport-Inglewood fault zone, the Palos Verde fault zone, and the Elsinore fault zone.

Nearby fault traces have been depicted in Figure 4, *Regional Fault and Historical Seismicity Map*.

- 2.4.2 Seismicity (Ground Shaking):** A principal seismic hazard that could impact this site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults throughout southern California. An evaluation of historical seismicity from significant past earthquakes related to the site was performed. Plotted on Figure 4, *Regional Fault and Historical Seismicity Map*, are epicenters of historic earthquakes in and around Costa Mesa, color-coded as a function of magnitude. Based on this map, it appears that the site has not been exposed to relatively significant seismic events. We are unaware of documentation indicating that past earthquake damage has occurred in the site vicinity. In addition, we are unaware of damage in the site vicinity as the result of liquefaction, lateral spreading, or other related phenomena.

2.5 **Secondary Seismic Hazards**

In general, secondary seismic hazards for sites in this region could include soil liquefaction, earthquake-induced settlement, slope instability and landslides, earthquake-induced seiches and tsunamis flooding. Site-specific potential for secondary seismic hazards is discussed in the following subsections:

2.5.1 Liquefaction Potential: Liquefaction is the loss of soil strength due to a buildup of excess pore-water pressure during strong and long-duration ground shaking. Liquefaction is associated primarily with loose (low density), saturated, relatively uniform fine- to medium-grained, clean cohesionless soils. As shaking action of an earthquake progresses, soil granules are rearranged, and the soil densifies within a short period. This rapid densification of soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, soil shear strength reduces abruptly and temporarily behaves similar to a fluid. For liquefaction to occur there must be:

- (1) loose, clean granular soils,
- (2) shallow groundwater, **and**
- (3) strong, long-duration ground shaking

The State of California has evaluated the site for liquefaction hazards and the site is **not** within a mapped liquefaction susceptibility zone (CGS, 1998). Groundwater was encountered in our borings at a depth of approximately 24 feet bgs. The site has been mapped with the historically highest groundwater table on the order of 20 feet below ground surface based on our review of the Seismic Hazard Report for the Anaheim and Newport Beach 7.5-Minute Quadrangle (CGS, 1997).

Our analysis was based on the modified Seed Simplified Procedure as detailed by Youd et al. (2001) and Martin and Lew (1999), which compares the seismic demand on a soil layer (Cyclic Stress Ratio, or CSR) to the capacity of the soil to resist liquefaction (Cyclic Resistance Ratio, or CRR) (Youd et al., 2001). A minimum required factor of safety of 1.3 was used in our analysis, with factor of safety defined as CRR/CSR. As required, our analysis assumes that the design earthquake would occur while the groundwater is at its estimated historically highest level. In the SPT method, soil resistance to liquefaction is estimated based on several factors, including SPT sampling blow counts normalized and corrected for several factors including fines content, and overburden pressure. Soil plasticity and moisture content are also considered in an evaluation of liquefaction. Parameters utilized in our analysis include

Standard Penetration Test (SPT) results from the borings, visual descriptions of soil samples retrieved, and geotechnical laboratory test results.

Due to relatively dense nature of the underlying soils, and the presence of cohesive fine-grained soils (silts and clays) which were screened for liquefaction susceptibility (Bray and Sancio, 2006), the underlying subsurface soils are **not** considered susceptible to liquefaction when considering a design historic high groundwater depth of 20 feet bgs and corresponding peak ground acceleration.

2.5.2 Lateral Spreading: Lateral spreading is unlikely to occur at the site due to the lack of liquefaction potential and lack of significant topographic relief at and around this site.

2.5.3 Seismically Induced Settlement: During a strong seismic event, non-liquefaction, seismically induced settlement can occur within loose and dry granular soils. Settlement caused by ground shaking is often unevenly distributed, which can result in differential settlement. Loose fill soils are typically highly susceptible to seismically induced settlement. Undocumented fill soils under the proposed building footprint are recommended (discussed later in this report) to be overexcavated and recompacted to reduce dynamic settlement concerns.

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed, and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). Design/historic high groundwater levels of 20 feet below ground surface were used in the analysis. Based on our analysis, a potential for approximately 0.1 inch of seismic settlement is estimated at the site. Results of our seismic settlement analysis is presented in Appendix C.

If the potential differential settlement is estimated as half of the total seismic settlement over a horizontal distance of 30 feet, this would result in a maximum differential settlement of under 0.1 inch in 30 feet, or angular distortion of 0.00014L. This would be within the differential settlement threshold of 0.002L for structures of Risk Category IV, as listed in Table 12.13-3 of ASCE 7-16. The structural engineer should determine Structure Type and Risk Category and evaluate whether the differential settlement estimates described above are tolerable. A copy of ASCE 7-16 Table 12.13-3 is provided as follows for

reference.

Table 12.13-3 Differential Settlement Threshold

Structure Type	Risk Category		
	I or II	III	IV
Single-story structures with concrete or masonry wall systems	0.0075L	0.005L	0.002L
Other single-story structures	0.015L	0.010L	0.002L
Multistory structures with concrete or masonry wall systems	0.005L	0.003L	0.002L
Other multistory structures	0.010L	0.006L	0.002L

2.5.4 Slope Instability and Landslides: Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The site has been mapped by the State to **not** be within a landslide hazard zone based on our review of Earthquake Zones of Required Investigations for the Newport Beach Quadrangle (CGS, 1998). Considering the site and vicinity are relatively flat, the potential for seismically induced landslide activity is less than significant.

2.5.5 Earthquake-Induced Seiches and Tsunamis: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are predominately ocean waves generated by undersea large magnitude fault displacement or major ground movement.

The site is located approximately 5 miles inland from the Pacific Ocean and is mapped outside of a tsunami hazard area as delineated by the State of California (CGS, 2021). There are no large nearby enclosed bodies of water in relation to the project site. Thus, the risk of seiches and tsunamis affecting the site is considered low.

2.5.6 Earthquake-Induced Inundation: This inundation hazard is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. Figure 7, *Dam Breach Inundation Map*, shows the subject **not** mapped within a dam breach inundation zone. Based on our review of available data, the potential for an earthquake induced inundation event is considered low.

2.6 **Storm-Induced Flood Hazard**

As depicted on Figure 6, *Flood Hazard Zone Map*, the site is mapped outside of a “100-year” flood zone; however, the northernmost portion of the site **is** mapped within a 0.2% Annual Flood Hazards zone also referred to as a “500-year” flood zone, as defined by the Federal Emergency Management Agency’s (FEMA’s) Flood Insurance Rate Map (FIRM). Based on our review FEMA’s FIRM map number 06059C02675 J (FEMA 2009) the site is mapped within Zone X “Other Flood Areas” which indicates the site is an area of 0.2% annual chance flood; area of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile and protected by levees from 1% annual chance flood. This corresponds to the nearby Santa Ana Delhi Channel which is located approximately 1,900 feet east of the project site. As such, the risk of flood hazards is considered minor, but the potential risk should be considered during the design of the proposed fire station.

2.7 **Infiltration Testing**

Infiltration testing was conducted within two of our borings onsite (LB-3/IT-1 and LB-4/IT-2) to estimate the infiltration characteristics of the onsite soils at the depths and locations tested. Infiltration testing was conducted with bottom test zone depths of approximately 15 feet bgs within boring IT-1 and 10 feet bgs within boring IT-2.

Borehole infiltration tests are useful for field measurements of soil infiltration rates, and are suited for testing when the design depth of the basin or chamber is deeper than current existing grades. It should be noted that this is a clean-water, small-scale test, and that correction factors need to be applied. A test consists of excavating a boring to the depth of the test (or deeper as long as it is partially backfilled with soil and a bentonite plug with a thin soil covering is placed just below the design test elevation). A layer of clean sand or gravel is then placed in the boring bottom to temporarily support a perforated well casing pipe system. Once the well casing pipe has been installed, coarse sand or gravel is poured in the annular space outside of the well casing within the test zone to prevent the boring from caving/collapsing or spalling when water is added. Water is added into the boring to an initial height. As water infiltrates into the soils, measurements are taken of the height of the water column at equally timed intervals (known as falling head test). We also conducted a constant water head by adding water to the borings as water infiltrated into the soil, while maintaining a relatively constant water head in the boring. The incremental infiltration rate as measured during

intervals of the test was defined as the incremental flow rate of water infiltrated, divided by the surface area of the infiltration interface. Well permeameter tests were conducted based on USBR 7300-89 method and in general accordance with Orange County guidelines.

Raw Infiltration rates for the well permeameter test yielded a rate of 26 inches/hour within Boring TI-2; however, confining fine grained soil layers (silts and clay) were encountered at depths as shallow as 12.5 feet bgs within the immediate area and as shown within our borings logs conducted at the site. Results of infiltration testing are provided in Appendix A. Further discussion of infiltration testing and related recommendations are included in Section 3.12.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions

This site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone for surface fault rupture. However, as is the case for most of southern California, the most significant geotechnical issue at the site is related to the potential for strong seismic shaking. Historic groundwater levels have been estimated to be on the order of 20 feet below the surface based on the Seismic Hazard Report for the Anaheim and Newport Beach 7.5-Minute Quadrangle Report (CGS, 1997). Based on our analysis and the relatively dense nature of the underlying soils, the potential for liquefaction and seismic induced settlement is considered low at this site. The northern portion of the site is mapped within a 500-year flood zone and special consideration of this occurrence should be taken into consideration during design of the project. Recommendations for design and construction of proposed improvements are provided in the following sections.

The recommendations below are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. These recommendations are also based upon proper field observation and testing during construction. The project geotechnical engineer should be notified of suspected variances in field conditions to evaluate the effect upon the recommendations presented herein. These recommendations are considered minimal and may be superseded by more restrictive requirements of the civil and structural engineers, the County of Orange, and other governing agencies.

3.2 Earthwork

Project earthwork is expected to include overexcavation and recompaction of undocumented fill soils, and partial removals of onsite alluvial soils, below the proposed new building footprint as described in the following subsections:

- 3.2.1 Earthwork Observation and Testing:** Leighton should observe and test all grading and earthwork to check that the site has been properly prepared, to assess that selected fill materials are satisfactory, and to evaluate that placement and compaction of fills has been performed in accordance with our

recommendations and the project specifications. Any imported soil or aggregate material to be evaluated for its suitability as onsite fill material should be submitted to a Leighton geotechnical laboratory at least two working days in advance of earth material placement and compaction. Project plans and specifications should incorporate recommendations contained in the text of this report.

Variations in site conditions are possible and may be encountered during construction. To confirm correlation between soil data obtained during our field and laboratory testing and actual subsurface conditions encountered during construction, and to observe conformance with approved plans and specifications, we should be retained to perform continuous or intermittent review during earthwork, excavation, and foundation construction phases. Conclusions and recommendations presented in this report are contingent upon construction geotechnical observation services.

3.2.2 Site Preparation: Prior to construction, the site should be cleared of vegetation, trash and debris, which should be disposed of offsite. Any underground obstructions should be removed. Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted. Unsuitable deposits and undocumented fill should be excavated and removed from the proposed building footprints prior to fill placement. Removal of these unsuitable deposits and undocumented fill should be as recommended by Leighton Consulting during grading and further evaluated based on the foundation system.

Based on encountered soil conditions, we recommend that all existing fill to be completely removed, which were found to be generally 3 to 5 feet thick in our exploratory borings, but locally thicker undocumented fills are expected to be exposed during grading. Furthermore, potentially compressible native soils are recommended to be excavated below proposed building footprints to at least 3 feet below the bottoms of proposed footings or at least 5 feet below existing grade, whichever is deeper. Overexcavation bottoms should be evaluated by Leighton, and localized deeper removals may be recommended during grading based on exposed conditions.

Where possible, overexcavation bottoms should extend horizontally a distance beyond the outside edges of proposed building perimeter footings equal to the depth of excavation below finish grade or at least 5 feet horizontally, whichever is greater. Where this is not achievable, overexcavation should be reviewed on a case-by-case basis. Overexcavation should encompass the whole new building footprint including attached columns.

Areas outside proposed building pads, planned for asphalt and/or concrete pavement, should be overexcavated to a minimum depth of 24 inches below existing or finish grade, or 12 inches below proposed pavement sections; whichever is deeper.

Resulting removal excavation bottom surfaces should be observed by Leighton prior to placement of any backfill or new construction. It is essential that all existing fill soils be excavated from the proposed building footprints, regardless of depth. After overexcavation is completed and prior to fill placement, exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to 3 percentage points above optimum moisture content, and recompact to a minimum 90 percent relative compaction as determined by ASTM D1557 standard test method (modified Proctor compaction curve).

3.2.3 Fill Placement and Compaction: Onsite soils free of organics and debris are suitable for use as compacted structural fill provided it is also free of oversized material (greater than 8 inches in its largest dimension). However, any soil to be placed as fill, whether onsite or imported material, should be first viewed by Leighton and then tested if and as necessary, prior to approval for use as compacted fill. All structural fill should be free of hazardous materials.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to at least 3 percentage points above optimum moisture content and compacted to a minimum 90% relative compaction as determined by ASTM D1557 standard test method (modified Proctor compaction curve) within the building footprint. Aggregate base for pavement sections should be compacted to a minimum of 95% relative compaction.

No debris or oversize cobbles were observed at the site during our exploration. If oversize material and debris are encountered during grading or during demolition of the existing fire station building and associated features onsite, it

should be removed from soils prior to placement as compacted fill.

3.2.4 Shrinkage or Bulking: The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as in processing an overexcavation bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Based on our laboratory test results for the underlying soils at the site we estimate the following earth volume changes will occur during grading:

Shrinkage and Subsidence	
Shrinkage	Approximately 3 +/- 5 percent
Subsidence (overexcavation bottom processing)	Approximately 0.1 foot

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

3.3 Seismic Design Parameters

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

The following parameters should be considered for design under the 2022 CBC:

Table 1 . 2022 CBC Site-Specific Seismic Parameters

2022 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2022 CBC
Site Latitude and Longitude: 33.6806, -117.8899	
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	D**
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_s	1.301 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.466g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.000
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	1.834*
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.301 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	0.855* g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	0.867 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.570* g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), PGA	0.558 g
Site Coefficient for Mapped MCE_G PGA (11.8.3.2), F_{PGA}	1.100
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), PGA_M	0.614 g

* See Section 11.4.8 of ASCE 7-16. A site-specific ground motion hazard analysis in accordance with Section 21.2 of ASCE 7-16 is required for this site. **Per Supplement 3 to ASCE 7-16, a site-specific ground motion hazard analysis is not required where the value of the parameters S_{M1} and S_{D1} in the table are increased by 50%.**

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

A Site Class analysis is included in Appendix C in accordance with ASCE 7-16 Chapter 20. Subsurface data below 50 feet was conservatively estimated using Standard Penetration Test (SPT) blowcounts from our deepest boring. Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 7.69 (M_W) at a distance on the order of 6.58 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

3.4 Foundations

Based on our exploration and our experience in the region, onsite soils exposed at pad grade are anticipated to exhibit a low expansive potential. As such, we recommend that the proposed structures be supported on conventional shallow

spread or strip footings foundation system in accordance with the California Building Code (CBC). Anticipated foundation loads were not available during preparation of this report. We assumed maximum column dead loads up to ($\leq$) 50 kips and wall loads of 3 kips per lineal foot for our preliminary foundation recommendations. Overexcavation and recompaction of footing subgrade soils should be performed as detailed in Section 3.2 of this report. Shallow foundation recommendations are provided in the following section.

