Geotechnical Evaluation

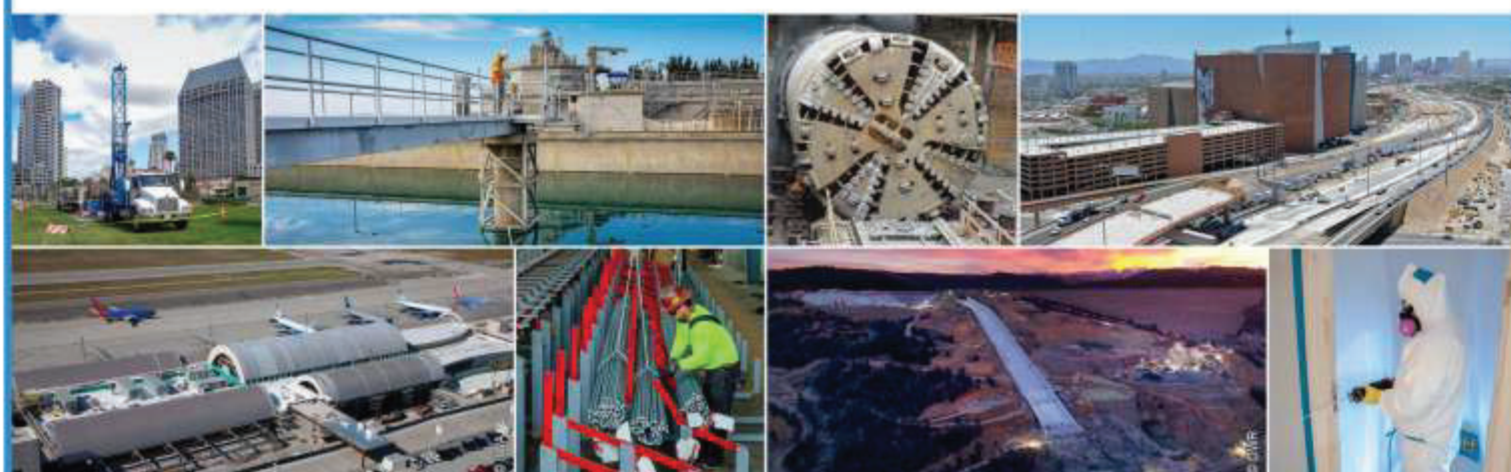
Almond Street Improvement Project
Via Verde Street to Carnelian Street

Rancho Cucamonga, California

Kimley-Horn

660 South Figueroa Street, Suite 2050 | Los Angeles, California 90017

January 15, 2026 | Project No. 213220001



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Mr. Patrick Wong

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1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the Almond Street Improvement Project located in the City of Rancho Cucamonga, California (Figure 1). We previously submitted our geotechnical evaluation report for the project on November 14, 2025 (Ninyo & Moore, 2025b), which is based on the 2025 California Building Code. Based on our recent discussions, it is our understanding that aspects of the project will be designed in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (8th Edition) with California Amendments. Therefore, we have updated our geotechnical evaluation report to provide recommendations and design parameters in accordance with the AASHTO LRFD Bridge Design Specifications (8th Edition) with California Amendments. The purpose of our study was to evaluate the soil and geologic conditions at the site in order to provide geotechnical recommendations for the design and construction of the proposed road improvements. This report presents our geotechnical findings, conclusions, and recommendations regarding the project improvements.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination, planning, and scheduling of the subsurface exploration.
- Review of readily available background material, including published geologic maps, fault and seismic hazards maps, groundwater data, topographic maps, and aerial photographs.
- Review of construction drawings for the project.
- Acquisition of an encroachment permit from the City of Rancho Cucamonga.
- Geotechnical site reconnaissance to observe the general site conditions, mark the boring locations, and coordinate with Underground Service Alert for utility clearance.
- Subsurface exploration consisting of the drilling, logging, and sampling of four hollow-stem auger exploratory borings to depths ranging from approximately 6 to 20 feet below the ground surface. The borings were logged by a representative of our firm and bulk and relatively undisturbed soil samples were collected at selected depths for laboratory testing.
- Laboratory testing on selected soil samples, including evaluation of in-situ moisture content and dry density, percentage of soil particles finer than the No. 200 sieve, direct shear strength, R-value, and soil corrosivity.
- Geotechnical engineering analyses of data from our background review, subsurface exploration, and laboratory testing.

- Preparation of this report presenting our findings, conclusions, and recommendations pertaining to the geotechnical aspects of the design and construction of the proposed improvements.

3 SITE DESCRIPTION AND PROPOSED CONSTRUCTION

The Almond Street Improvement Project will extend the pavement for Almond Street between Via Verde Street and Carnelian Street in the City of Rancho Cucamonga, California (Figure 1). The street alignment is generally bounded by residential properties to the south and by orange orchards to the north that are bisected by a north-south trending unnamed natural drainage. Residential properties are also located on the north side of Almond Street near the east and west ends of the street improvement project. The topography of the site is gently sloping to the south with ground surface elevations ranging from approximately 2,135 to 2,160 feet above mean sea level across the length of the proposed road (Kimley-Horn, 2026). The site is currently used as an unpaved equestrian and walking trail.

Based on our review of the project plans (Kimley-Horn, 2025 and 2026), we understand that the proposed improvements include constructing a new paved roadway to connect the gap between Via Verde Street and Carnelian Street. The proposed improvements include a new 36-foot-wide road, including curb and gutter, an equestrian trail and sidewalks, a retaining wall, street light poles, and a drainage culvert. Cut and fill grading will be needed to construct the roadway, with cuts and fills of up to approximately 7 feet. The proposed retaining wall will be adjacent to the public right-of-way boundary along the northeast portion of the alignment, and will be approximately 104 feet long and up to approximately 8 feet high. The existing inlet structure will be demolished in order to construct the new drainage culvert. The new drainage culvert will consist of three segments: a warped wingwall to the north, two 4-foot by 7-foot cast-in-place reinforced concrete box (RCB) culverts in the center, and a transition structure (single RCB culvert) to the south. There will be approximately 6 to 8.5 feet of cover over the double RCB culvert after roadway grading. The warped wing walls and the transition structure will be up to 10.5 feet high. It is our understanding that the retaining walls, RCB culverts, and warped wingwalls will be designed in accordance with Caltrans Standard Plans (2025a). Some relatively minor 2:1 (horizontal to vertical) fill slopes less than 5 feet in height are also proposed.

4 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration was performed on September 26, 2025, and consisted of drilling, logging, and sampling of three hollow-stem auger borings (B-1 through B-3) and one hand-auger boring (B-4). The borings were drilled to depths ranging from approximately 6 to 20 feet below the ground surface. It should be noted that drilling refusal was encountered in boring B-4 at a depth of approximately 6 feet on possible cobbles and boulders (proposed depth of 10 feet) and boring B-3 was terminated early due to an unmarked water pipeline (proposed depth of 10 feet).

Borings B-1 through B-3 were drilled using a truck-mounted drill rig with 8-inch-diameter hollow-stem augers and boring B-4 was excavated using an approximately 4-inch-diameter hand-auger. The borings were drilled to evaluate the subsurface conditions and logged by a representative from our firm. Bulk and relatively undisturbed samples were collected at selected depths for laboratory testing. Representative samples were transported to our laboratory for geotechnical testing. Logs of the exploratory borings are presented in Appendix A. The approximate locations of the exploratory borings are shown on Figure 2.

Laboratory testing was performed on representative soil samples to evaluate in-situ moisture content and dry density, percentage of soil particles finer than the No. 200 sieve, direct shear strength, R-value, and soil corrosivity. The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The remaining laboratory test results are presented in Appendix B.

5 GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Regional Geology

The site is located in the southwestern portion of San Bernardino County, north of the Chino Basin, and south of the San Gabriel Mountains, in the Transverse Ranges Geomorphic Province of southern California (Norris and Webb, 1990). The Transverse Ranges are a belt of east/west-trending folds and associated thrusts that formed in response to northeast to north-east crustal shortening that initiated in Pliocene time, approximately 4 to 5 million years before the present. This regional north-south compression causes the bedrock units to become progressively folded and faulted, forming valleys and uplands that are generally bounded by reverse/thrust faults, which generally dip north along the southern range fronts and dip south along the northern range fronts. The site is located between the Sierra Madre Fault Zone to the north, the San Andreas and San Jacinto Fault Zones to the east and southeast, and the Whittier-Elsinore Fault system to the south and southwest.

5.2 Site Geology

Based on our review of geologic maps, the site is underlain by very young to young alluvial-fan deposits consisting of silt, sand, pebbly to cobbly sand, gravel, and boulder-filled alluvial fan deposits (Morton and Miller, 2006) as shown on Figure 3.

Materials encountered during our subsurface exploration consisted of fill soils and alluvial soils. Fill was encountered in borings B-1 and B-2 to depths ranging from approximately 1 to 1.5 feet and fill was encountered in borings B-3 and B-4 to the explored depths of approximately 6 to 7 feet. The fill generally consisted of moist, medium dense to very dense silty sand with gravel. Alluvium was encountered beneath the fill to the explored depths of approximately 11.5 feet and 20 feet in borings B-1 and B-2, respectively. The alluvium consisted of moist, medium dense to dense silty sand and clayey sand. The fill and alluvial soils contained varying amounts of gravel and cobbles; difficult drilling conditions were encountered in layers containing cobbles. Boring B-3 was terminated at a depth of approximately 7 feet due to encountering an unmarked water pipeline. Boring B-4 encountered refusal on possible cobbles and boulders at a depth of approximately 6 feet. More detailed descriptions of the subsurface materials encountered during our subsurface exploration are presented on the boring logs in Appendix A.

5.3 Groundwater

Groundwater was not encountered in our exploratory borings drilled to a depth of up to approximately 20 feet below the ground surface. Historic high groundwater data was not available for the site. However, a groundwater monitoring well located approximately 1.7 miles southwest of the site recorded groundwater at approximately 463 feet below the ground surface (California Department of Water Resources, 2025). Fluctuations in groundwater level will occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors that may not have been evident at the time of our field evaluation.

6 FAULTING AND SEISMICITY

The subject site is not located within a State of California Earthquake Fault Zone (EFZ) (formerly known as the Alquist-Priolo Special Studies Zone) (California Department of Conservation, Division of Mines and Geology [CDMG], 1995). However, the site is located approximately 700 feet south of the Sierra Madre Fault Zone. The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered significant during the design life of the proposed project. Figure 4 shows the approximate site location relative to the major faults in the region.

The principal seismic hazards evaluated at the subject site are surface fault rupture, ground motion, liquefaction, and landslides. A brief description of these hazards and the potential for their occurrences on-site are discussed in the following sections.

6.1 Surface Fault Rupture

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

6.2 Ground Motion

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. Version 4.0 of the Caltrans Acceleration Response Spectra (ARS) online tool (Caltrans, 2025c) was used to calculate the design seismic event with respect to the proposed improvements. The design ARS represents the United States Geological Survey (USGS, 2014) 5 percent in 50 years hazard at 5 percent damping and includes near-fault effects and basin amplification effects.

The input parameters for the Caltrans ARS online tool consist of the site latitude, site longitude, and average shear wave velocity (V_S) in the upper 100 feet (i.e., 30 meters) (V_{S30}). Based on our review of California Geological Survey's (CGS) shear wave velocity map, the V_{S30} is estimated to be approximately 1,155 feet per second (352 meters per second) (CGS, 2015). This shear wave velocity is representative of a site mapped on dense sand or very stiff clay. The design ARS curve evaluated for the site is presented on Figure 5. The calculated peak ground acceleration (PGA) is 0.75g and the mean moment magnitude is 7.1.

6.3 Liquefaction Potential

Liquefaction is the phenomenon in which loosely deposited granular soils and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

According to the San Bernardino County Land Use Plan map (San Bernardino County, 2010), the site is not mapped within an area considered to be susceptible to liquefaction. In addition, based on the depth of the groundwater level in the vicinity of the site and the presence of relatively dense native soils, liquefaction is not a design consideration for the project.

6.4 Landslides

Landslides, slope failures, and debris flows of earth materials generally occur where slopes are steep and the earth materials are too weak to support themselves. Landslides may be induced by strong vibratory motion produced by earthquakes. Research and historical data indicate that seismically induced landslides tend to occur in weak soil and rock on sloping terrain. The process for zoning earthquake-induced landslides incorporates expected future earthquake shaking, existing landslide features, slope gradient, and strength of earth materials on the slope.

The project area is gently sloping and there are no significant slopes planned at the site. Based on our review of the San Bernardino County Land Use Plan map, the site is not mapped in an area susceptible to landslides (San Bernardino County, 2010). However, the foothills of the San Gabriel Mountains located several hundred feet to the north of the site are mapped as moderately to highly susceptible to landslides.

7 CONCLUSIONS

Based on the results of our evaluation, it is our opinion that the proposed Almond Street Improvement Project is feasible from a geotechnical perspective, provided the recommendations presented in this report are incorporated into the design and construction of the project. The primary geotechnical consideration for the project is the location of the proposed retaining wall with respect to the public right-of-way site boundary.

As shown on the Almond Street Improvement Project plans (Kimley-Horn, 2026), a retaining wall is proposed to be constructed along the northeast right-of-way boundary. Where the retaining wall is proposed on the north side of Almond Street, a typical 1.5:1 (horizontal to vertical) temporary backcut would be needed to construct the retaining wall, which will extend off-site into the neighboring properties. In this area, permission to grade off-site would be needed to construct the backcut. If permission to grade off-site is not acquired, installation of a temporary shoring system would be appropriate where the backcut exceeds a height of approximately 4 feet.

In general, the following additional conclusions were made:

- The site is underlain by fill and alluvial soils. Artificial fill was encountered in borings B-1 and B-2 to depths ranging from approximately 1 to 1.5 feet and in borings B-3 and B-4 to depths ranging from approximately 6 to 7 feet. The fill generally consisted of moist, medium dense to very dense silty sand with gravel. Alluvium was encountered in borings B-1 and B-2 and consisted of moist, moist, medium dense to dense silty sand and clayey sand.

- The fill and alluvial soils contained varying amounts gravel and cobbles. The presence of cobbles in the alluvial soils resulted in difficult drilling conditions. Difficult excavation conditions due to oversize cobbles should be anticipated and planned for by the contractor. Although not encountered during drilling, boulders are common in the alluvial deposits in Rancho Cucamonga and they could be encountered during earthwork operations.

- The granular soils encountered at the site have low cohesion and may be subject to caving, including caving of retaining wall backcut excavations and drilled pier foundation excavations. These soils should be considered Type C soils in accordance with Occupational Safety and Health Administration (OSHA) soil classifications. Appropriate shoring systems for these types of soils should be considered during planning if site constraints do not allow for sloped excavations. Casing may be needed for drilled pier foundations where caving is encountered.

- Boring B-3 struck an unmarked water pipeline that resulted in a pipe rupture. The pipeline owner repaired the pipeline; however, Ninyo & Moore was not on-site to observe the backfill of the excavation that was made to repair the pipe. If soil was used to backfill the excavation for the emergency repair, any loose soil should be removed and placed back as compacted fill.

- The on-site soils should be suitable for use as compacted fill provided that the soils are free of trash, debris, roots, vegetation, deleterious materials, and cobbles or hard lumps of material in excess of 4 inches in diameter.

- Groundwater was not encountered in our exploratory borings up to a depth of approximately 20 feet below the ground surface. Groundwater is not anticipated to be a design consideration for the project. Fluctuations in groundwater levels may occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors which may not have been evident at the time of our field evaluation.