3.4.1 Minimum Embedment and Width

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

3.4.2 Allowable Bearing

An allowable bearing pressure of 2,000 pounds-per-square-foot (psf) may be used, based on an assumed embedment depth of 18 inches and minimum width described above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,000 psf. If higher bearing pressures are required, this should be reviewed on a case-by-case basis and may include additional overexcavation and/or soil reinforcement. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

3.4.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.30. The passive resistance may be computed using an allowable equivalent fluid pressure of 240 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.4.4 Increase in Bearing and Friction – Short Duration Loads

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

3.4.5 Settlement Estimates

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

3.5 Concrete Slab-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with 2022 CBC requirements for soil with a low expansion potential. More stringent requirements may be required by the structural engineer and/or architect; however, slabs-on-grade should have the following minimum recommended components:

- **Subgrade:** Slab-on-grade subgrade soil should be moisture conditioned to or within 3% over optimum moisture content, to a minimum depth of 24 inches within building footprints, and compacted to 95% of the modified Proctor (ASTM D1557) laboratory maximum density prior to placing either a moisture barrier, steel and/or concrete.
- **Moisture Barrier:** A moisture barrier consisting of at least 15-mil-thick Stego-wrap vapor barriers (see: http://www.stegoindustries.com/products/stego_wrap_vapor_barrier.php), or equivalent, should then be placed below slabs where moisture-sensitive floor coverings or equipment will be placed.
- **Reinforced Concrete:** A conventionally reinforced concrete slab-on-grade with a thickness of at least 4 inches should be placed in pedestrian areas without heavy loads. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 4 rebar placed at 18 inches on-center, each direction (perpendicularly), mid-depth in the slab. A modulus of subgrade reaction (k) as a linear spring constant, of 90 pounds per square inch per inch deflection (pci) can be used for design of heavily loaded slabs-on-grade, assuming a linear response up to deflections on the order of ¾ inch.

- **Slab-On-Grade Control Joints:** Slab-on-grade crack control joint locations and spacing should be designed by the project Structural Engineer (SE). We suggest control joints of 12 feet on center. Control joints should form square panels.

Minor cracking of the concrete as it cures, due to drying and shrinkage, is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

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

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

3.6 **Sulfate Attack and Ferrous Corrosion Protection**

- 3.6.1 Sulfate Exposure:** Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. A potentially high sulfate content could also cause corrosion of reinforcing steel in concrete. Section 1904A of the 2022 California Building Code (CBC) defers to the American Concrete Institute's (ACI's) ACI 318-19 for concrete durability requirements. Table 19.3.1.1 of ACI 318-19 lists "*Exposure categories and classes*," including sulfate exposure as follows:

Table 2. Sulfate Concentration and Exposure

Soluble Sulfate in Water (parts-per-million)	Water-Soluble Sulfate (SO ₄) in soil (percentage by weight)	ACI 318-19 Sulfate Class
0-150	0.00 - 0.10	S0 (negligible)
150-1,500	0.10 - 0.20	S1 (moderate*)
1,500-10,000	0.20 - 2.00	S2 (severe)
>10,000	>2.00	S3 (very severe)

*or seawater

3.6.2 Ferrous Corrosivity: Many factors can modify corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled “*Effects of Soil Characteristics on Corrosion*” (February 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as follows:

Table 3. Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to buried metallic structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. Chloride and sulfate ion concentrations, and pH appear to play secondary roles in modifying corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures.

3.6.3 Corrosivity Estimates: To evaluate corrosion potential of soils sampled from this site, we tested a bulk soil sample for soluble sulfate content, soluble chloride content, pH and resistivity. Results of these tests are summarized below:

Table 4. Results of Corrosivity Testing

Locations	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
Boring LB-2	0 - 5	99	120	9.1	1,620

Note: mg/kg = milligrams per kilogram, or parts-per-million (ppm)

These results are discussed as follows:

- Sulfate Exposure:** Based on recovered near surface soils and Table 19.3.1.1 of ACI 318-19, in our opinion, sulfate exposure should be considered “negligible” with an Exposure Class S0 for soils sampled at the site. Based on Table 19.3.2.1 of ACI 318-19, for this Exposure Category S0, there would be no restrictions on cement type (“cementitious material”) nor water/cement ratio, and an f_c' (28-day compressive strength) of at least 2,500 pounds per square inch (psi) is required at a minimum for structural concrete.
- Ferrous Corrosivity:** As shown above, minimum soil resistivity of 1,620 ohm-centimeters was measured in our laboratory test. In our opinion, it appears for site soils that corrosion potential to buried steel may be characterized as “severely corrosive” at the site. Ferrous pipe buried in moist to wet site earth materials may be replaced by high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Or ferrous pipe may be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site earth materials. A corrosion engineer may also be consulted to further evaluation for more detailed recommendations relating to onsite soil corrosion characteristics.

3.7 **Pavement Section Design**

Based on design procedures outlined in the 2017 Caltrans *Highway Design Manual* and a design R-value of less than 5 for onsite subgrade soils based on laboratory testing, preliminary flexible pavement sections were calculated for the Traffic Indices (TIs) tabulated, and are listed below:

Table 5. Hot Mixed Asphalt (HMA) Pavement Sections

Assumed Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0 (automobile parking, driveways)	3.0	10.0
6.0 (truck traffic)	4.0	11.5
7.0 (roadways and heavy truck traffic)	6.0	12.0

For fire truck (60,000-pound “apparatus”) lanes, asphalt pavements designed for a TI=6.0 are recommended. However, note that undistributed apparatus outrigger loads could cause local asphalt pavement punching damage. When possible, outrigger loads should be distributed over asphalt pavements with planks and plywood. Otherwise, areas where outrigger loads are anticipated could be paved with 8-inch-thick concrete as described below.

Onsite clays are expected to have a low expansive potential, and R-value test on a near-surface sample was low ($R < 5$). A subgrade sample should be collected during grading to perform additional R-Value tests to verify pavement design. Alternatively, in order to reduce the pavement section thicknesses provided in the table above, the subgrade soils can be cement-treated. We recommend that additional samples be collected during construction for R-Value testing, and also to determine the ideal cement treatment ratio (typically on the order of 3 to 5 percent). The following table provides pavement sections for cement-treated subgrade.

**Table 6. Hot Mixed Asphalt (HMA) Pavement Sections
Cement Treated Subgrade***

Assumed Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0 (automobile parking, driveways)	3.0	5.5
6.0 (truck traffic)	3.5	7.5
7.0 (roadways and heavy truck traffic)	4.0	9.5

*subgrade treated with 3%-5% cement

Portland cement concrete (PCC) pavement sections were calculated in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for three Traffic Indices (TIs) are presented below:

Table 7. Portland Cement Concrete Pavement Sections

Assumed Traffic Index	PC Concrete (inches)	Base Course (inches)
5.0 (automobile parking, driveways)	6	8
6.0 (roadways and truck traffic)	8	

We have assumed that this Portland cement concrete will have a compressive strength of at least 3,000 psi. Prior to placement of aggregate base, subgrade soils should be scarified to a minimum depth of 8 inches, moisture-conditioned, as necessary, and recompact to a minimum of 95 percent relative compaction, determined in accordance with ASTM D1557 modified Proctor laboratory maximum density. Aggregate base should be placed in thin lifts; moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction. Field observation and periodic testing, as needed during placement of base course materials, should be undertaken to ensure that requirements of current Caltrans' *Standard Specifications* and Special Provisions are fulfilled. Consideration should be given to reinforce concrete pavements where large outrigger point loads are anticipated.

Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet. All pavement construction should be performed in accordance with the current Caltrans *Standard Specifications* or *Standard Specifications for Public Works Construction* ("Greenbook"). Recommended structural pavement materials should conform to the specified provisions in the current Caltrans *Standard Specifications* including grading and quality requirements, shown below:

- **Asphalt Concrete (Hot Mixed Asphalt)** for pavement should be Type A and should conform to Section 39 of the *Standard Specifications*. Asphalt concrete specimens should be tested for surface abrasion in accordance with CT-360.
- **Portland Cement Concrete (PCC)** pavement should conform to Section 40 of the *Standard Specifications*. PCC pavement materials (pavement, structures, minor concrete) should conform to Section 90 of the *Standard Specifications*.
- **Class II Aggregate Base (AB)** should conform to Section 26 of the *Standard Specifications*.

Traffic Indices (TIs) used in our pavement design are considered reasonable values for typical parking lot areas, and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance.

Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving, will result in premature pavement failure. Traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel-load analysis or a traffic study. The project Civil Engineer should confirm the TI assumptions.

- 3.8 Surface Drainage:** Pad drainage should be designed to collect and direct surface water away from structures to approved drainage facilities. Hardscape drains should be installed and drain to approved storm water disposal systems. Drainage patterns and drainpipes approved at the time of fine grading should be maintained throughout the life of proposed structures.

3.9 Retaining Wall Recommendations

The following retaining wall recommendations are included for design consideration of walls with a height less than 6 feet. We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 8, *Retaining Wall Backfill and Subdrain Detail*. Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall and are, therefore, not recommended. Retaining wall locations and configurations are unknown at the time of this report.

Table 8. Retaining Wall Design Parameters

Static Equivalent Fluid Pressure (pcf)	
Condition	Level Backfill
Active	45
At-Rest (drained, compacted-fill backfill)	65
Passive (ultimate)	230 (Max. 3,000 psf)

The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

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

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.35 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design. A third of uniform vertical surcharge-loads should be applied at the surface 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. To account for automobile parking surcharge, we suggest that a uniform horizontal pressure of 100 psf (for restrained walls) or 70 psf (for cantilever walls) be added for design, where autos are parked within a horizontal distance behind the retaining wall less than the height of the retaining wall stem.

We recommend that the wall designs for walls 6 feet tall or taller be checked seismically using an *additive seismic* Equivalent Fluid Pressure (EFP) of 17 pcf, which is added to the EFP. The *additive seismic* EFP should be applied at the retained midpoint.

Conventional retaining wall footings should have a minimum width of 24 inches and a minimum embedment of 12 inches below the lowest adjacent grade. An allowable bearing pressure of 2,000 psf may be used for retaining wall footing design, based on the minimum footing width and depth. This bearing value may be increased by 250 psf per foot increase in width or depth to a maximum allowable bearing pressure of 3,000 psf.

3.10 Infiltration Recommendations

Based on our onsite observations, and infiltration test results summarized in the table below, reliance of infiltration into onsite near surface native soils is not considered feasible due to encountered fine grained soils (clays and silts) within the near surface soils and the mapped historical high groundwater table within the area.

Orange County Guidelines indicate that BMP system invert elevations need to have a minimum 10-foot vertical setback from historical groundwater elevations. As mentioned in Section 2.3, the historical high ground water level at the site is mapped to be approximately 20 feet bgs, resulting in an maximum allowable invert elevation of 10 feet bgs.

Although infiltration testing with a bottom depth of 10 feet bgs produced moderate rates for the test itself, soils with higher percent fines and fine-grained soils (silts and clays) were generally located above and below this very narrow test zone with favorable soils at depths ranging from 8 to 10 feet, causing a confining layer within neighboring borings. The very narrow clean sand layer (6% fines) was not encountered within any of our other borings onsite. It is likely that water infiltrated at depths of approximately 10 feet bgs through prolonged infiltration will tend to migrate laterally rather than vertically and produce lower infiltration values.

4.0 CONSTRUCTION CONSIDERATIONS

4.1 Trench Excavations

To protect workers entering excavations, all temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA and Cal-OSHA requirements, and the current edition of the California Construction Safety Orders.

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

Contractors should be advised that fill soils should initially be considered Type C soils as defined in the California Construction Safety Orders. As indicated in Table B-1 of Article 6, Section 1541.1, Appendix B, of the California Construction Safety Orders, excavations less-than (<) 20 feet deep within Type C soils should be sloped back no steeper than 1½:1 (horizontal:vertical), where workers are to enter the excavation. This may be impractical near adjacent existing utilities and structures; so shoring may be required depending on trench locations. Stiff undisturbed native clays will stand steeper.

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

4.2 Temporary Shoring

Temporary cantilever shoring can be designed based on the active equivalent fluid pressure of 45 pounds-per-cubic-foot (pcf). If excavations are braced at the top and at specific depth intervals, then braced earth pressure may be approximated by a uniform rectangular soil pressure distribution. This uniform pressure

expressed in pounds-per-square-foot (psf), may be assumed to be 25 multiplied by H for design, where H is equal to the depth of the excavation being shored, in feet. These recommendations are valid only for trenches not exceeding 15 feet in depth at this site.

4.3 **Trench Backfill**

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* (SSPWC, “Greenbook”), 2024 Edition. Utility trenches may be backfilled with onsite material free of rubble, debris, organic and oversized material up to 3 inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

- (1) **Granular Bedding:** a uniform sand material with a Sand Equivalent (SE) greater-than- or-equal-to ($\geq$) 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer). Due to the possibility of shallow groundwater in select areas, sand bedding should not be jetted, but should be mechanically compacted in accordance with the Greenbook, latest edition.
- (2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the SPWC. CLSM bedding should be placed to 1-foot (0.3 m) over the top of the conduit and vibrated.

We recommend that open-graded crushed rock or similar material not be used as bedding material, unless special provisions are implemented to limit the migration of surrounding soil into the open-graded material, including surrounding the open-graded material with filter fabric (Mirafi 140N or equivalent), or mixing sand with the open-graded material. Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. The bedding and shading sand is recommended to be densified in place by vibratory, lightweight compaction equipment.

Trench backfill over the pipe bedding zone may consist of native and clean fill soils. All backfill should be placed in thin lifts (appropriate for the type of compaction equipment), moisture conditioned to slightly above optimum, and mechanically compacted to at least 90 percent of the laboratory derived maximum density as determined by ASTM Test Method D 1557.

4.4 **Geotechnical Services During Construction**

Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical exploration, testing and/or analysis may be required based on final plans. Leighton Consulting, Inc. should review site grading, foundation, and shoring (if any) plans when available, to comment further on geotechnical aspects of this project and check to see general conformance of final project plans to recommendations presented in this report.

Leighton Consulting, Inc. should be retained to provide geotechnical observation and testing during excavation and all phases of earthwork. Our conclusions and recommendations should be reviewed and verified by us during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- During all excavation,
- During installation of ground improvements
- During compaction of all fill materials,
- After excavation of all footings and prior to placement of concrete,
- During utility trench backfilling and compaction,
- During pavement subgrade and base preparation, and/or
- If and when any unusual geotechnical conditions are encountered.

5.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This exploration was performed with the understanding that this subject site is proposed for development as described in Section 1.2 of this report. Please also refer to Appendix E, *GBA's Important Information About This Geotechnical-Engineering Report*, presenting additional information and limitations regarding geotechnical engineering studies and reports.

Until reviewed and accepted by the reviewing government agency, this report may be subject to change. Changes may be required as part of the review process. Leighton Consulting, Inc. assumes no risk or liability for consequential damages that may arise due to design work progressing before this report is reviewed and accepted.

This report was prepared for PBK Architects based on their needs, directions and requirements at the time of our exploration, in accordance with generally accepted geotechnical engineering practices at this time in Orange County for public sites. This report is not authorized for use by, and is not to be relied upon by, any party except PBK Architects and their design and construction management team, with whom Leighton Consulting, Inc. has contracted for this work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton Consulting, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, and/or strict liability of Leighton Consulting, Inc.