- According to San Bernardino County, the site is not located in an area mapped as being susceptible to liquefaction (San Bernardino County, 2010). Based on the depth of groundwater level in the vicinity of the site and the relatively dense native soils, liquefaction is not a design consideration for the project.

- According to San Bernardino County, the site is not located in an area mapped as being susceptible to landslides (San Bernardino County, 2010). However, the foothills of the San Gabriel Mountains located several hundred feet to the north of the site are mapped as moderately to highly susceptible to landslides.

- The subject site is not located within a State of California EFZ (CGS, 2018). However, the site is located approximately 700 feet south of the Sierra Madre Fault Zone. Based on our review of published geologic maps, there are no known active faults underlying the site. Therefore, the potential for surface fault rupture at the site is considered to be low. The site is located in a seismically active area and the potential for strong ground motion in the project area is considered significant during the design life of the proposed project.
- Our laboratory corrosion testing indicates that the site soils can be classified as non-corrosive based on the California Department of Transportation (Caltrans, 2021a) corrosion guidelines.

8 RECOMMENDATIONS

The recommendations presented in the following sections provide geotechnical criteria regarding the design and construction of the proposed new road and associated improvements. The recommendations are based on the results of our subsurface evaluation, geotechnical analyses, and project understanding. The proposed work should be performed in conformance with the recommendations presented in this report, project specifications, and requirements of applicable governing agencies. The retaining walls, RCB culverts, and warped wingwalls may be designed in accordance with Caltrans Standard Plans (2025a).

8.1 Earthwork

We anticipate that earthwork for the project will consist of relatively shallow remedial grading for the proposed new road, sidewalks and trails, culvert, and retaining wall. Earthwork operations should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented in the following sections of the report.

8.1.1 Construction Plan Review and Pre-Construction Conference

We recommend that the construction plans be submitted to Ninyo & Moore for review to evaluate conformance to the geotechnical recommendations provided in this report. We further recommend that consideration be given to holding a pre-construction conference. The owner and/or their representative, the governing agencies' representatives, the civil engineer, the geotechnical consultant, and the contractor should attend to discuss the work plan, project schedule, and earthwork requirements.

8.1.2 Clearing and Site Preparation

Prior to performing excavations or other earthwork, the area should be cleared of existing improvements, including concrete, rubble and debris, abandoned utilities, surface obstructions, and other deleterious materials. Existing utilities within the project limits should be re-routed or protected from damage by construction activities. Materials generated from the clearing operations should be removed from the project site and disposed of at a legal dumpsite.

8.1.3 Excavation Characteristics

Based on our field exploration, we anticipate that excavations at the site may be accomplished with conventional earth-moving equipment in good working condition. We anticipate that the existing fill and alluvial materials encountered during construction will be generally comprised of medium dense to very dense clayey sand and silty sand with variable amounts of gravel and cobbles. Based on our borings, our review of the regional geologic conditions, and our experience with the depositional environment, the contractor should anticipate encountering oversized materials such as cobbles and boulders. Difficult drilling/excavating conditions should be anticipated due to the presence of cobbles and possible boulders.

8.1.4 Treatment of Near-Surface Soils

In order to provide suitable support and reduce the potential settlements of the proposed improvements, we recommend the following:

8.1.4.1 Culvert and Retaining Wall

Remedial grading will be needed in order to provide suitable support for the new buried culvert and the proposed retaining wall where competent native materials are not encountered at the foundation subgrade elevation. Based on the planned cuts, we anticipate that competent native materials will be encountered for the retaining wall along the north side of Almond Street where deeper excavations are needed to reach the foundation level. However, if the excavations on the north side of Almond Street do not remove the existing undocumented fill and/or or expose loose native soils, additional overexcavation down to competent native soils will be appropriate. Where additional overexcavation is needed, the overexcavation should extend a minimum of two feet below the foundation subgrade elevation. However, the actual depth of overexcavation should be evaluated by a Ninyo & Moore representative during earthwork and will depend on the conditions that are exposed.

Similarly, the construction drawings indicate that excavations on the order of 5 feet deep will be needed to install the new RCB culvert. Based on our subsurface exploration, we anticipate that competent native soils will be exposed at the foundation subgrade elevation for the new RCB culvert based on the planned excavation depths. If the excavation for the RCB culvert does not remove the existing undocumented fill and/or or exposes loose native soils, additional overexcavation down to competent native soils will be appropriate. If additional overexcavation is needed, the overexcavation should extend a minimum of two feet below the foundation subgrade elevation. However, the actual depth of overexcavation should be evaluated by a Ninyo & Moore representative during earthwork and will depend on the conditions that are exposed.

In summary, where competent native soils are not exposed at the foundation subgrade elevation of the proposed culvert and retaining wall, we recommend that the existing fill and loose native soils be removed and recompacted (i.e., overexcavated) to a depth of approximately 2 feet beneath the footprints of the structures in order to provide suitable support and reduce the potential for settlement. The excavations should extend deep enough to remove the existing undocumented fill and loose native soils and should expose relatively firm and unyielding alluvial deposits. The excavation bottoms should be evaluated by our representative during the excavation work. Additional excavation of loose, soft, and/or wet areas may be appropriate. The limits of removal should extend approximately 2 feet beyond the outside edges of the foundation footprint or a distance equal to the depth of the excavations beneath the foundations, whichever is farther.

If drainage rock is placed beneath the foundations for the RCB culverts, the rock can be considered to be part of the 2-foot-thick layer of compacted fill beneath the foundations. Prior to placing compacted fill and/or drainage rock, the upper approximately 8 inches of the exposed bottom should be scarified, moisture-conditioned to near the laboratory optimum moisture content, and recompacted to a relative compaction of 90 percent as evaluated by ASTM International (ASTM) test method D 1557. If drainage rock is used beneath the structure(s), we recommend that it consist of 0.75-inch to 1.5-inch-diameter, open-graded gravel underlain by filter fabric consisting of Mirafi 140N or equivalent. Alternatively, drainage rock and filter fabric installed in accordance with Caltrans specifications is also acceptable.

8.1.4.2 Pavement

In order to provide suitable support for the proposed pavements subject to vehicular traffic, we recommend that the upper approximately 2 feet of material beneath the bottom of the proposed pavement section be removed and recompacted. The lateral limits of the excavation should extend 2 feet or more beyond the outside edges of the pavements. The excavation should expose relatively dense fill and/or alluvium in a relatively non-yielding condition. The excavation bottom should be observed by our representative during the excavation work. Additional excavation of loose, soft, wet areas, areas containing excessive construction debris or organics (generally more than 5 percent), and/or yielding subgrade materials may be appropriate. As indicated in the Conclusions section of this report, additional remedial grading may be needed to remove loose backfill associated with the pipeline repair near boring B-3. The upper approximately 8 inches of the approved subgrade should be scarified, moisture-conditioned to slightly above the optimum moisture content, and compacted to 90 percent or more as evaluated by ASTM D 1557.

The quality of the finished pavement subgrade soils should be evaluated at the time of construction to check the appropriate pavement design sections, including additional R-value testing, as needed.

8.1.4.3 Hardscape

In order to provide suitable support for the proposed hardscape areas, we recommend that the upper approximately 1 foot of material beneath the bottom of the proposed hardscape, be removed and recompacted. The lateral limits of the excavation should extend 1 foot or more beyond the outside edges of the hardscape. The excavation should expose relatively dense fill and/or alluvium. The excavation bottom should be observed by our representative during the excavation work. Additional excavation of loose, soft, wet areas, areas containing excessive construction debris or organics (generally more than 5 percent) may be appropriate. The upper approximately 8 inches of the approved subgrade beneath the proposed hardscape should be scarified, moisture-conditioned to slightly above the optimum moisture content, and compacted to a relative compaction of 90 percent or more as evaluated by ASTM D 1557.