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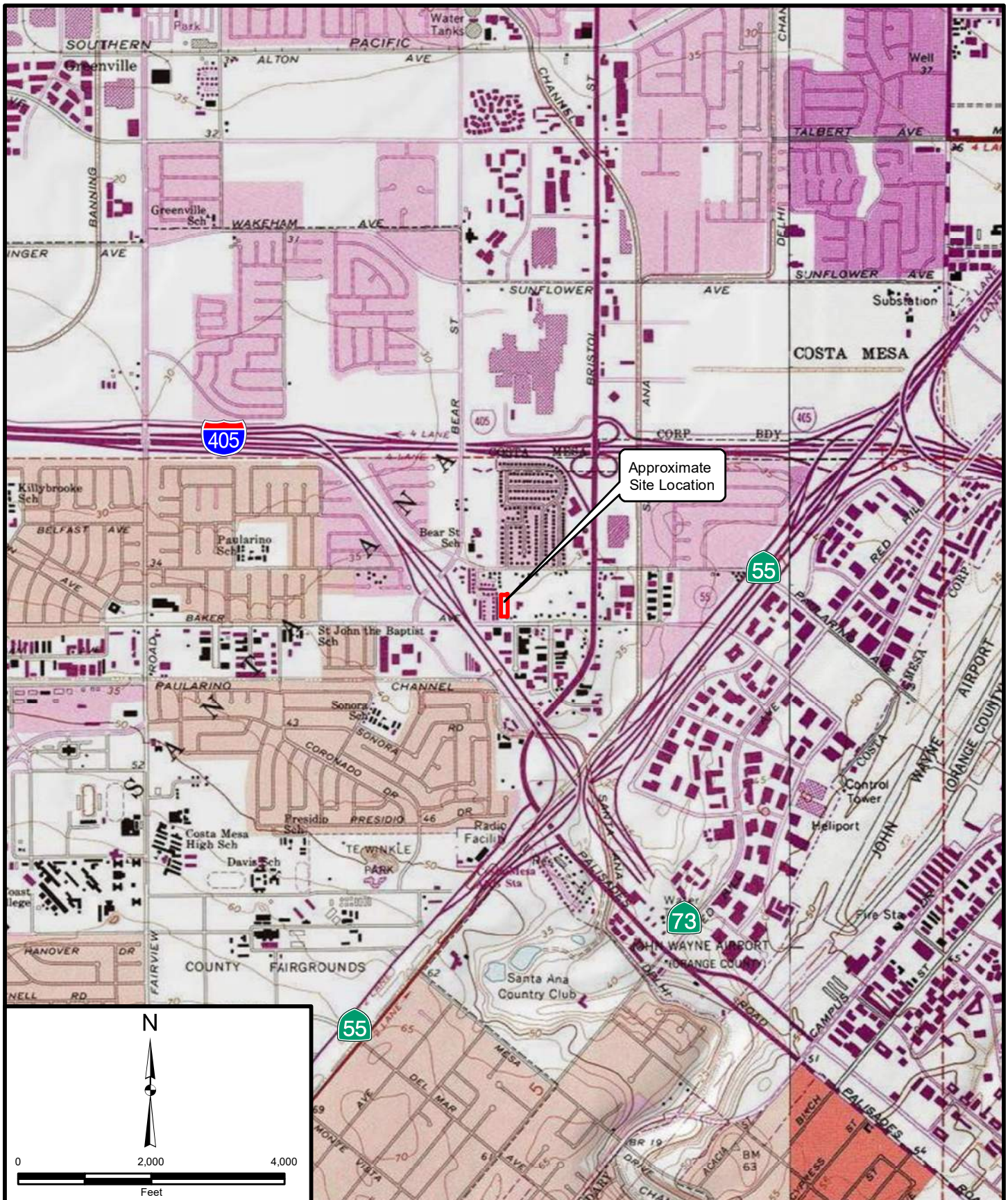
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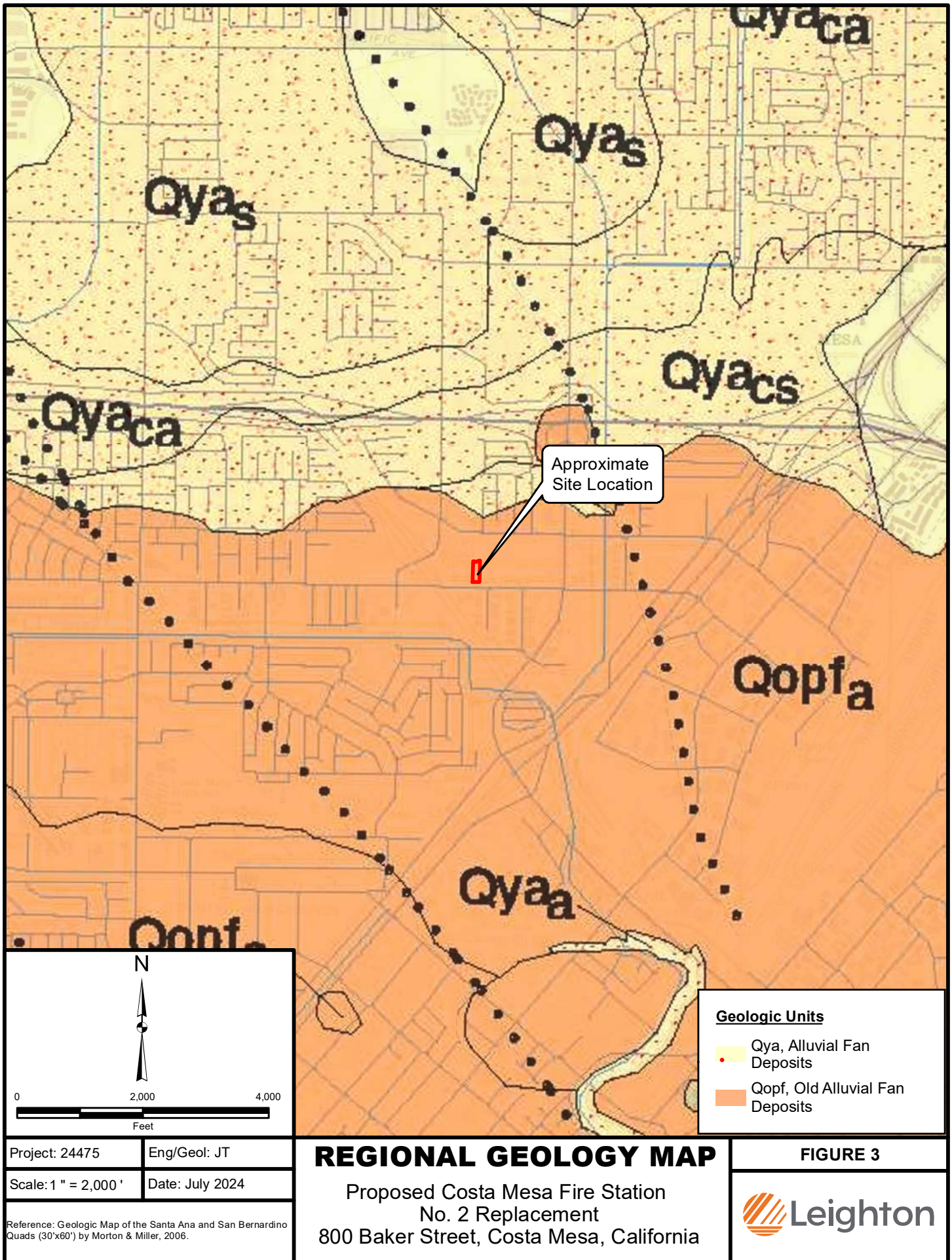
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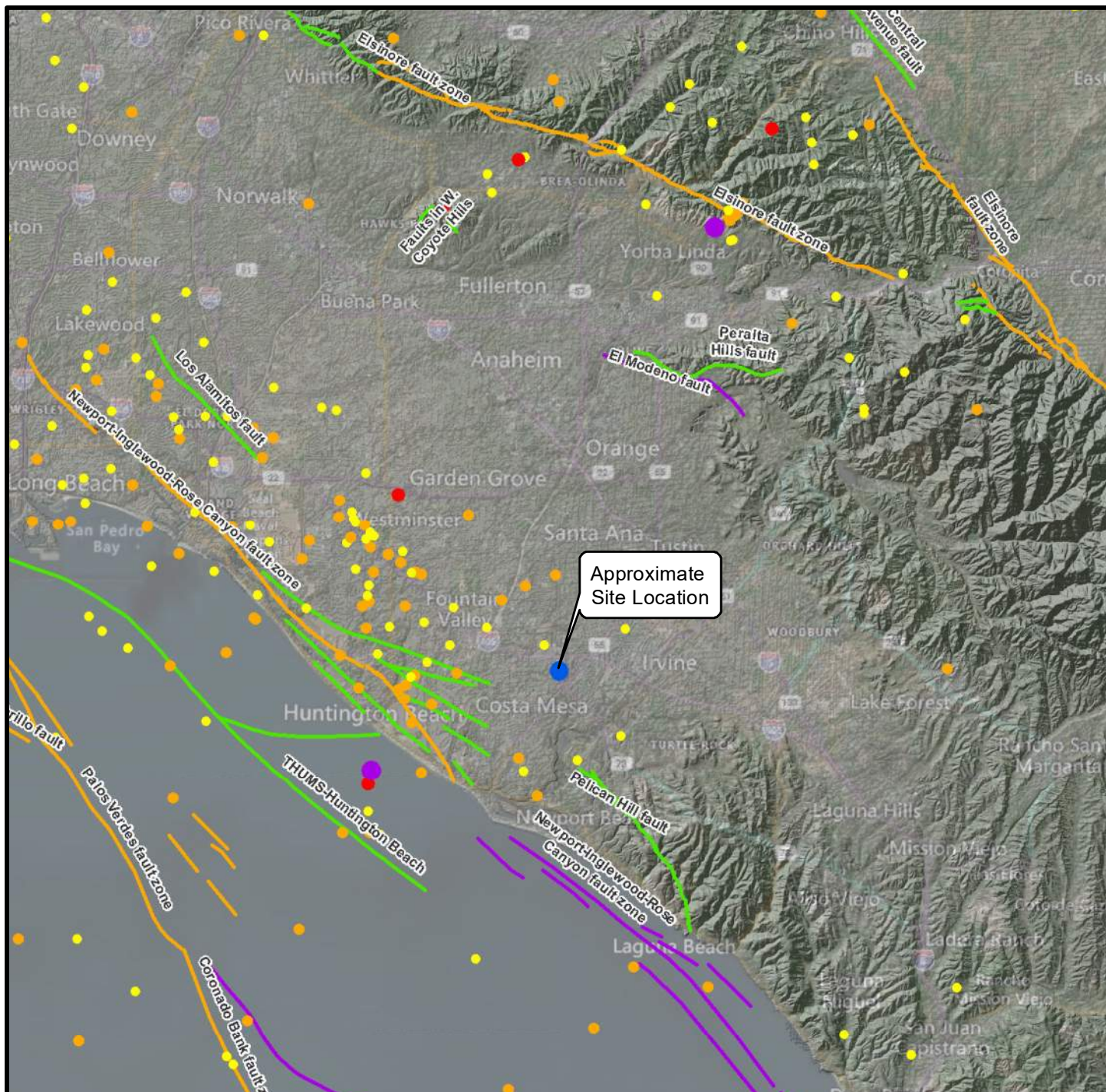
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Project: 24475	Eng/Geol: JT	SITE LOCATION MAP Proposed Costa Mesa Fire Station No. 2 Replacement 800 Baker Street, Costa Mesa, California	FIGURE 1
Scale: 1 " = 2,000 '	Date: July 2024		Leighton
Reference: Copyright:© 2013 National Geographic Society, i-cubed			





Approximate
Site Location

Legend

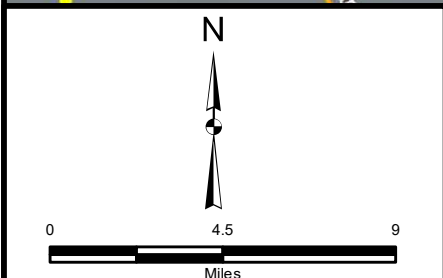
Fault activity

Recency of Movement

- Historic (<200 years)
- Holocene (<11,700 years)
- Late Quaternary (last 700,000 years)
- Quaternary (<1.6M years)

Historical Earthquakes ($\geq M3.5$)

- 3.5 - 3.99
- 4.0 - 4.99
- 5.0 - 5.99
- 6.0 - 6.99



Project: 24475

Eng/Geol: JT

Scale: 1" = 5 miles

Date: July 2024

Basemap Reference: © 2024 Microsoft Corporation
Earthstar Geographics SIO © 2024 TomTom
Seismicity Data Reference: maps.conservation.ca.gov

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Proposed Costa Mesa Fire Station
No. 2 Replacement
800 Baker Street, Costa Mesa, California

FIGURE 4



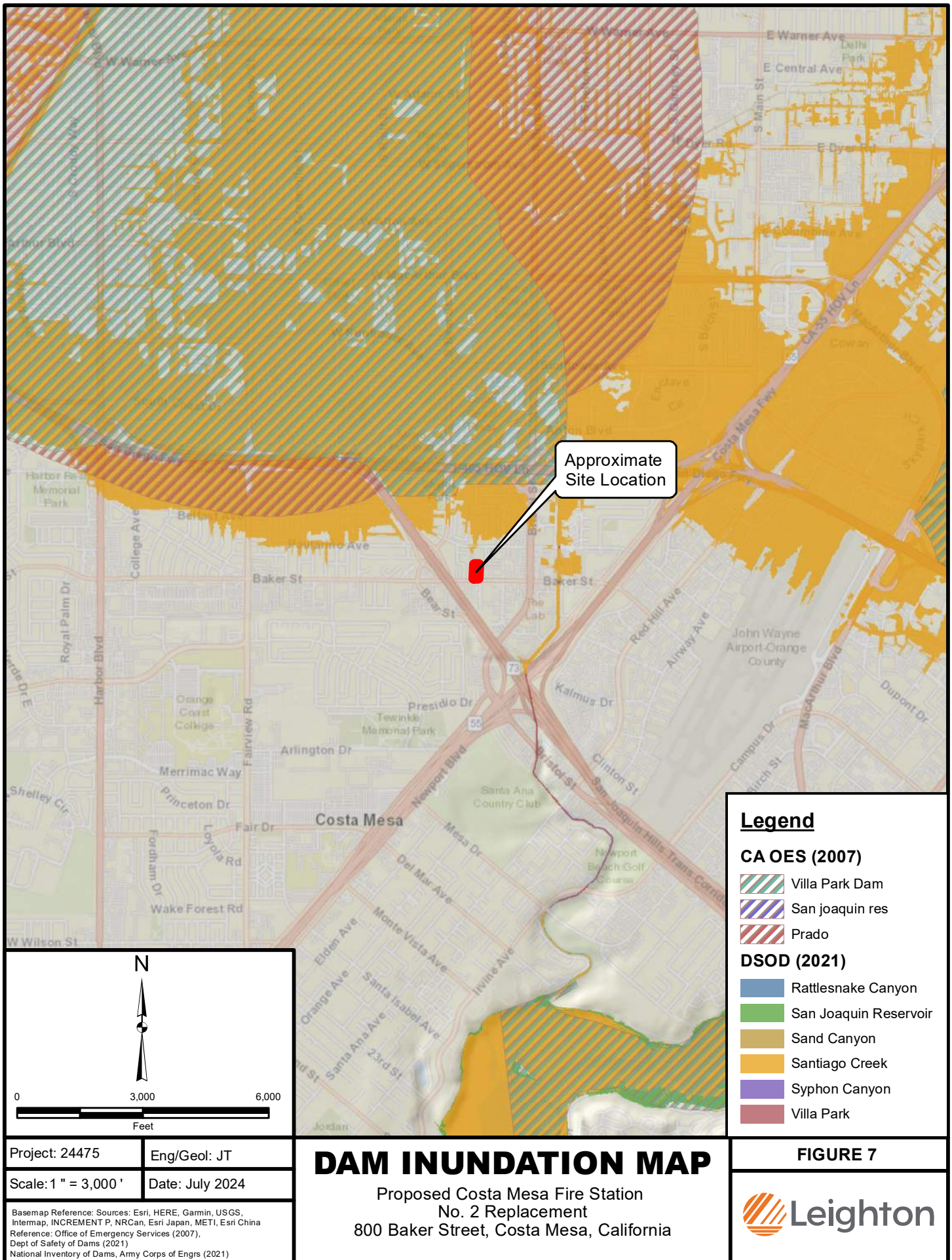


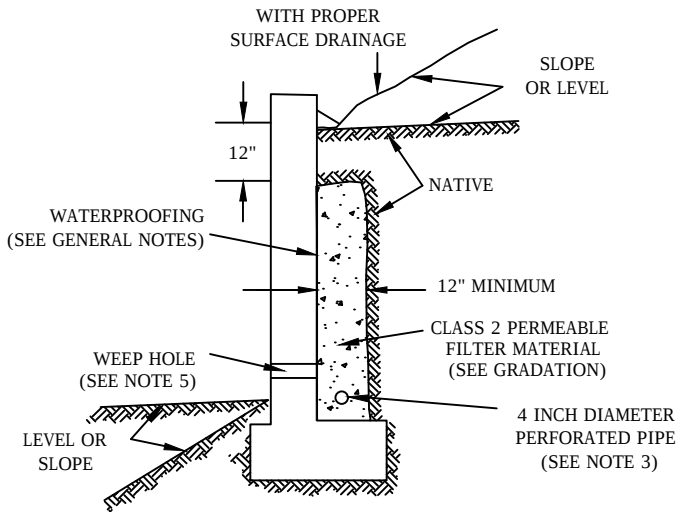
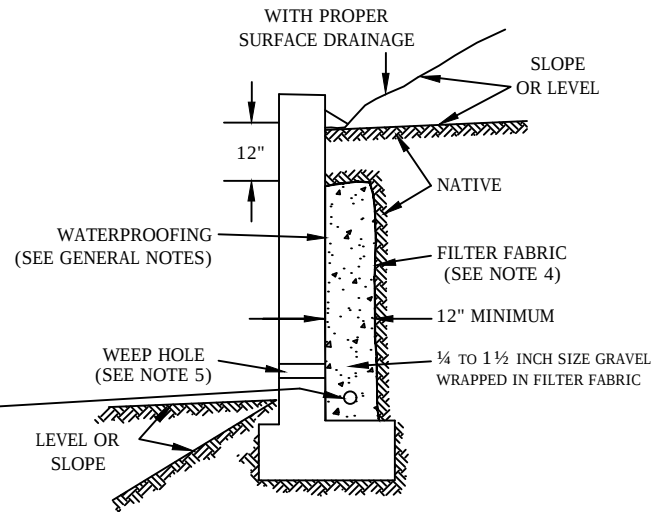
Reference: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community
FEMA (<http://www.fema.gov/index.shtm>), DWR (<http://www.dwr.ca.gov>)

Proposed Costa Mesa Fire Station
No. 2 Replacement
800 Baker Street, Costa Mesa, California



Leighton



SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤ 50 **OPTION 1: PIPE SURROUNDED WITH
CLASS 2 PERMEABLE MATERIAL****OPTION 2: GRAVEL WRAPPED
IN FILTER FABRIC**

Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

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

GENERAL NOTES:

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

Notes:

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

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

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤ 50



Figure 8

APPENDIX A

FIELD EXPLORATION

Our field exploration consisted of geologic reconnaissance and a subsurface exploration program consisting of four (4) geotechnical borings. These subsurface exploration locations are plotted on Figure 2, *Exploration Location Map*, and describe in more detail below:

Hollow Stem Auger Borings: On June 17, 2024, four (4) borings were drilled with a truck-mounted drill rig, logged and sampled to depths ranging from approximately 10 feet to 51.5 feet bgs. After sampling and logging, all borings were immediately backfilled with soil cuttings generated during drilling and asphalt cold patched. Encountered soils were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Near surface bulk soil samples were collected from these borings. Boring logs are included as part of this appendix.

Subsurface Variations and Limitations: These attached subsurface exploration logs and related information depict subsurface conditions only at the approximate locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these locations. Passage of time may result in altered subsurface conditions due to possible environmental changes. In addition, any stratification lines depicted on these logs represent an approximate boundary between soil types, but these transitions can be gradual.

CM-06-04-2026

GEOTECHNICAL BORING LOG LB-1

Project No.	038.0000024475	Date Drilled	6-17-24
Project	PBK Costa Mesa FS No. 2 Replacement GE	Logged By	BTM
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	39'
Location	See Figure 2 - Exploration Location Map	Sampled By	BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									<i>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.</i>	
	0	N S							Artificial Fill, undocumented (Afu)	
				B-1				CL	@Surface: 5-inches asphalt concrete over 5-inches base	SA, MD, EI
				R-1	12 16 24	122	12	CL	@10": SANDY CLAY, reddish brown, moist, mostly fine sand, few medium sand, 60-70% fines (field estimate), low plasticity	
	5			R-2	9 16 25	121	13	CL	@2.5': SANDY CLAY, very stiff, reddish brown, moist, fine sand, some medium sand, 60-70% fines (field estimate), low to medium plasticity, trace asphalt chunks	
	35									
	5			R-3	12 11 12	119	10	CL-SC	Old Alluvial Fan Deposit (Qopf)	CO
	30			R-4	7 9 20	103	17	SM	@7.5': SANDY CLAY to CLAYEY SAND, very stiff to medium dense, reddish brown, moist, fine sand, 45-60% fines (field estimate), low plasticity	
	10			R-5	10 16 23			ML	@10": SILTY SAND, medium dense, yellowish brown, slightly moist, fine sand, 25-35% fines (field estimate), low plasticity, grading slightly coarser toward bottom, 10-20% fines (field estimate)	
	25			R-6	9 16 22			ML	@12.5': SANDY SILT, very stiff, gray brown, slightly moist, very fine sand, 70-80% fines (field estimate), low plasticity	
	15									
	20			S-7	5 7 9			ML	@15': SANDY SILT, very stiff, gray brown, slightly moist, very fine sand, 50-60% fines (field estimate), low plasticity	
	20									
	15									
	25			R-8	19 24 22			ML-SM	@20': SANDY SILT with CLAY, very stiff, grayish brown, moist, fine sand, 55-65% fines (field estimate), low plasticity	
	10								@25': SANDY SILT to SILTY SAND, hard to dense, gray brown, moist to very moist, very fine sand, 45-60% fines (field estimate), low plasticity	
	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



CM-06-04-2026

GEOTECHNICAL BORING LOG LB-1

Project No. 038.0000024475
Project PBK Costa Mesa FS No. 2 Replacement GE
Drilling Co. 2R Drilling, Inc.
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - Exploration Location Map

Date Drilled 6-17-24
Logged By BTM
Hole Diameter 8"
Ground Elevation 39'
Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							<i>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.</i>	
30				S-9	4 5 6		24	CL	@30': LEAN CLAY with SAND, stiff, gray and orange, very moist to saturated, 79% fines (lab), low to medium plasticity, oxidation staining	-200, AL
5				R-10	9 15 22			ML	@35': SANDY SILT, very stiff, light gray, very moist, very fine sand, 70-80% fines (field estimate), low plasticity	
35				S-11	9 27 31			SP-SM	@40': POORLY-GRADED SAND with SILT, very dense, yellowish brown, very moist to saturated, mostly fine sand, few medium to coarse sand, few fines, nonplastic	
40				R-12	50/6"			SP-SM	@45': POORLY-GRADED SAND with SILT, very dense, yellowish brown, very moist to saturated, mostly fine sand, some medium to coarse sand, few fines, nonplastic	
-5				S-13	5 7 10			SP-SM	@50': POORLY-GRADED SAND with SILT, very dense, yellowish brown, very moist to saturated, mostly fine sand, some medium to coarse sand, few fines, nonplastic	
45							27	CL	@51': LEAN CLAY, very stiff, dark gray, very moist, very fine sand, 90% fines (lab), low plasticity, slightly micaceous	-200, AL
-10									Total Depth: 51.5 feet bgs Groundwater encountered at 31 feet bgs, rose to 25 feet after ~10 minutes Boring backfilled with soil cuttings and patched at surface with cold-mix asphalt.	
50										
55										
-15										
60										
-20										

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
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 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



CM-06-04-2026

GEOTECHNICAL BORING LOG LB-2

Project No.	038.0000024475	Date Drilled	6-17-24
Project	PBK Costa Mesa FS No. 2 Replacement GE	Logged By	BTM
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	38'
Location	See Figure 2 - Exploration Location Map	Sampled By	BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>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.</i>	Type of Tests
	0	N S							Artificial Fill, undocumented (Afu) @Surface: 5.5-inches asphalt concrete over 4-inches base	
				B-1				CL	@9.5": SANDY CLAY, reddish brown, moist, fine sand, 55-65% fines (field estimate), low plasticity	RV, CR
35				R-1	8 15 22	120	14	CL	@2.5': SANDY CLAY, very stiff, brown, mottled with lighter brown, slightly moist, fine sand, 65-75% fines (field estimate), low to medium plasticity	
	5			R-2	10 20 32			CL	Old Alluvial Fan Deposit (Qopf) @5': SANDY CLAY, hard, reddish brown, slightly moist, fine sand, few medium sand, 65-75% fines (field estimate), low plasticity, slightly micaceous	
30				R-3	7 10 16	103	21	ML	@7.5': SANDY SILT, very stiff, white to light gray, slightly moist, very fine sand, few medium to coarse sand, 60-75% fines (field estimate), low plasticity; grading to SILTY SAND, light brown, slightly moist, fine sand, 20-30% fines (field estimate)	
10				R-4	9 11 11			SM	@10': SILTY SAND, medium dense, light brown, slightly moist, fine sand, 20-30% fines (field estimate), low plasticity, slightly micaceous; grading to SANDY SILT with CLAY, stiff, gray, slightly moist, fine sand, 60-70% fines (field estimate), low plasticity, slight oxidation	
25				R-5	11 15 20	105	24	ML	@12.5': SANDY SILT with CLAY, very stiff, gray, slightly moist, fine sand, 60-70% fines (field estimate), low plasticity, oxidation staining	
15				S-6	5 10 12			ML	@15': SANDY SILT with CLAY, very stiff, gray, slightly moist, fine sand, 50-60% fines (field estimate), low plasticity, oxidation staining	
20										
	20			R-7	10 20 30			ML-SM	@20': SILTY SAND to SANDY SILT, dense to hard, grayish brown, slightly moist, fine sand, 40-60% fines (field estimate), low plasticity, slightly micaceous	
15										
	25			S-8	4 8 12			ML-SM	@25': SILTY SAND to SANDY SILT, medium dense to very stiff, light brown, moist, fine sand, 40-60% fines (field estimate), low plasticity, slightly micaceous	
10										
	30									

SAMPLE TYPES:

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

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-200 % FINES PASSING
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CM-06-04-2026

GEOTECHNICAL BORING LOG LB-2

Project No.	038.0000024475	Date Drilled	6-17-24
Project	PBK Costa Mesa FS No. 2 Replacement GE	Logged By	BTM
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	38'
Location	See Figure 2 - Exploration Location Map	Sampled By	BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							<i>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.</i>	
	30			R-9	10 14 15			CL	@30': SANDY CLAY, very stiff, brown, very moist to saturated, fine sand, 60-75% fines (field estimate), low to medium plasticity	
5									Total Depth: 31.5 feet bgs Groundwater encountered at 27 feet bgs, rose to 24 feet after ~10 minutes Boring backfilled with soil cuttings and patched at surface with cold-mix asphalt.	
35										
0										
40										
-5										
45										
-10										
50										
-15										
55										
-20										
60										

SAMPLE TYPES:

B BULK SAMPLE

C CORE SAMPLE

G GRAB SAMPLE

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SE SAND EQUIVALENT

SG SPECIFIC GRAVITY

UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-3 / IT-1

Project No. 038.0000024475
 Project PBK Costa Mesa FS No. 2 Replacement GE
 Drilling Co. 2R Drilling, Inc.
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2 - Exploration Location Map

Date Drilled 6-17-24
 Logged By BTM
 Hole Diameter 8"
 Ground Elevation 38'
 Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							<i>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.</i>	
	0								Artificial Fill, undocumented (Afu) @Surface: 5 inches asphalt over 9 inches of old asphalt and base mixture @1.17': SANDY CLAY, reddish brown, moist, fine sand, 65-75% fines (field estimate), medium plasticity	
35				B-1				CL		
	5			S-1	7 10 11			SC	Old Alluvial Fan Deposit (Qopf) @5': CLAYEY SAND, very stiff, reddish brown, moist, fine sand, few medium sand, 35-45% fines (field estimate), low plasticity, micaceous	
30										
10				S-2	5 5 6			SM CL-ML	@10': SILTY SAND, medium dense, light brown, slightly moist, very fine to fine sand, 20-30% fines (field estimate), low plasticity, micaceous @11': Grades quickly to SANDY SILTY CLAY, stiff, light brown to grayish brown, slightly moist, very fine to fine sand, 70% fines (lab), low plasticity, micaceous	-200
25				S-3	5 8 10			CL	@13.5': SANDY LEAN CLAY, very stiff, light brown, moist, very fine sand, 61% fines (lab), low plasticity, little oxidation staining	-200
15										
20									Total Depth: 15 feet bgs No groundwater encountered during drilling Temporary percolation well installed; screened from 10 to 15 feet bgs Upon completion of infiltration testing, boring backfilled with soil cuttings to surface and patched at surface with cold-mix asphalt.	
20										
15										
25										
10										
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



GEOTECHNICAL BORING LOG LB-4 / IT-2

Project No. 038.0000024475
 Project PBK Costa Mesa FS No. 2 Replacement GE
 Drilling Co. 2R Drilling, Inc.
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2 - Exploration Location Map

Date Drilled 6-17-24
 Logged By BTM
 Hole Diameter 8"
 Ground Elevation 37'
 Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>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.</i>	
	35							CL	Artificial Fill, undocumented (Afu) @Surface: 5-inches asphalt concrete over 5-inches base @10": SANDY CLAY, reddish brown, moist, fine sand, few medium sand, 60-70% fines (field estimate), medium plasticity	
	5			S-1	6 7 8			SC-SM	Old Alluvial Fan Deposit (Qopf) @5": SILTY CLAYEY SAND, medium dense, reddish brown, moist, fine to medium sand, 48% fines (lab), low plasticity, micaceous	-200
	30			S-2	8 11 14			SP-SM	@8.5': POORLY-GRADED SAND with SILT, medium dense, brown to reddish brown, moist, fine to medium sand, trace coarse sand, 6% fines (lab), low plasticity, slightly micaceous	-200
	10									
	25									
	15									
	20									
	20									
	15									
	25									
	10									
	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										



[illegible]

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of physical and mechanical properties of soils underlying proposed improvements, and to aid in verifying soil classification.