8.1.5 Temporary Excavations

Soils at the project site include granular soils with little cohesion that are considered to be prone to caving. In particular, bedding materials for existing pipelines, if encountered, may be prone to caving. Temporary slopes in the site soils should be stable at inclinations of approximately 1:1 (horizontal to vertical) up to a depth of about 4 feet below the existing grade and stable at inclinations of approximately 1.5:1 (horizontal to vertical) for excavations deeper than 4 feet but not exceeding a depth of 20 feet below the existing grade. Temporary excavations should be evaluated in the field and constructed in accordance with the applicable OSHA guidelines. The site soils should be considered as OSHA Soil Type C. On-site safety of personnel is the responsibility of the contractor.

We anticipate that temporary shoring may be needed due to public right-of-way boundary constraints as well as to protect existing buried utilities. Shoring systems, if used, should be designed for the anticipated soil conditions using the lateral earth pressure values presented on Figures 6 and 7 for braced and cantilevered shoring, respectively. The recommended design earth pressures are based on the assumption that the shoring system will be constructed without raising the ground surface elevation behind the shored sidewalls of the excavation, that there will be no surcharge loads such as soil stockpiles and construction materials, and that no loads will act above a 1:1 (horizontal to vertical) plane ascending from the base of the shoring system. For a shoring system subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the lateral earth pressures acting on the shored walls. Spoils should not be placed near the edge of the open cut excavation. For open cut excavations, the spoil pile should be placed at a distance more than the depth of excavation from the top of the excavation. OSHA and other applicable agency requirements pertaining to worker safety should be met during the excavation activities.

We anticipate that settlement of the ground surface will occur behind shored excavations. The amount of settlement depends heavily on the type of shoring system, the contractor's workmanship, and soil conditions. To reduce the potential for distress to adjacent improvements, we recommend that the shoring system be designed to limit the ground settlement behind the shoring system to 0.5 inch or less. Possible causes of settlement that should be addressed include settlement during installation of the shoring elements, excavation for structure construction, construction vibrations, and removal of the support system. We recommend that shoring installation be evaluated carefully by the contractor prior to construction and that ground vibration and settlement monitoring be performed during construction where shoring is installed within 50 feet of the existing structures that will be protected in-place.

The contractor should retain a qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are minimum requirements, and the contractor should evaluate the adequacy of these parameters and make the appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

8.1.6 Fill Material

In general, the on-site soils should be suitable for re-use as fill and retaining wall backfill. Fill should be free of trash, debris, roots, vegetation, or other deleterious materials. Fill should generally be free of rocks or lumps of material in excess of 4 inches in diameter. Oversize cobbles should be broken into smaller pieces (less than 4 inches in diameter) or removed from the site.

Imported fill materials should consist of clean, non-expansive, granular materials with a low expansion potential, corresponding to an expansion index of 50 or less, and conform to the Greenbook specifications (Public Works Standard, Inc., 2024) or Section 19-3.02C of the Caltrans Standard Specifications (2025b) for structural backfill. The soil should also be tested for corrosive properties prior to importing. We recommend that the imported materials meet the 2021 Caltrans criteria for non-corrosive soils (i.e., soils having a chloride concentration of less than 500 parts per million [ppm], a soluble sulfate content of less than approximately 0.15 percent [1,500 ppm], a pH value higher than 5.5, and an electrical resistivity more than 1,500 ohm-centimeters [ohm-cm]). Materials for use as fill should be evaluated by Ninyo & Moore prior to importing. The contractor should be responsible for the uniformity of imported materials brought to the site.

8.1.7 Fill Placement and Compaction

Fill should be placed and compacted in accordance with the project specifications and sound construction practices. Fill materials should be moisture-conditioned to slightly above the optimum moisture content. The lift thickness for fill soils will vary depending on the type of compaction equipment used, but should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. Fill materials should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Special care should be taken to avoid pipe damage when compacting trench backfill above pipes. Fill should be tested for specified compaction level by Ninyo & Moore.

8.2 Underground Utilities

We anticipate that existing underground pipelines will not be replaced or relocated as part of this project. However, new pipelines should be supported on engineered fill or native alluvial deposits. The depths of the utilities are not known; however, we anticipate that the pipe invert depths will not exceed 10 feet.

8.2.1 Pipe Bedding

We recommend that pipes be supported on 6 inches or more of granular bedding material, such as sand, with a sand equivalent (SE) value of 30 or more. Bedding material should be placed around the pipe and 12 inches or more above the top of the pipe in accordance with the Caltrans Standard Specifications (2025b). Special care should be taken not to allow voids beneath the pipe. Compaction of the bedding material and backfill should proceed up both sides of the pipe. Trench backfill, including bedding material, should be placed in accordance with the recommendations presented in the Earthwork section of this report.

8.2.2 Trench Backfill

Based on our subsurface evaluation, the on-site soils should generally be suitable for re-use as trench backfill provided that those are free of organic material, clay lumps, debris, and rocks more than approximately 4 inches in diameter. Fill should be moisture-conditioned to at or slightly above the optimum moisture content. Wet soils should be allowed to dry to a moisture content near the optimum prior to their placement as trench backfill. Trench backfill should be compacted to a relative compaction of 90 percent or more as evaluated by ASTM D 1557. Lift thickness for trench backfill will depend on the type of compaction equipment used, but should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipelines during compaction of the backfill.

8.2.3 Modulus of Soil Reaction for Pipe Design

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines for the purpose of evaluating deflections caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of 1,000 pounds per square inch (psi) be used for design, provided that granular bedding material is placed adjacent to the pipe, as recommended in the previous section.

8.3 Foundations

It is our opinion that the proposed drainage culvert (wing walls, double RCB culvert, and transition structure) may be supported on a mat foundation, the retaining wall may be supported on spread footings, and the light poles may be supported on drilled piers. In general, the foundations should be prepared in accordance with AASHTO LRFD Bridge Design Specifications (8th Edition) with California Amendments and other applicable Caltrans design manuals.

8.3.1 Shallow Foundations

Shallow foundations should be supported on competent native alluvium or compacted engineered fill in accordance with the recommendations presented in the Earthwork section of this report. Foundation dimensions used in our analysis including width, length, embedment depth, etc., are based on the project plans (Kimley-Horn, 2026). It should be noted that loading information of the proposed improvements was not provided to us at the time of this evaluation.

The shallow foundations were evaluated in accordance with Section 10.6 of the AASHTO LRFD Bridge Design Specifications (8th Edition) with California Amendments. We calculated the required factored nominal resistance of the proposed foundations based on the Service, Strength, and Extreme Event Limit States. The bearing resistance of the footings was calculated using the principle of general shear failure of foundation materials subjected to a footing of finite shape and loading. The Service Limit State was based on a total permissible settlement of 1 inch per our discussions with Kimley-Horn, and the corresponding differential settlement is estimated to be approximately 0.5 inch over a horizontal distance of approximately 40 feet. A total unit weight of 120 pounds per cubic foot and a friction angle of 32 degrees for compacted engineered fill and/or alluvium beneath the footings were used in our geotechnical calculations for the shallow foundations. Our shallow foundation design recommendations are provided in the following table.