In-Situ Moisture and Density: As-sampled soil moisture content was measured (ASTM D 2216) on selected samples recovered from our borings. In addition, in place dry density was measured (ASTM D 2937) on selected relatively undisturbed soil samples. Results of these tests are shown on our logs at the appropriate sample depths in Appendix A.

Percent Passing No. 200 Sieve: Percent fines (silt and clay) passing the No. 200 U.S. Standard Sieve was determined for soil samples in accordance with ASTM D 1140 Standard Test Method. Samples were dried and passed through a No. 4 sieve, then a No. 200 sieve. Result of this grain size analysis, as percent by dry weight passing the No. 200 U.S. Standard Sieve, is tabulated in this appendix and entered on our boring logs.

Modified Proctor Compaction Curve: A laboratory modified Proctor compaction curve (ASTM D1557) was established for bulk soil-sample to evaluate the modified Proctor laboratory maximum dry density and optimum moisture content. Results of this test are presented on the following *Modified Proctor Compaction Test* sheet in this appendix.

Corrosivity Tests: To evaluate corrosion potential of subsurface soils at the site, we tested a bulk soil sample collected during our subsurface exploration for pH, electrical resistivity (CTM 532/643), soluble sulfate content (CTM 417 Part II) and soluble chloride content (CTM 422) testing. Results of these tests are enclosed at the end of this appendix.

Expansion Index (EI): An Expansion Index (EI) test was performed on a representative shallow bulk soil sample from this site, in general accordance with the ASTM D4829 Standard Test Method.

Atterberg Limits (AL): Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) were determined for soil samples in accordance with ASTM D4318 Standard Test Method. Soil

samples were air-dried and passed through a No. 40 sieve, then re-moisturized. Liquid Limit and Plastic Limit tests were performed on fraction of soil samples passing the No. 40 sieve. Results of these tests are presented on the “*Atterberg Limits*” sheet in this appendix.

Swell or Collapse of Soils (CO): Swell or collapse of soil tests were performed on relatively-undisturbed ring-lined drive-sampler soil samples, to measure the magnitude of one-dimensional wetting-induced swell or collapse on unsaturated soils. Results are presented in this appendix on the *One-Dimensional Swell or Collapse of Soils* (ASTM D 4546) sheets.

Resistance Value (R-Value): R-Value for a shallow bulk soil sample was established by California Test Method 301 to assist in preliminary pavement design recommendations. R-Value results are presented in this appendix on the *R-Value Test Results* sheets.



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: PBK Costa Mesa FS No. 2 Replacement Tested By: M. Saba Date: 07/02/24
 Project No.: 038.0000024475 Checked By: A. Santos Date: 07/05/24
 Boring No.: LB-1 Depth (ft.): 2-5
 Sample No.: B-1
 Soil Identification: Yellowish brown sandy lean clay s(CL)

Preparation Method:



Moist
Dry



Mechanical Ram
Manual Ram

Mold Volume (ft³)0.03320

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	<u>3731</u>	<u>3905</u>	<u>3909</u>			
Weight of Mold (g)	<u>1780</u>	1780	1780			
Net Weight of Soil (g)	1951	2125	2129			
Wet Weight of Soil + Cont. (g)	<u>864.3</u>	<u>959.3</u>	<u>928.2</u>			
Dry Weight of Soil + Cont. (g)	<u>832.6</u>	<u>900.8</u>	<u>852.3</u>			
Weight of Container (g)	<u>75.6</u>	<u>76.5</u>	<u>79.2</u>			
Moisture Content (%)	4.19	7.10	9.82			
Wet Density (pcf)	129.6	141.1	141.4			
Dry Density (pcf)	124.3	131.8	128.7			

Maximum Dry Density (pcf)

131.9

Optimum Moisture Content (%)

7.6

PROCEDURE USED

☒ Procedure A

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

☐ Procedure B

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

☐ Procedure C

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

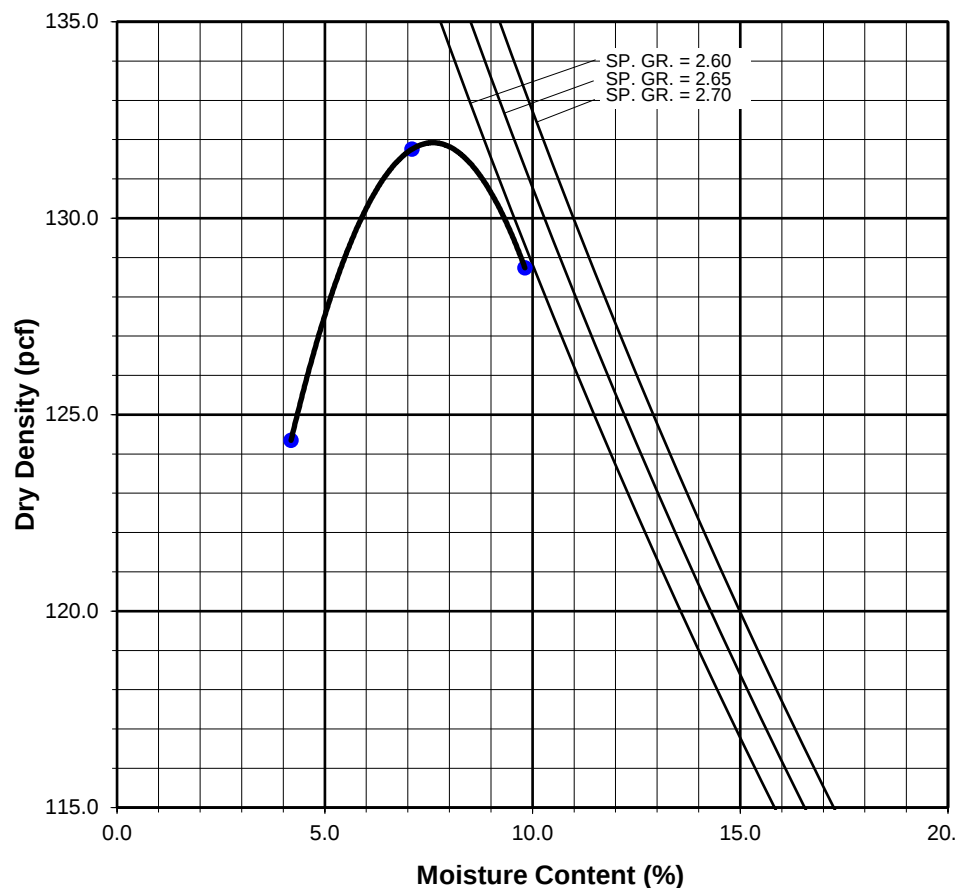
Particle-Size Distribution:

1:46:53

GR:SA:FI

Atterberg Limits:

LL, PL, PI





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

ASTM D6913

Project Name: PBK Costa Mesa FS No. 2 ReplacementTested By: G. Bathala Date: 06/26/24Project No.: 038.0000024475Checked By: J. Ward Date: 07/09/24Boring No.: LB-1Depth (feet): 2-5Sample No.: B-1Soil Identification: Yellowish brown sandy lean clay s(CL)

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	CP-15	WR	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	2406.8	721.9	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	223.1	236.6	Wt. of Container No. (g)	1.0	1.0
Dry Wt. of Soil (g)	2183.7	485.3	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve	Container No.	WR
	Wt. of Dry Soil + Container (g)	472.0
	Wt. of Container (g)	236.6
	Dry Wt. of Soil Retained on # 200 Sieve (g)	235.4

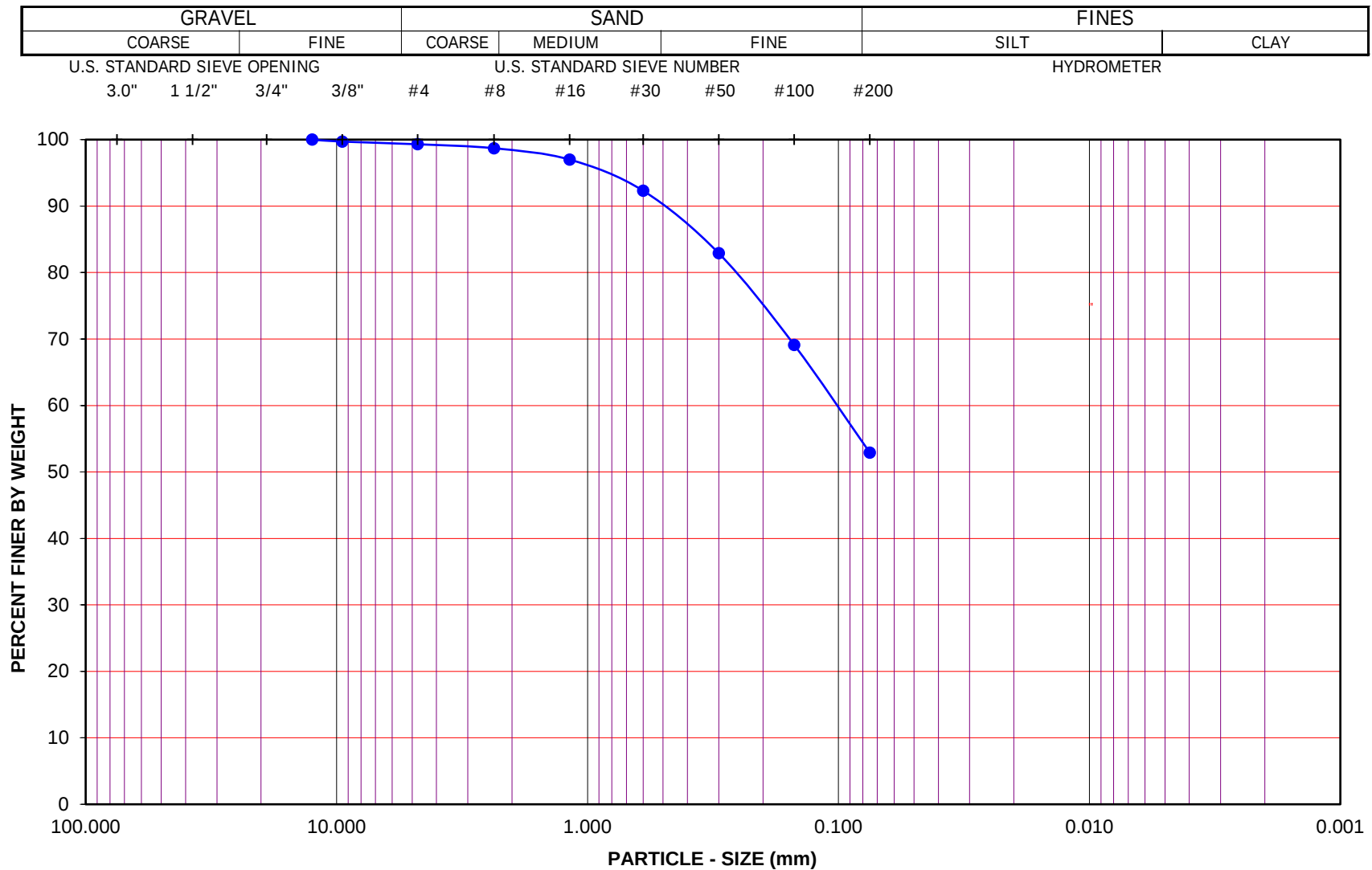
U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
3"	75.0			
1 1/2"	37.5			
1"	25.0			
3/4"	19.0			
1/2"	12.5	0.0		100.0
3/8"	9.5	6.6		99.7
#4	4.75	15.4		99.3
#8	2.36		3.0	98.7
#16	1.18		11.3	97.0
#30	0.600		34.4	92.3
#50	0.300		80.0	82.9
#100	0.150		147.6	69.1
#200	0.075		227.0	52.9
PAN				

GRAVEL: **1 %**SAND: **46 %**FINES: **53 %**GROUP SYMBOL: **s(CL)**

Cu = D60/D10 = _____

Cc = (D30)²/(D60*D10) = _____

Remarks: _____



Project Name: PBK Costa Mesa FS No. 2 Replacement

Project No.: 038.0000024475

Boring No.: LB-1

Sample No.: B-1

Depth (feet): 2-5

Soil Type : s(CL)

Soil Identification: Yellowish brown sandy lean clay s(CL)

GR:SA:FI : (%) **1 : 46 : 53**



**PARTICLE - SIZE
DISTRIBUTION
ASTM D 6913**

Jul-24

Boring No.	LB-1	LB-1	LB-3/IT-1	LB-3/IT-1	LB-4/IT-2	LB-4/IT-2		
Sample No.	S-9	S-13B	S-2B	S-3	S-1	S-2		
Depth (ft.)	30.0	51.0	11.0	13.5	5.0	8.5		
Sample Type	SPT	SPT	SPT	SPT	SPT	SPT		
Soil Identification	Light olive brown lean clay with sand (CL)s	Olive gray lean clay (CL)	Olive gray sandy silty clay s(CL-ML), caliche noted	Olive gray sandy lean clay s(CL)	Yellowish brown silty, clayey sand (SC-SM)	Yellowish brown poorly-graded sand with silt (SP-SM)		
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00		
Dry Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00		
Weight of Container (g)	1.00	1.00	1.00	1.00	1.00	1.00		
Moisture Content (%)	0.00	0.00	0.00	0.00	0.00	0.00		
Sample Dry Weight Determination								
Weight of Sample + Container (g)	418.47	424.61	351.20	427.67	423.20	657.39		
Weight of Container (g)	77.06	82.49	75.46	74.50	87.76	76.67		
Weight of Dry Sample (g)	341.41	342.12	275.74	353.17	335.44	580.72		
Container No.:								
After Wash								
Method (A or B)	B	B	B	B	B	A		
Dry Weight of Sample + Cont. (g)	148.34	116.45	157.74	213.80	262.40	621.02		
Weight of Container (g)	77.06	82.49	75.46	74.50	87.76	76.67		
Dry Weight of Sample (g)	71.28	33.96	82.28	139.30	174.64	544.35		
% Passing No. 200 Sieve	79.1	90.1	70.2	60.6	47.9	6.3		
% Retained No. 200 Sieve	20.9	9.9	29.8	39.4	52.1	93.7		
	PERCENT PASSING No. 200 SIEVE ASTM D 1140				Project Name: <u>PBK Costa Mesa FS No. 2 Replacement</u>			
					Project No.: <u>038.0000024475</u>			
					Tested By: <u>G. Bathala</u> Date: <u>06/28/24</u>			



EXPANSION INDEX of SOILS

ASTM D 4829

Project Name: PBK Costa Mesa FS No. 2 Replacement Tested By: G. Berdy Date: 06/27/24
 Project No.: 038.0000024475 Checked By: J. Ward Date: 07/09/24
 Boring No.: LB-1 Depth (ft.): 2-5
 Sample No.: B-1
 Soil Identification: Yellowish brown sandy lean clay s(CL)

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	1.0080
Wt. Comp. Soil + Mold (g)	621.20	456.52
Wt. of Mold (g)	190.00	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (g)	730.00	646.52
Dry Wt. of Soil + Cont. (g)	679.10	591.13
Wt. of Container (g)	0.00	190.00
Moisture Content (%)	7.50	13.81
Wet Density (pcf)	130.1	136.6
Dry Density (pcf)	121.0	120.0
Void Ratio	0.393	0.404
Total Porosity	0.282	0.288
Pore Volume (cc)	58.4	60.1
Degree of Saturation (%) [S meas]	51.5	92.2