Table 1 – Foundation Design Recommendations for Shallow Foundations

Support Location	Assumed Foundation Size (ft)		Bottom of Foundation Elevation (ft)	Foundation Embedment (ft)	Total Permissible Support Settlement (in)	Service Limit State	Strength Limit State (φb = 0.45)	Extreme Event Limit State (φb = 1.0)
	Width (B)	Length (L)				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
Retaining Wall	9.8	103.8	2,147.97	4.0	1	3.5	13.0	28.8
Culvert – Warped Wingwall (North)	9.5	14.0 to 49.2	2,144.52 to 2,145.09	7.8 to 9.9 (Apron Bottom – 7 inches)	1	3.7	6.6	14.8
Culvert – Double Box Culvert (Center)	14.0	50.0	2,141.19 to 2,144,52	9.8 to 12.5	1	4.4	24.6	54.7
Culvert – Transition Structure (South)	6.7 to 19.9	21.3	2,139.92 to 2,141.49	7.1 to 10.4	1	4.9	15.4	34.2

A coefficient of friction of 0.62 (Equation 10.6.3.4-2 of AASHTO LRFD Bridge Design Specifications [8th Edition] with California Amendments) should be used between the horizontal surface of the spread footings and the underlying foundation materials in order to evaluate resistance against sliding. Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the mat. The appropriate contact pressure(s) beneath the bases of mat foundations will vary with their size, shape, and other factors. A design modulus of subgrade reaction (K) of 100 pounds per cubic inch (pci) may be used for the subgrade soils in evaluating such deflections. This value is based on a unit square foot area and should be adjusted for the planned mat size. The coefficient of subgrade reaction, K_b, for a mat of a specific width may be evaluated using the following equation:

$$\mathbf{K_b = K_v[(b+1)/2b]^2}$$

Where, b is the width of the foundation in feet.

8.3.2 Drilled Pier Foundations

Drilled pier foundations for the light poles should have a diameter of 24 inches or more. For drilled piers with an embedment depth of 6 feet or more in native materials (alluvium), we recommend an allowable side friction value of 80 pounds per square foot (psf) in compression under static loading condition starting at a depth of 1 foot below the ground surface. End bearing should be ignored for these drilled piers. In addition, an allowable side friction of 55 psf for uplift (tension) can also be used in design. The allowable axial capacities are based on a factor of safety of 2.0. The allowable axial capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. The lateral capacity of drilled piers may be evaluated using a passive resistance of 200 psf per foot of depth, up to a value of 2,000 psf, acting against the width of the pier. The passive resistance can be increased by a factor of 2 to account for the soil arching effect on individual piers. Passive resistance should be ignored in the upper 1 foot of soil. If the ground surface at and around the piers is finished with hardscape or asphalt concrete, passive resistance may be used in the upper 1 foot of depth. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces. These recommendations assume that the piers will have a minimum spacing of three times the pier diameter. The project structural engineer should evaluate the design depth of the piers based on the recommendations provided above.

8.3.2.1 Drilled Shaft Construction Considerations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our exploratory borings, site observations, and experience with similar materials. The drilled shaft contractor should mobilize equipment of sufficient size and operating capability to achieve the recommended pier embedment depth. The excavation technique chosen by the contractor should not adversely affect the quality or strength of the shaft side or end bearing materials. If refusal is encountered in these materials during actual installation, Ninyo & Moore should be notified to evaluate the subsurface condition to establish that true refusal has been met with adequate drilling equipment.

Based on our recent borings and review of nearby groundwater data, groundwater is not expected to be encountered during drilling; however, seepage and perched water conditions could be encountered. The contractor should be prepared to place the concrete pier in the drilled hole under such conditions. If significant seepage is encountered, a temporary casing may be used in the drill-hole to reduce water infiltration. Temporary casing may also be needed if caving is encountered in the drill-hole where granular soils with little cohesion are encountered.

In addition, we recommend that concrete be placed by tremie method so that the aggregate and cement do not segregate during concrete placement. Concrete utilized in the drilled shafts should be a fluid mix with sufficient slump so that it will fill the void between the rebar cage and the drill-hole wall. The contractor should take care to reduce enlargement of the excavation at the tops of drilled shafts, which could result in mushrooming of the drilled shaft top.

Drilled shaft holes shall be cleaned prior to placement of concrete. Care shall be taken to check that the soils at the drilled shaft bottom have not been disturbed. The successful advancement of drill-holes for the construction of drilled shafts will depend largely on the suitability of the drilling equipment and the skill of the operator. The drilled foundation contractor should try to reduce the time during which the excavation remains open. The contractor should schedule the sequence of operations so that each excavation can be finished, the rebar cage placed, and the concrete placed within the same work day. Drilled shaft excavations should not be left open overnight. In case of delays in placing concrete within the drill-hole due to equipment breakdown or other unforeseen circumstances, casing may be used to protect the integrity of the hole. While placing concrete, the casing should be withdrawn gradually.

The drilled shaft installation should be observed by the Geotechnical Engineer or a qualified representative to check that, among other things: 1) subsurface conditions are as anticipated from the borings and the drilled shaft extends into competent native alluvium, 2) the drilled shaft bottom is clean and competent, 3) the drilled shaft is constructed to the specified size and depth, and 4) the drilled shaft is within allowable tolerances for plumbness. These items are fundamental to the installation and behavior of the pier foundations. Furthermore, we recommend the following for the installation of drilled shaft:

- The clear spacing between the rebar cage and the drill-hole surface should be three times the maximum size of the coarse aggregate used in the concrete.
- Centralizers should be installed to keep the bottom rebar cage positioned per project specifications.
- If casing is used, a sufficient head of concrete that fills the casing should be used before pulling the casing.

8.4 Lateral Earth Pressures for Underground Structures

Below-grade walls for the subterranean structures (i.e., RCB culvert) may be considered to be restrained from lateral displacement under static loading conditions. Restrained walls subjected to lateral earth pressures should be designed using the parameters presented on Figure 8. For restrained walls, application of the additional active seismic earth pressure component will not be needed when considering the at-rest earth pressure condition.

8.5 Lateral Earth Pressures for Retaining Walls

Based on our review of the project drawings (Kimley-Horn, 2026), we understand that the proposed retaining wall will be designed in accordance with the Caltrans Standard Plans (2025a) for Type 7 retaining walls. A total unit weight of 120 pounds per cubic foot and a friction angle of 34 degrees should be used for retaining wall backfill. We considered a retaining wall with a design height of up to 8 feet. The lateral earth pressure will act on the retaining wall side of the wedge at an angle of 22.7 degrees (two thirds of the friction angle) from the direction normal to the side of the retaining wall on which soil wedge slides, and point against soil wedge movement (Caltrans, 2022).

Unrestrained (yielding) cantilevered retaining walls should be designed for an equivalent fluid weight of 34, 36, and 50 psf per foot for level backfill, 10:1 (horizontal to vertical) sloping backfill based on the measured angle of the northern slope (approximately 5.5 degrees), and 2:1 (horizontal to vertical) sloping backfill, respectively. Restrained retaining walls may be designed for an equivalent fluid weight of 53, 58, and 77 psf per foot for level backfill, 10:1 (horizontal to vertical) sloping backfill, and 2:1 (horizontal to vertical) sloping backfill, respectively. These lateral earth pressure values are unfactored.

The dynamic lateral earth pressures for retaining walls were evaluated using the Mononobe-Okabe Method based on the underlying soil characteristics that may induce lateral movement of the retaining walls. A horizontal seismic acceleration component (k_h) was taken as one third of the PGA (0.25) in accordance with Caltrans guidelines. The dynamic component for yielding retaining walls should consist of an equivalent fluid weight of 19, 22, and 27 psf per foot for level backfill, 10:1 (horizontal to vertical) sloping backfill, and 2:1 (horizontal to vertical) sloping backfill, respectively. These lateral earth pressure values are unfactored. For restrained retaining walls, application of the additional dynamic component will not be needed considering the at-rest earth pressure condition.

Recommendations for lateral earth pressures to be used in the design of the yielding (i.e., cantilevered, above-ground) and restrained (i.e., basement) retaining wall are provided on Figures 9 and 10, respectively. Retaining walls may be supported by spread footings founded in compacted fill designed in accordance with the recommendations presented in Section 8.1.4.1 of this report.

Measures should be taken to reduce moisture build-up behind retaining walls. If the walls do not allow for drainage, then hydrostatic loading should be added to the lateral earth pressure values that we provided. Retaining wall backfill should meet the specifications for structure backfill (Caltrans, 2025b) for free-draining conditions. Retaining walls should include free-draining backfill materials and perforated drains as designed by the project civil engineer and should be constructed in accordance with Bridge Detail 3-5 on Plan B0-3 of the Standard Plans (Caltrans, 2023a). Typical drainage design details are shown on Figure 11. Alternatively, geocomposite drains installed in accordance with Caltrans specifications are also acceptable.

8.6 Preliminary Pavement Design

We anticipate that pavements for the roadway and associated improvements will consist of asphalt concrete (AC) and Portland cement concrete (PCC). Traffic loading information was not available for our design at the time of preparation of this report. For planning purposes, we evaluated the structural pavement sections assuming a traffic index (TI) of 5 for light-duty pavements and a TI of 6 and 7 for medium-duty pavements. A TI of 5 is generally associated with light automobile traffic (passenger cars); a TI of 6 is generally associated with driveways, light truck driveways, and frequent commercial automobile traffic; and a TI of 7 is generally associated with roadways and periodic heavy truck traffic.

Preliminary pavement design was performed based on our evaluation of the subgrade soil conditions and laboratory testing. The R-value characteristics of the subgrade soils was evaluated from a representative near-surface soil sample obtained from our exploratory borings. Laboratory R-value testing indicated an R-value of 73. For the purpose of this project, we used a design subgrade R-value of 60 in our design.

Our pavement analysis was performed using the methodology outlined in the Caltrans Highway Design Manual (Caltrans, 2023b) and the Navy Pavement Design Manual (Naval Facilities Engineering Command, 1979). The analysis assumes an approximately 20-year design life for new pavement. For the design of PCC pavement, we assumed a 28-day compressive strength of 2,500 psi for the concrete. Based on the design subgrade R-value and TIs, recommendations for new pavement sections are provided in Table 2.

Table 2 – Preliminary Structural Pavement Sections

Traffic Index	AC over CAB or AC over CMB (inches)	Full Depth AC (inches)	PCC (inches)
5.0	3.0 over 4.0	4.0	5.5
6.0	3.5 over 4.0	5.0	6.0
7.0	4.0 over 4.0	6.0	8.0

Notes:
AC – Asphalt Concrete
CAB – Crushed Aggregate Base
CMB – Crushed Miscellaneous Base
PCC – Portland Cement Concrete with a 28-day compressive strength of 2,500 pounds per square inch.

Aggregate base material should conform to Section 26 of the Caltrans Standard Specifications (2025b) and should be compacted to a relative compaction of 95 percent in accordance with ASTM D 1557. AC should conform to Section 39 of the Caltrans Standard Specifications (2025b) and should be compacted to a relative compaction of 95 percent in accordance with ASTM D 1560 or California Test (CT) method 366. PCC should conform to Section 40 of the Caltrans Standard Specifications (2025b).

Pavement sections should be selected based on actual anticipated traffic loading conditions and evaluation of the subgrade materials, including subgrade R-value testing, at the time of construction. We recommend that the paving operations be observed and tested by Ninyo & Moore. We further recommend that the mix design for the various pavements be made by an engineering company specialized in this type of work.

8.7 Hardscape

We recommend that new exterior concrete sidewalks and flatwork (hardscape) have a thickness of 4 inches. The hardscape should be underlain by 4 inches of clean sand and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking. Positive drainage should be established and maintained adjacent to flatwork. To reduce the potential for differential offset, joints between the new hardscape and adjacent curbs, existing hardscape, building walls, and/or other structures, and between sections of new hardscape, should be doweled.

8.8 Corrosivity

Laboratory testing was performed on a near-surface soil sample collected from boring B-2 to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with CT 643. Chloride content testing was performed in general accordance with CT 422. Sulfate content testing was performed in general accordance with CT 417.

The pH of the tested sample was measured at approximately 7.5 and the electrical resistivity was measured at approximately 5,175 ohm-cm. The chloride content was measured at approximately 20 ppm and the sulfate content was measured at approximately 0.001 percent by weight (i.e., 10 ppm). According to the 2021 Caltrans corrosion criteria, the soils at the project site can be classified as non-corrosive, which is defined as having earth materials with less than 500 ppm chlorides, less than 0.15 percent sulfates (i.e., 1,500 ppm), a pH more than 5.5, and an electrical resistivity more than 1,500 ohm-cm. The corrosivity test results are presented in Appendix B.

8.9 Concrete

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. The potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight, moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight, severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight, and very severe for water-soluble sulfate contents over 2.00 percent by weight (American Concrete Institute, 2025b). The soil sample tested for this evaluation, using CT 417, indicates a water-soluble sulfate content of approximately 0.001 percent by weight (i.e., 10 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. However, due to the potential variability of site soils, consideration should be given to using Type II/V cement for the project.

To reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed improvements be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We further recommend that concrete cover over reinforcing steel for foundations be provided in accordance with Caltrans Standard Specifications (2025b). The structural engineer should be consulted for additional concrete specifications.

8.10 Drainage

Positive surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to transport surface water away from foundations and other site improvements. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet or more away from the foundations. Surface water should not be allowed to pond adjacent to the foundations.

8.11 Landscaping

Project landscaping should consist of drought tolerant plants. Landscape irrigation should be kept to a level just sufficient to maintain plant vigor. Overwatering should not be permitted.

9 CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and our evaluation of the data collected based on subsurface conditions observed in our exploratory borings. It is imperative that Ninyo & Moore checks the subsurface conditions during construction.

During construction, we recommend that Ninyo & Moore’s duties include, but not be limited to:

- Observing clearing, grubbing, and removals.
- Observing excavation bottoms, including culvert and retaining wall bottoms.
- Observing pier foundation excavations, if implemented.
- Observing preparation of pavement subgrades.

- Observing placement and compaction of fill, including trench backfill and structural backfill.
- Evaluating imported materials prior to their use as fill, if used.
- Performing field tests to evaluate fill compaction.
- Performing material testing services including concrete compressive strength and steel tensile strength tests and inspections.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that the services of Ninyo & Moore are not utilized during construction, we request that the selected consultant retained for geotechnical observation and testing services during construction provide you with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report.

10 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analyses of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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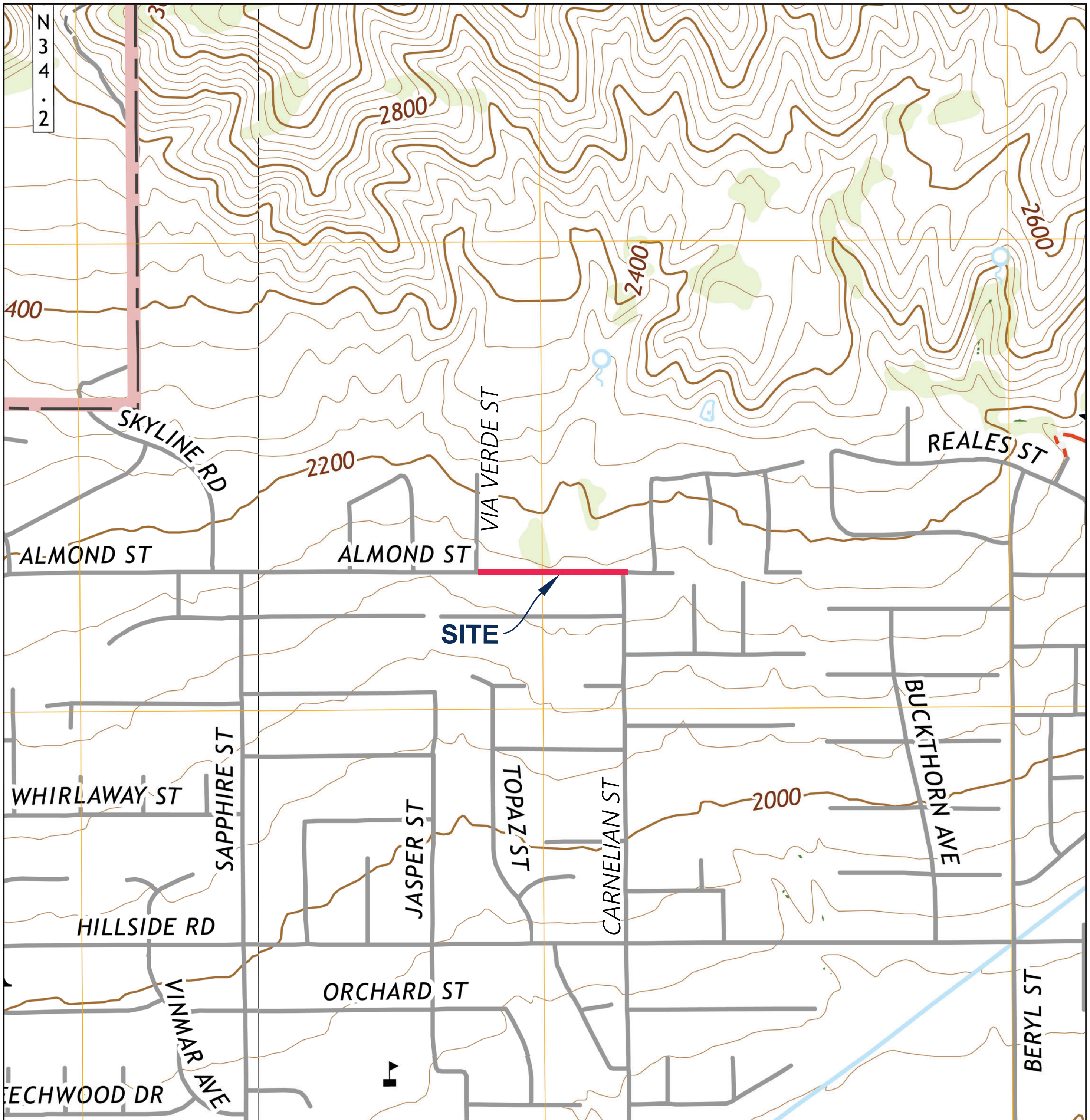
FIGURES