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

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
06/27/24	12:24	1.0	0	0.5315
06/27/24	12:34	1.0	10	0.5310
Add Distilled Water to the Specimen				
06/27/24	13:32	1.0	58	0.5365
06/28/24	6:08	1.0	1054	0.5395
06/28/24	9:29	1.0	1255	0.5395

Expansion Index (EI _{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	8
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ATTERBERG LIMITS

ASTM D 4318

Project Name: PBK Costa Mesa FS No. 2 Replacement Tested By: J. Domingo Date: 07/02/24
 Project No. : 038.0000024475 Input By: G. Bathala Date: 07/05/24
 Boring No.: LB-1 Checked By: J. Ward
 Sample No.: S-9 Depth (ft.) 30.0
 Soil Identification: Light olive brown lean clay with sand (CL)s

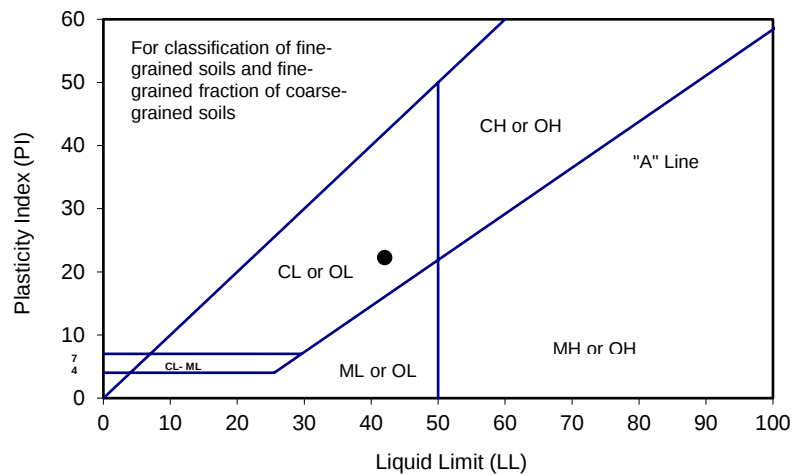
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	24	15	
Wet Wt. of Soil + Cont. (g)	9.83	9.91	20.39	19.65	20.88	
Dry Wt. of Soil + Cont. (g)	8.39	8.45	14.82	14.09	14.71	
Wt. of Container (g)	1.10	1.06	1.10	1.07	1.13	
Moisture Content (%) [Wn]	19.75	19.76	40.60	42.70	45.43	

Liquid Limit	42
Plastic Limit	20
Plasticity Index	22
Classification	CL

PI at "A" - Line = $0.73(LL-20)$ 16.06

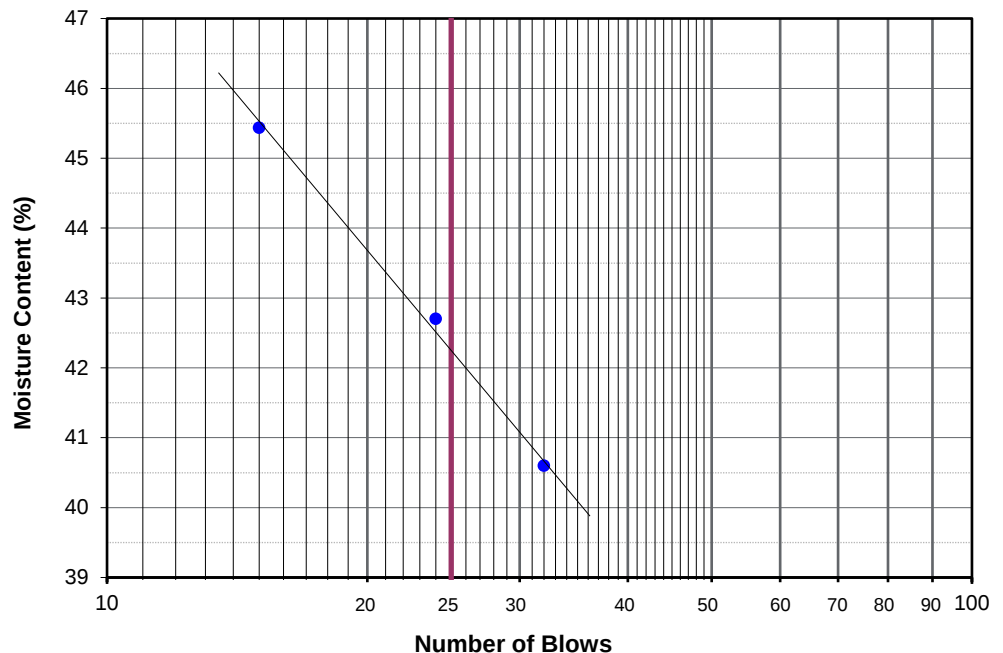
One - Point Liquid Limit Calculation

$$LL = Wn(N/25)^{0.12}$$



PROCEDURES USED

- ☐ Wet Preparation
 Multipoint - Wet
☒ Dry Preparation
 Multipoint - Dry
☒ Procedure A
 Multipoint Test
☐ Procedure B
 One-point Test





ATTERBERG LIMITS

ASTM D 4318

Project Name: PBK Costa Mesa FS No. 2 Replacement Tested By: J. Domingo Date: 07/06/24
 Project No. : 038.0000024475 Input By: JD/JHW Date: 07/08/24
 Boring No.: LB-1 Checked By: J. Ward
 Sample No.: S-13B Depth (ft.) 51.0
 Soil Identification: Olive gray lean clay (CL)

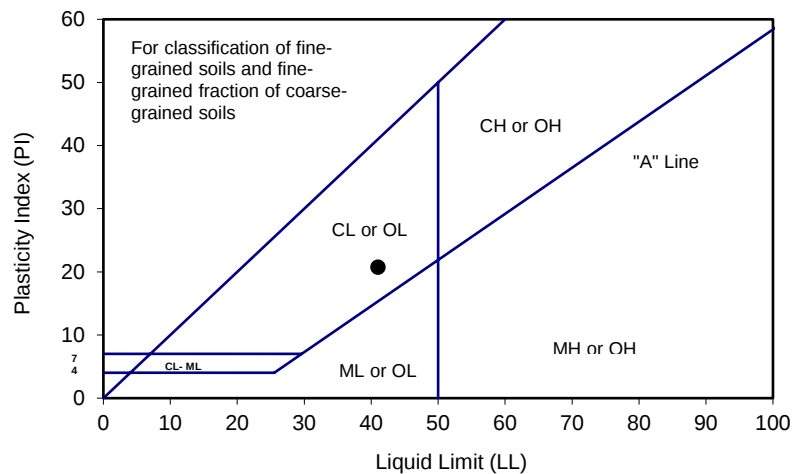
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			31	25	17	
Wet Wt. of Soil + Cont. (g)	9.40	9.38	21.45	21.84	21.50	
Dry Wt. of Soil + Cont. (g)	8.00	7.98	15.64	15.79	15.31	
Wt. of Container (g)	1.06	1.08	1.08	1.11	1.08	
Moisture Content (%) [Wn]	20.17	20.29	39.90	41.21	43.50	

Liquid Limit	41
Plastic Limit	20
Plasticity Index	21
Classification	CL

PI at "A" - Line = $0.73(LL-20)$ 15.33

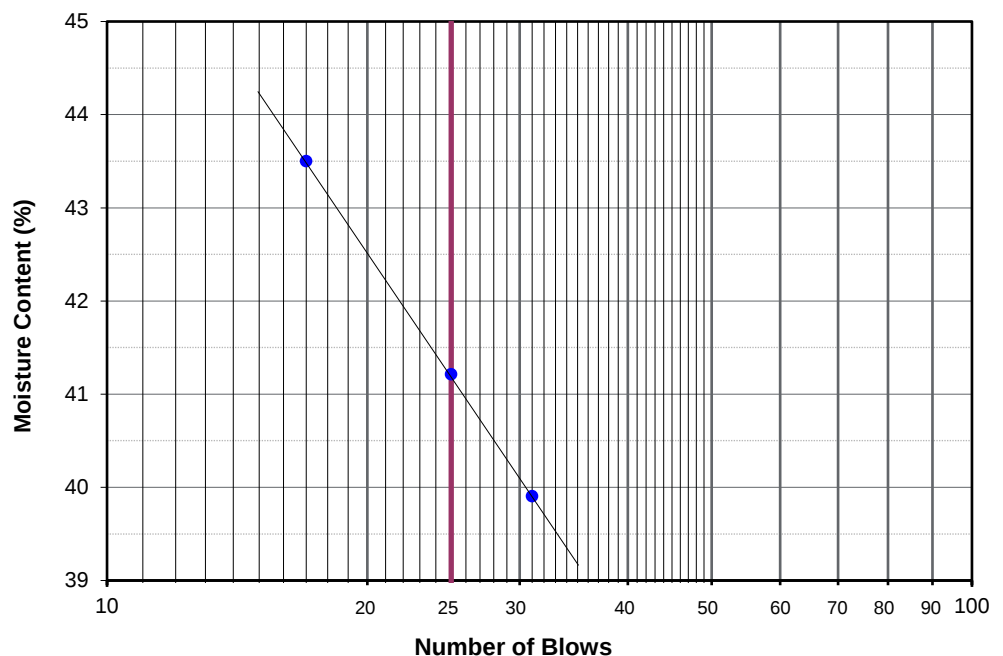
One - Point Liquid Limit Calculation

$$LL = W_n(N/25)^{0.12}$$



PROCEDURES USED

- ☐ Wet Preparation
 Multipoint - Wet
☒ Dry Preparation
 Multipoint - Dry
☒ Procedure A
 Multipoint Test
☐ Procedure B
 One-point Test





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

Project Name: PBK Costa Mesa FS No. 2 Replacement Tested By: G. Bathala Date: 06/26/24
 Project No.: 038.0000024475 Checked By: J. Ward Date: 07/09/24
 Boring No.: LB-1 Sample Type: Ring
 Sample No.: R-3 Depth (ft.): 7.5
 Sample Description: Yellowish brown lean clay (CL)

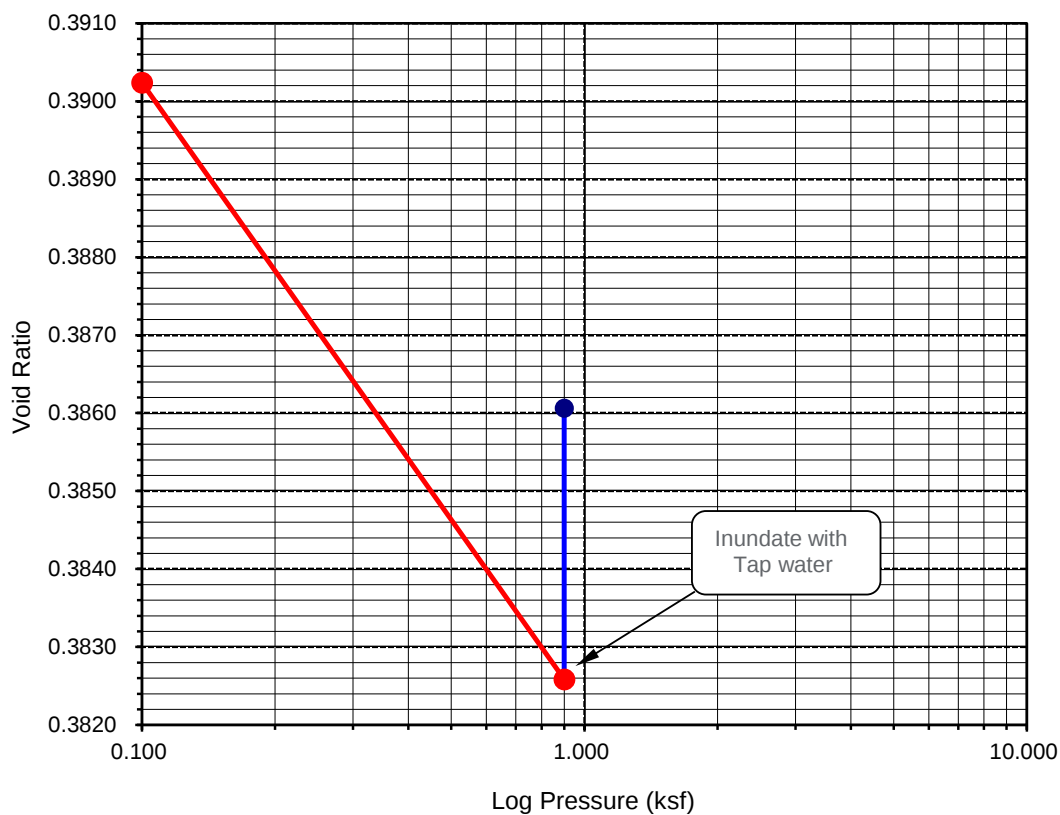
Initial Dry Density (pcf):	121.1
Initial Moisture (%):	10.30
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0276
Diameter(in):	2.415

Final Dry Density (pcf):	121.9
Final Moisture (%) :	15.5
Initial Void ratio:	0.3914
Specific Gravity(assumed):	2.70
Initial Saturation (%)	71.0

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.0284	0.9992	0.00	-0.08	0.3902	-0.08
0.900	0.0352	0.9924	0.13	-0.76	0.3826	-0.63
H2O	0.0327	0.9949	0.13	-0.51	0.3861	-0.38

Percent Swell (+) / Settlement (-) After Inundation = **0.25**

Void Ratio - Log Pressure Curve





**TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS**

Project Name: PBK Costa Mesa FS No. 2 Replacement Tested By : G. Berdy Date: 06/26/24
 Project No. : 038.0000024475 Checked By: J. Ward Date: 07/09/24

Boring No.	LB-2			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Brown CL			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.74			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	15			
Crucible No.	302/303			
Furnace Temperature (°C)	860			
Time In / Time Out	7:30/8:15			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	122.9876			
Wt. of Crucible (g)	122.9852			
Wt. of Residue (g) (A)	0.0024			
PPM of Sulfate (A) x 41150	98.76			
PPM of Sulfate, Dry Weight Basis	99			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO ₃ Soln. Used in Titration (C)	0.8			
PPM of Chloride (C - 0.2) * 100 * 30 / B	120			
PPM of Chloride, Dry Wt. Basis	120			

pH TEST, DOT California Test 643

pH Value	9.10			
Temperature °C	21.7			



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: PBK Costa Mesa FS No. 2 Replacement