N

FEET

0 1,000 2,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: USGS, 2021a.

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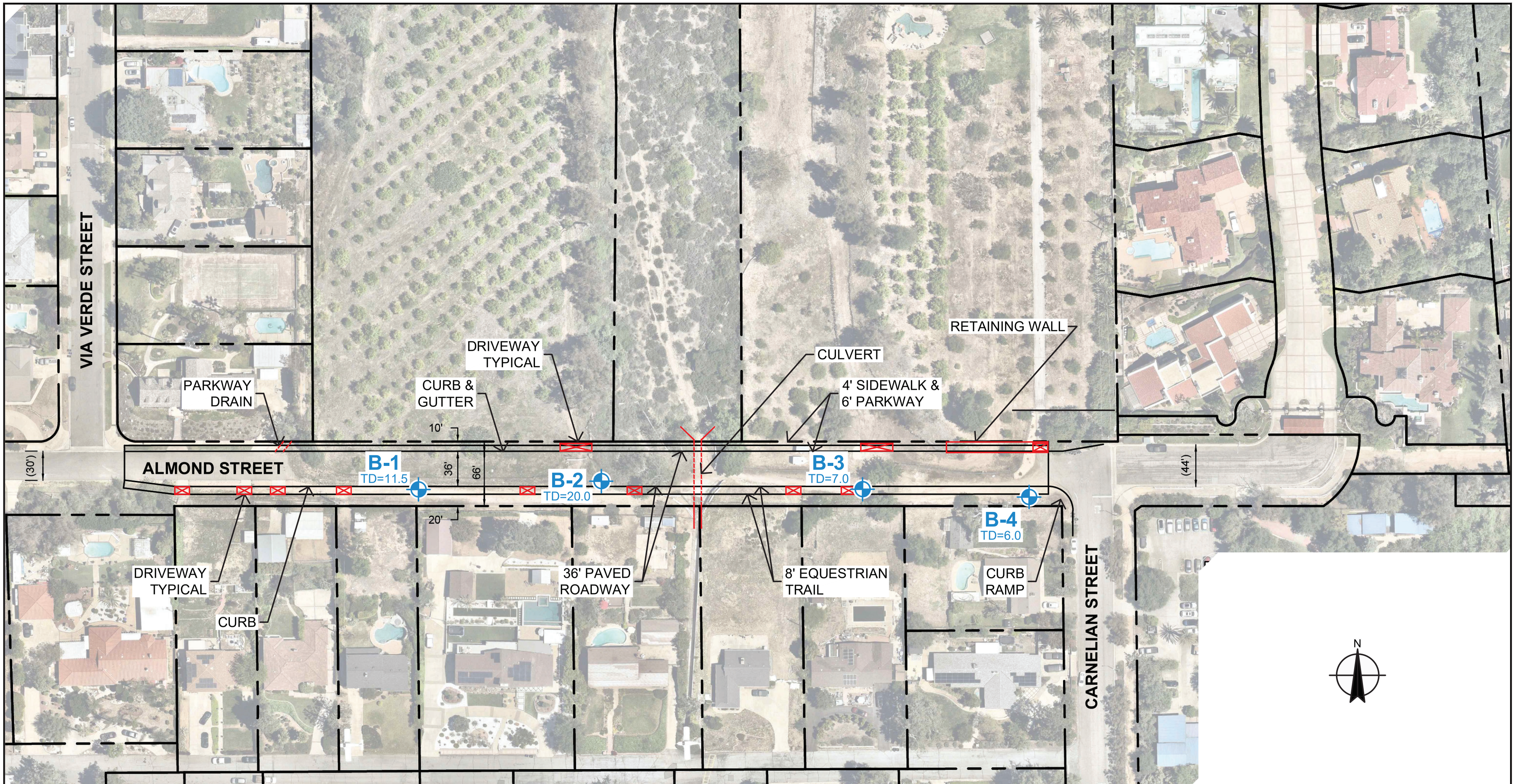
FIGURE 1

SITE LOCATION

ALMOND STREET IMPROVEMENT PROJECT
RANCHO CUCAMONGA, CALIFORNIA

213220001 I 1/26

Ninyo & Moore
A SOCOTEC COMPANY



LEGEND

B-4 TD=6.0 BORING; TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: KIMLEY-HORN, 2025.