Tested By : G. Berdy Date: 06/27/24

Project No. : 038.0000024475

Checked By: J. Ward Date: 07/09/24

Boring No.: LB-2

Depth (ft.) : 0-5

Sample No. : B-1

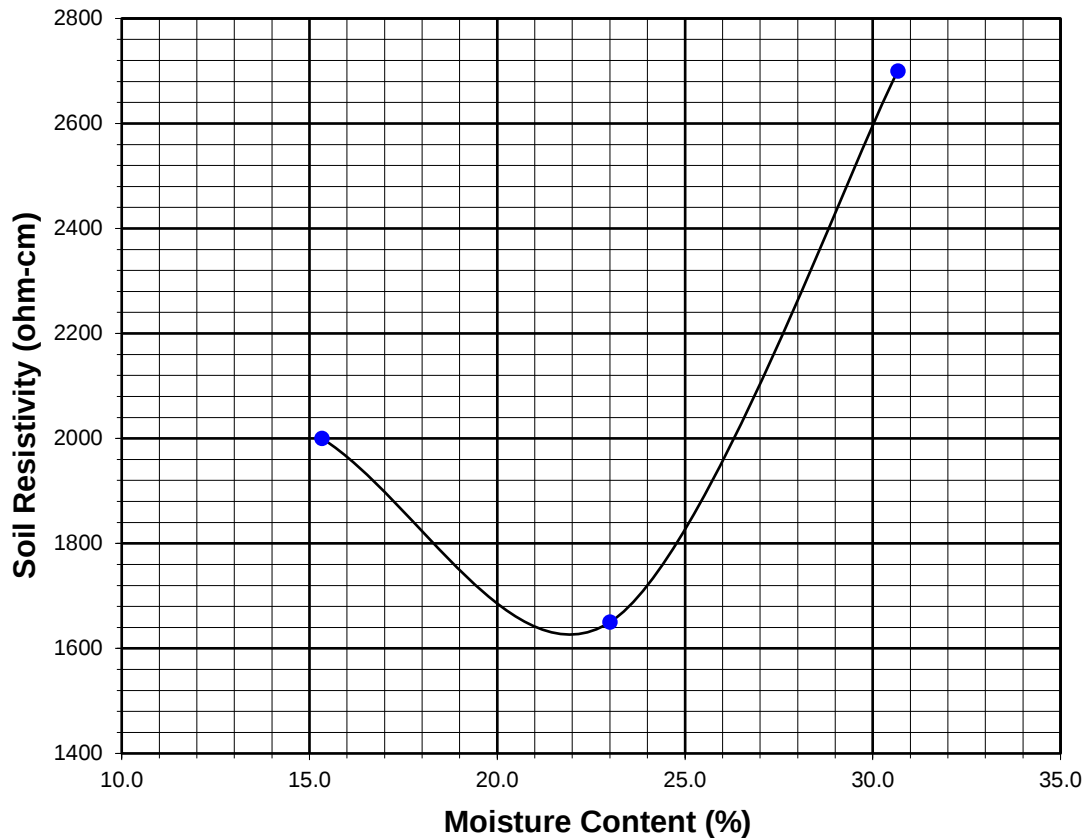
Soil Identification:* Brown CL

*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	15.34	2000	2000
2	30	23.00	1650	1650
3	40	30.67	2700	2700
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)	130.42
Box Constant	1.000
$MC = (((1 + Mci/100) \times (Wa/Wt + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1620	22.0	99	120	9.10	21.7





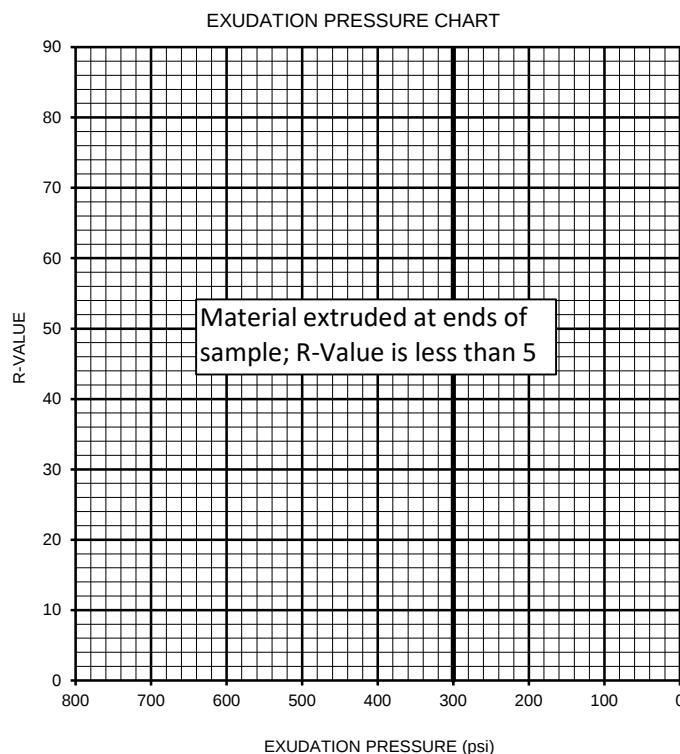
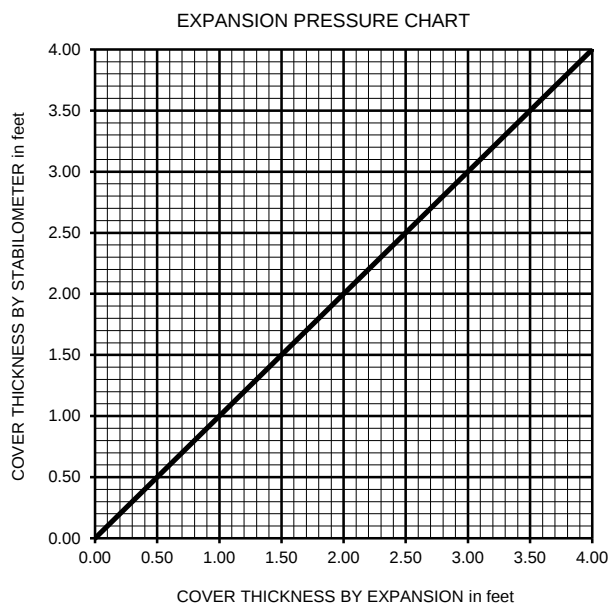
R-VALUE TEST RESULTS

DOT CA Test 301

PROJECT NAME:	<u>PBK Costa Mesa FS No.2 Replacement</u>	PROJECT NUMBER:	<u>038.0000024475</u>
BORING NUMBER:	<u>LB-2</u>	DEPTH (FT.):	<u>0 - 5.0</u>
SAMPLE NUMBER:	<u>B-1</u>	TECHNICIAN:	<u>F. Mina</u>
SAMPLE DESCRIPTION:	<u>Brown lean clay (CL)</u>	DATE COMPLETED:	<u>7/2/2024</u>

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %			
HEIGHT OF SAMPLE, Inches			
DRY DENSITY, pcf			
COMPACTOR PRESSURE, psi			
EXUDATION PRESSURE, psi			
EXPANSION, Inches x 10exp-4			
STABILITY Ph 2,000 lbs (160 psi)			
TURNS DISPLACEMENT			
R-VALUE UNCORRECTED	N/A	N/A	N/A
R-VALUE CORRECTED	N/A	N/A	N/A

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.	N/A	N/A	N/A
EXPANSION PRESSURE THICKNESS, ft.	N/A	N/A	N/A



R-VALUE BY EXPANSION:	<u>N/A</u>
R-VALUE BY EXUDATION:	<u>N/A</u>
EQUILIBRIUM R-VALUE:	<u>< 5</u>

APPENDIX C

SEISMIC

Determination of Site Class and Estimation of Shear Wave Velocity

Project: 24475 Costa Mesa FS No. 2

Depth (ft)	di, Layer Thick (ft)	Field Blow Counts, Ni Corrected for Cs and sampler type Blows per foot (bpf)		Average Ni (bpf)	Ni Hammer Corr:	di / Ni
		LB-1	LB-2		1.3	
5	7.5	25	31	28	36	0.21
10	5	17	13	15	20	0.26
15	5	23	22	23	29	0.17
20	5	16	30	23	30	0.17
25	5	28	20	24	31	0.16
30	5	11	17	14	18	0.27
35	5	22		22	29	0.17
40	5	58		58	75	0.07
45	5	60		60	78	0.06
50	7.5	17		17	22	0.34
60	10	17	*Assumed based on blowcount at 50'	17	22	0.45
70	10	17		17	22	0.45
80	10	17		17	22	0.45
90	10	17		17	22	0.45
100	5	17		17	22	0.23
Summation	100					3.92
Navg = Sum(di) / Sum(di / Ni) =						26

Extract of ASCE 7-16 Table 20.3-1 Site Classification (2019 CBC 1613A.2.2):

Site Class	Soil Profile Name	Avg. N upper 100'		Vs30 (ft/sec)		Vs30 (m/s)		Site Avg N	Interpolated vs30 (ft/s)
		from	to	from	to	from	to		
A	Hard Rock	-		5000	10000	1524	3048	26	781
B	Rock	-		2500	5000	762	1524		
C	VD soil & soft rock	50.001	100	1200	2500	366	762		
D	Stiff Soil	15	50	600	1200	183	366		
E	Soft Soil	0	14.999	0	600	0	183		
F		-	-			0	0		

SITE CLASS, Table 20.3-1: **D****Estimation of Average Shear Wave Velocity in upper 100 ft (Vs30):**

	ft/s	m/s
Approx. Vs30 (interpolation of Table 20.3-1) =	781	238
Approx. Vs30 sands (Imai and Tonouchi, 1982) =	968	295
Approx. Vs30 sands (Sykora and Stokoe, 1983) =	839	256
Approx. Vs30 (Maheswari, Boominathan, Dodagoudar, 2009) =	797	243



Latitude, Longitude: 33.6806, -117.8899



Date	7/11/2024, 2:23:11 PM
Design Code Reference Document	ASCE7-16
Risk Category	IV
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.301	MCE_R ground motion. (for 0.2 second period)
S_1	0.466	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.301	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.867	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	0.558	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.614	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.301	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.412	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.159	Factored deterministic acceleration value. (0.2 second)
S1RT	0.466	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.504	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.746	Factored deterministic acceleration value. (1.0 second)
PGAd	0.888	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.558	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.922	Mapped value of the risk coefficient at short periods
C_{R1}	0.925	Mapped value of the risk coefficient at a period of 1 s
C_v	1.36	Vertical coefficient

DISCLAIMER

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Unified Hazard Tool



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

Please also see the new [USGS Earthquake Hazard Toolbox](#) for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

^ Input

Edition

Dynamic: Conterminous U.S. 2014 (u...

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.6806

Time Horizon

Return period in years

2475

Longitude

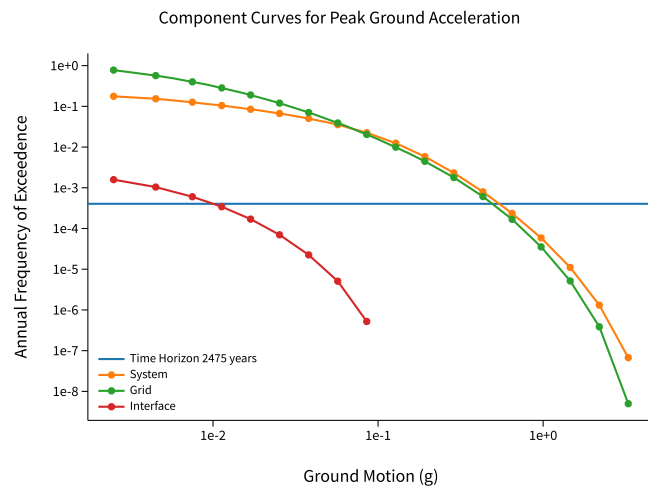
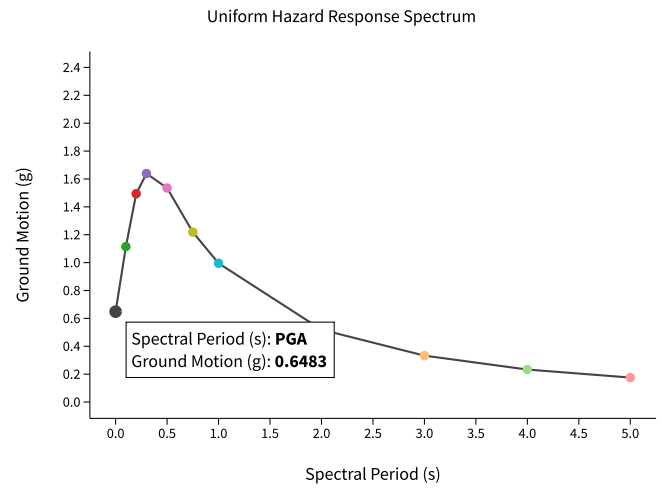
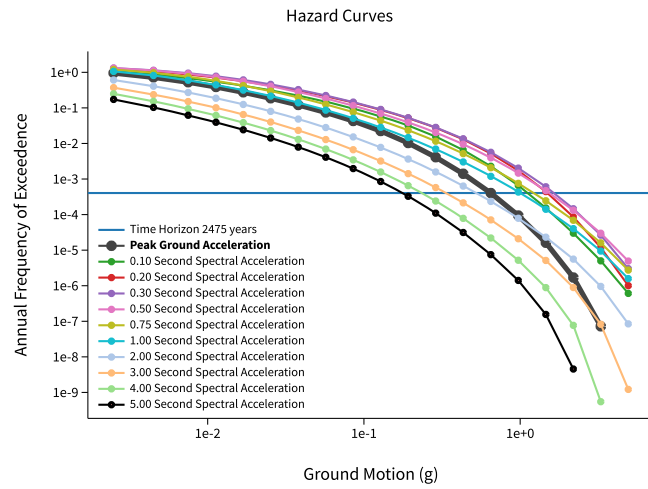
Decimal degrees, negative values for western longitudes

-117.8899

Site Class

259 m/s (Site class D)

^ Hazard Curve

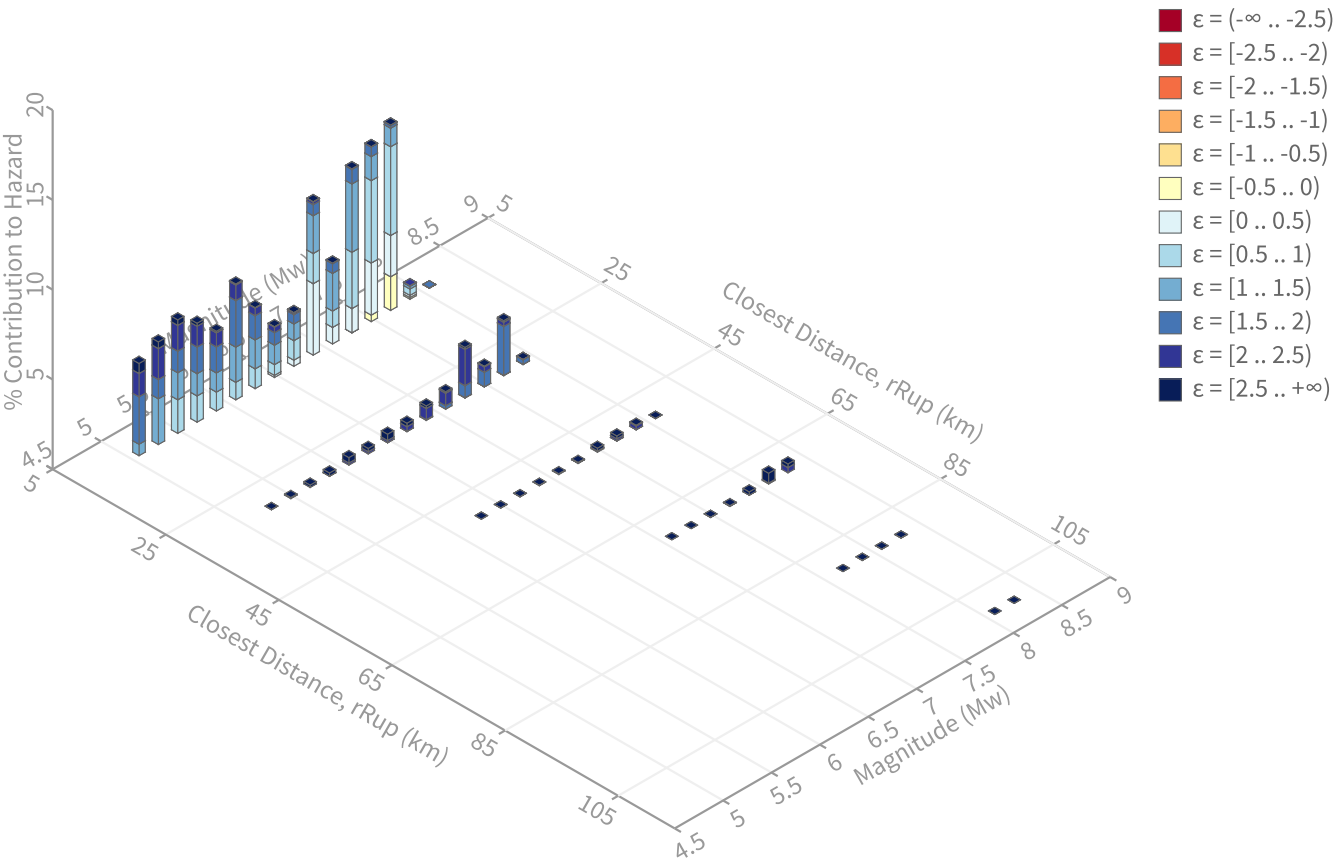


[View Raw Data](#)

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr⁻¹
PGA ground motion: 0.64830776 g

Recovered targets

Return period: 2964.6396 yrs
Exceedance rate: 0.00033730913 yr⁻¹

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.07 %

Mean (over all sources)

m: 6.67
r: 11.46 km
ε₀: 1.28 σ

Mode (largest m-r bin)

m: 7.69
r: 6.58 km
ε₀: 0.54 σ
Contribution: 10.32 %

Mode (largest m-r-ε₀ bin)

m: 7.68
r: 7.37 km
ε₀: 0.67 σ
Contribution: 4.94 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set	Source	Type	r	m	ϵ_0	lon	lat	az	%
UC33brAvg_FM32		System							30.77
	San Joaquin Hills [0]		3.22	7.14	0.44	117.895°W	33.672°N	204.03	10.90
	Newport-Inglewood alt 2 [0]		7.87	7.49	0.84	117.956°W	33.638°N	232.07	5.82
	Compton [0]		15.29	7.35	1.13	118.043°W	33.702°N	279.77	3.47
	Palos Verdes [6]		25.96	7.46	2.00	118.134°W	33.567°N	240.77	1.78
	Newport-Inglewood (Offshore) [0]		10.27	6.59	1.56	117.915°W	33.591°N	192.91	1.20
	Whittier alt 2 [2]		26.88	7.65	1.89	117.746°W	33.891°N	29.58	1.07
UC33brAvg_FM31		System							27.19
	San Joaquin Hills [0]		3.22	7.52	0.34	117.895°W	33.672°N	204.03	7.58
	Newport-Inglewood alt 1 [0]		7.99	7.45	0.84	117.958°W	33.639°N	233.47	6.43
	Compton [0]		15.29	7.27	1.17	118.043°W	33.702°N	279.77	3.30
	Palos Verdes [6]		25.96	7.29	2.10	118.134°W	33.567°N	240.77	1.65
	Whittier alt 1 [2]		26.95	7.59	1.92	117.749°W	33.893°N	28.85	1.40
	Newport-Inglewood (Offshore) [0]		10.27	6.50	1.60	117.915°W	33.591°N	192.91	1.30
UC33brAvg_FM31 (opt)		Grid							21.16
	PointSourceFinite: -117.890, 33.703		5.71	5.61	1.17	117.890°W	33.703°N	0.00	4.77
	PointSourceFinite: -117.890, 33.703		5.71	5.61	1.17	117.890°W	33.703°N	0.00	4.77
	PointSourceFinite: -117.890, 33.757		8.95	5.94	1.54	117.890°W	33.757°N	0.00	1.58
	PointSourceFinite: -117.890, 33.757		8.95	5.94	1.54	117.890°W	33.757°N	0.00	1.58
	PointSourceFinite: -117.890, 33.766		10.17	5.77	1.76	117.890°W	33.766°N	0.00	1.06
	PointSourceFinite: -117.890, 33.766		10.17	5.77	1.76	117.890°W	33.766°N	0.00	1.06
UC33brAvg_FM32 (opt)		Grid							20.88
	PointSourceFinite: -117.890, 33.703		5.73	5.60	1.18	117.890°W	33.703°N	0.00	4.60
	PointSourceFinite: -117.890, 33.703		5.73	5.60	1.18	117.890°W	33.703°N	0.00	4.60
	PointSourceFinite: -117.890, 33.757		8.97	5.93	1.54	117.890°W	33.757°N	0.00	1.59
	PointSourceFinite: -117.890, 33.757		8.97	5.93	1.54	117.890°W	33.757°N	0.00	1.59
	PointSourceFinite: -117.890, 33.766		10.19	5.76	1.77	117.890°W	33.766°N	0.00	1.10
	PointSourceFinite: -117.890, 33.766		10.19	5.76	1.77	117.890°W	33.766°N	0.00	1.10

Liquefaction Susceptibility Analysis: SPT Method

Youd and Idriss (2001), Martin and Lew (1999)

Description: Costa Mesa FS No. 2; Case 1; PGAm 0.614; design GW 20; No overex 0

Project No.: 24475

Jun 2024

Leighton

General Boring Information:

[illegible]

General Parameters:

$$a_{\max} = 0.61g$$
$$M_W = 7.69$$

MSF eq: 1

$$\text{MSF} = 0.94$$

Hammer Efficiency = 84

$$C_E = 1.40$$
$$C_B = 1$$

C_s for SPT? TRUE

Unlined, but room for liner

Rod Stickup (feet) = 3

Ring sample correction = 0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Leighton

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

Project: Costa Mesa FS No. 2; Case 1; PGAm 0.614; design GW 20; No overex 0

Project No.: 24475

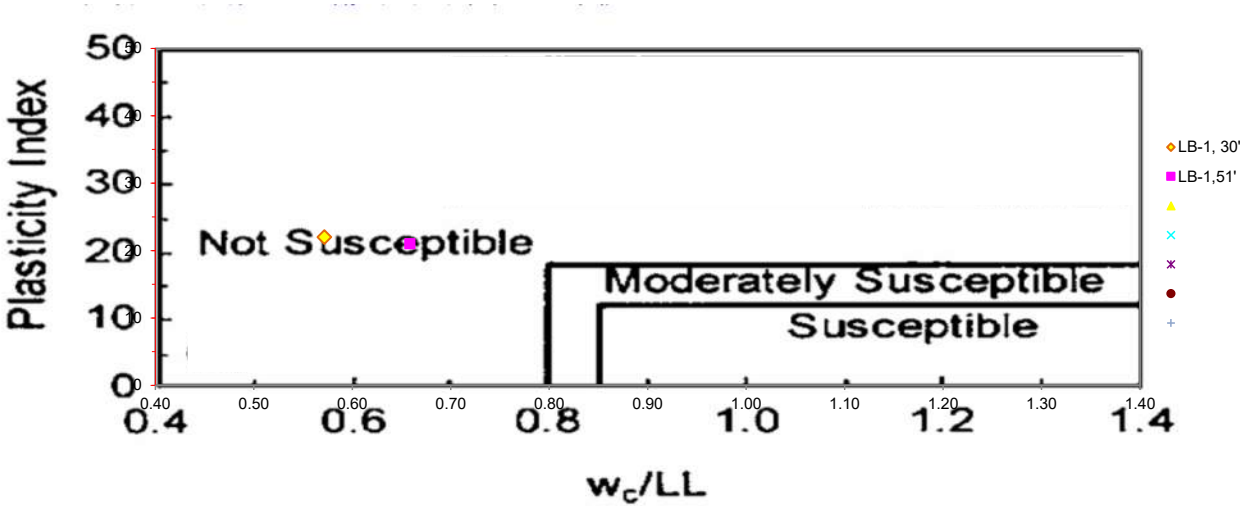
Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thickness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	$CRR_{7.5}$	Design			Liquefaction Factor of Safety	$(N_1)_{60CS}$ (for Settlement)	Dry Sand Strain (%) (Tok/ Seed 87)	Sat Sand Strain (%) (Tok/ Seed 87)	Seismic Sett. of Layer (in.)	Cummulative Seismic Settlement (in.)	
														σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M							
LB-1	0 to 3.8	2.5	3.8		65	120	40	2	1	26.0	300	46.4	60.7	>Range	300	0.40	0.42	NonLiq	60.7	0.01		0.00	0.1
LB-1	3.8 to 6.3	5	2.5		65	120	41	2	1	26.7	600	47.6	62.1	>Range	600	0.39	0.42	NonLiq	62.1	0.01		0.00	0.1
LB-1	6.3 to 8.8	7.5	2.5		50	120	23	2	1	15.0	900	25.5	35.6	>Range	900	0.39	0.42	NonLiq	35.6	0.06		0.02	0.1
LB-1	8.8 to 11.3	10	2.5		25	120	29	2	1	18.9	1200	29.6	37.3	>Range	1200	0.39	0.42	NonLiq	37.3	0.09		0.03	0.1
LB-1	11.3 to 13.8	12.5	2.5		75	120	39	2	1	25.4	1500	35.6	47.7	>Range	1500	0.39	0.41	NonLiq	47.7	0.03		0.01	0.1
LB-1	13.8 to 17.5	15	3.8		55	120	38	2	1	24.7	1800	31.7	43.0	>Range	1800	0.39	0.41	NonLiq	43.0	0.02		0.01	0.0
LB-1	17.5 to 20.0	20	2.5		55	120	16	1	1.25	19.9	2400	24.7	34.7	>Range	2400	0.38	0.41	NonLiq	34.7	0.10		0.03	0.0
LB-1	20.0 to 22.5	20	2.5		55	120	16	1	1.25	19.9	2400	24.7	34.7	>Range	2400	0.38	0.41	NonLiq	34.7			0.00	0.0
LB-1	22.5 to 27.5	25	5.0		50	120	46	2	1	29.9	2938	33.5	45.2	>Range	2688	0.42	0.45	NonLiq	45.2			0.00	0.0
LB-1	27.5 to 32.5	30	5.0	n	79	120	11	1	1.14	12.5	3226	14.1	22.0	>Range	2976	0.45	0.48	NonLiq	22.0			0.00	0.0
LB-1	32.5 to 37.5	35	5.0		75	120	37	2	1	24.1	3514	26.0	36.1	>Range	3264	0.46	0.49	NonLiq	36.1			0.00	0.0
LB-1	37.5 to 42.5	40	5.0		10	120	58	1	1.3	75.4	3802	78.2	80.8	>Range	3552	0.46	0.49	NonLiq	80.8			0.00	0.0
LB-1	42.5 to 47.5	45	5.0		10	120	100	2	1	65.0	4090	65.0	67.3	>Range	3840	0.45	0.48	NonLiq	67.3			0.00	0.0
LB-1	47.5 to 52.0	50	4.5	n	90	120	17	1	1.2	20.3	4378	19.7	28.6	>Range	4128	0.44	0.47	NonLiq	28.6			0.00	0.0
LB-2	0 to 3.8	2.5	3.8		55	120	37	2	1	24.1	300	42.9	56.5	>Range	300	0.40	0.42	NonLiq	56.5	0.01		0.00	0.1
LB-2	3.8 to 6.3	5	2.5		65	120	52	2	1	33.8	600	60.3	77.4	>Range	600	0.39	0.42	NonLiq	77.4	0.01		0.00	0.1
LB-2	6.3 to 8.8	7.5	2.5		65	120	26	2	1	16.9	900	28.8	39.6	>Range	900	0.39	0.42	NonLiq	39.6	0.07		0.02	0.1
LB-2	8.8 to 11.3	10	2.5		65	120	22	2	1	14.3	1200	22.4	31.9	>Range	1200	0.39	0.42	NonLiq	31.9	0.16		0.05	0.1
LB-2	11.3 to 13.8	12.5	2.5		65	120	35	2	1	22.8	1500	31.9	43.3	>Range	1500	0.39	0.41	NonLiq	43.3	0.06		0.02	0.1
LB-2	13.8 to 17.5	15	3.8		55	120	22	1	1.3	28.6	1800	36.7	49.0	>Range	1800	0.39	0.41	NonLiq	49.0	0.07		0.03	0.0
LB-2	17.5 to 20.0	20	2.5		45	120	50	2	1	32.5	2400	40.3	53.4	>Range	2400	0.38	0.41	NonLiq	53.4	0.03		0.01	0.0
LB-2	20.0 to 22.5	20	2.5		45	120	50	2	1	32.5	2400	40.3	53.4	>Range	2400	0.38	0.41	NonLiq	53.4			0.00	0.0
LB-2	22.5 to 27.5	25	5.0		45	120	20	1	1.29	25.8	2938	28.9	39.7	>Range	2688	0.42	0.45	NonLiq	39.7			0.00	0.0
LB-2	27.5 to 32.0	30	4.5		65	120	29	2	1	18.9	3226	21.2	30.5	>Range	2976	0.45	0.48	NonLiq	30.5			0.00	0.0

Plasticity and Liquefaction Susceptibility (based on Bray and Sancio, 2006)

Leighton

Costa Mesa FS No. 2; Case 1; PGAm 0.614; design GW 20; No overex 0
24475
Jun 2024

<u>Boring depth</u>	<u>USCS</u>	<u>Fines Content</u>	<u>Moisture content, Wc (%)</u>	<u>Liquid Limit, LL</u>	<u>Plasticity Index, PI</u>	<u>wc/LL</u>	<u>Preliminary Susceptibility (considering only plasticity, not consistency)</u>	<u>Susceptibility, ignoring in situ moisture content</u>
LB-1, 30'	CL	79	24	42	22	0.57	Not Susceptible	Not Susceptible
LB-1,51'	CL	90	27	41	21	0.658537	Not Susceptible	Not Susceptible



Reference:
Bray, J.D., and Sancio, R.B., 2006, Assessment of the Liquefaction Susceptibility of Fine-Grained Soils, Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, Vol. 132, No. 9, dated September 1, 2006, pp. 1165-1177.

APPENDIX D

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADINGTable of Contents

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1.3	The Earthwork Contractor	2
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2.2	Processing	3
2.3	Overexcavation	3
2.4	Benching	3
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LEIGHTON CONSULTING, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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Standard Details

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B - Oversize Rock Disposal	Rear of Text
C - Canyon Subdrains	Rear of Text
D - Buttress or Replacement Fill Subdrains	Rear of Text
E - Transition Lot Fills and Side Hill Fills	Rear of Text

LEIGHTON CONSULTING, INC.
General Earthwork and Grading Specifications

1.0 General

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

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

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

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

LEIGHTON CONSULTING, INC.
General Earthwork and Grading Specifications

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

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

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

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

2.0 Preparation of Areas to be Filled

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

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

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

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

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

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3.0 Fill Material

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

4.0 Fill Placement and Compaction

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

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

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5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

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

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

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

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

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

APPENDIX E

GBA'S IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This Geotechnical-Engineering Report

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer about Change

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

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

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

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

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

This Report May Not Be Reliable

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

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

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

Most of the “Findings” Related in This Report Are Professional Opinions

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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



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