FEET 0 150 300

FIGURE 2

SITE PLAN WITH BORING LOCATIONS

ALMOND STREET IMPROVEMENT PROJECT
RANCHO CUCAMONGA, CALIFORNIA

213220001 I 1/26

Ninyo & Moore
A SOCOTEC COMPANY

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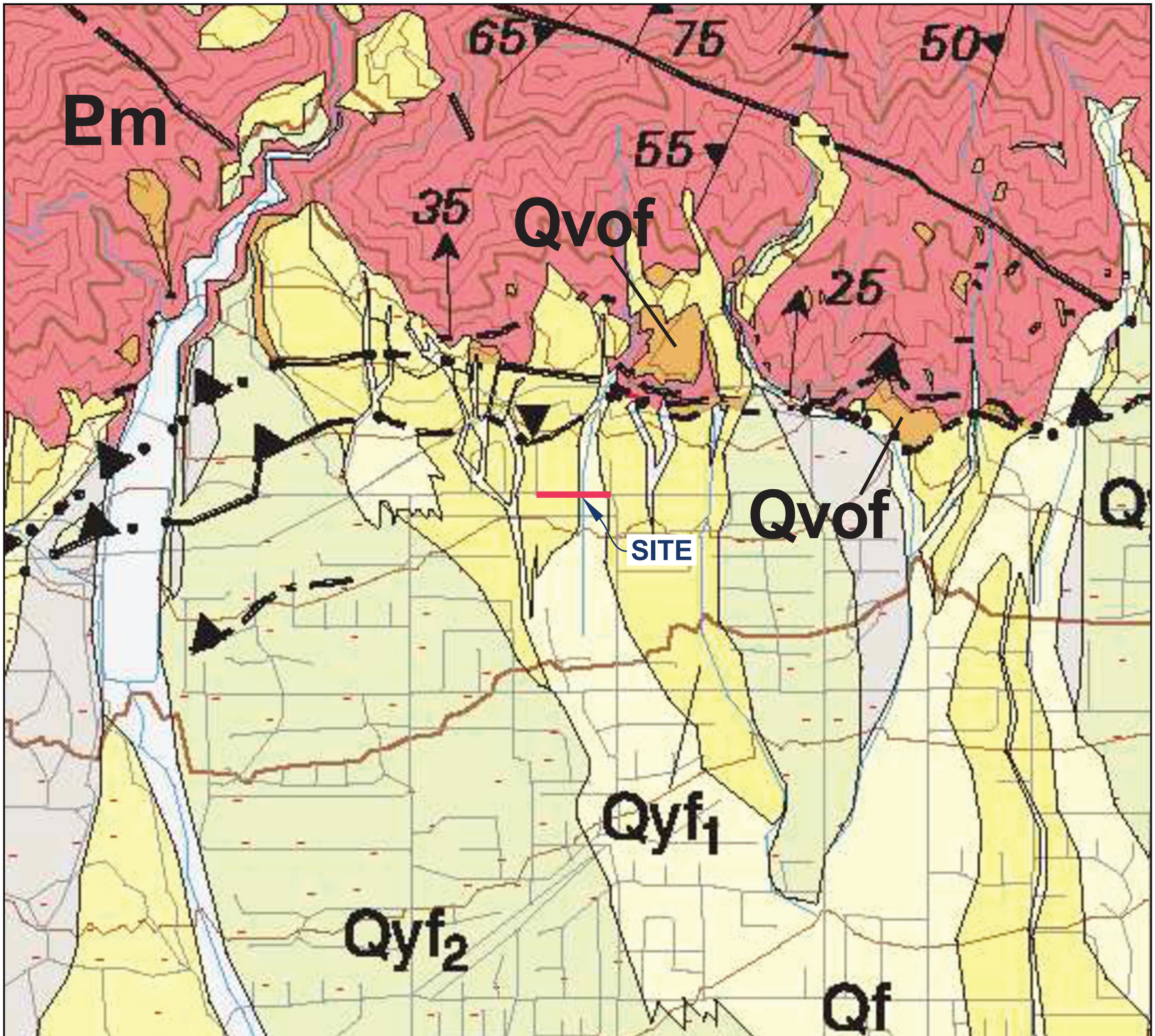

LEGEND

Qf	VERY YOUNG ALLUVIAL-FAN DEPOSITS
Qyf1	YOUNG ALLUVIAL-FAN DEPOSITS - UNIT 1
Qyf2	YOUNG ALLUVIAL-FAN DEPOSITS - UNIT 2
Qvof	VERY OLD ALLUVIAL-FAN DEPOSITS
Pm	GRANULITIC GNEISS, MYLONITE, AND CATACLASITE RETROGRADE

20 THRUST FAULT (TEETH ON UPPER PLATE, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED, AND ARROW AND NUMBER INDICATE DIRECTION AND AMOUNT OF DIP)

75 STRIKE AND DIP OF FOLIATION

GEOLOGIC CONTACT

FAULT

N

FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: MORTON AND MILLER, 2006.

213220001.dwg_RG 01/08/2026 DRAFTED BY: JLC

FIGURE 3

Ninyo & Moore

A SOCOTEC COMPANY

REGIONAL GEOLOGY

ALMOND STREET IMPROVEMENT PROJECT
RANCHO CUCAMONGA, CALIFORNIA

213220001 I 1/26

LEGEND

CALIFORNIA FAULT ACTIVITY

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

SOURCES: QUATERNARY FAULTS DATABASE - U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES, ACCESSED NOVEMBER 4, 2024, AT: https://www.usgs.gov/programs/earthquake-hazards/faults

N

MILES

0 8 16

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

P213220001.aprx 10/2/2025 DRAFTED BY: JDL

FIGURE 4

FAULT LOCATIONS

ALMOND STREET IMPROVEMENTS PROJECT

RANCHO CUCAMONGA, CALIFORNIA

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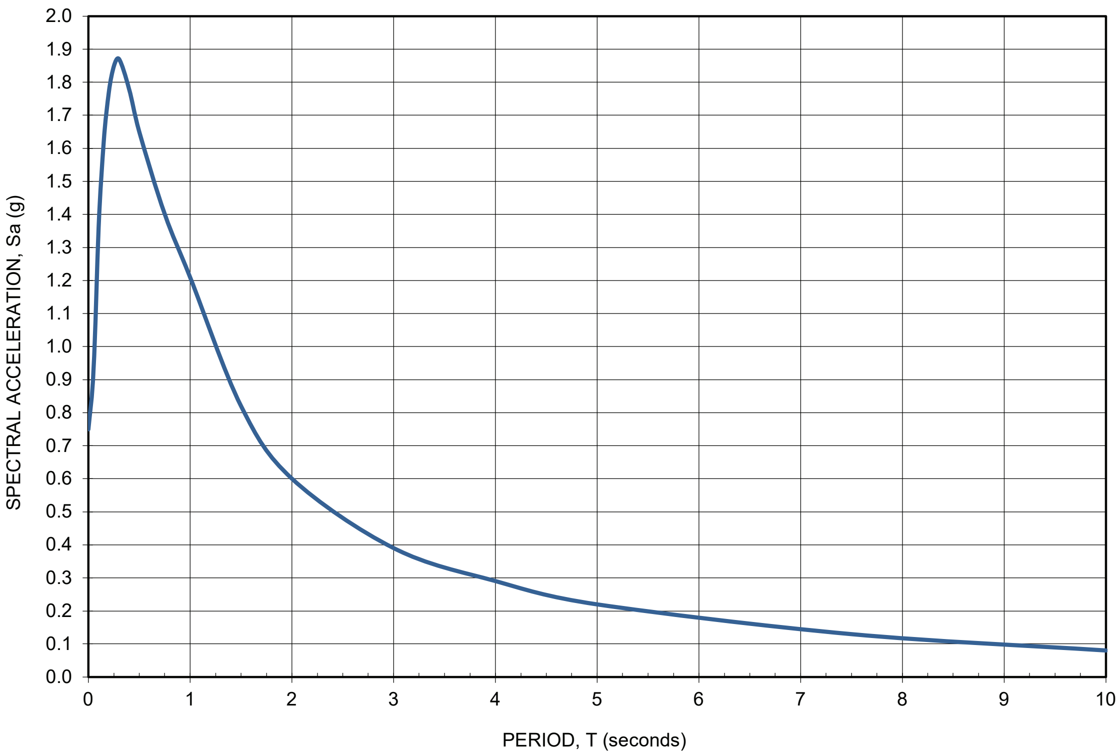
PERIOD (seconds)	SPECTRAL ACCELERATION Sa (g)
PGA	0.75
0.05	0.93
0.10	1.36
0.15	1.62
0.20	1.77
0.25	1.85
0.30	1.87
0.40	1.78
0.50	1.65
0.75	1.40
1.00	1.21
1.50	0.82
2.00	0.60
3.00	0.39
4.00	0.29
5.00	0.22
7.50	0.13
10.00	0.08



NOTES:

1 SITE LATITUDE = 34.16195 DEGREES
SITE LONGITUDE = -117.61789 DEGREES

2 AVERAGE SHEAR WAVE VELOCITY IN THE UPPER 30 METERS (ASSUMED) = 352 METERS/SECOND

3 THE ACCELERATION RESPONSE SPECTRUM REPRESENTS THE UNITED STATES GEOLOGICAL SURVEY 5% IN 50 YEARS HAZARD (2014) AT 5% DAMPING.

4 THE ACCELERATION RESPONSE SPECTRUM INCLUDES NEAR-FAULT EFFECTS AND BASIN AMPLIFICATION EFFECTS.

5 PEAK GROUND ACCELERATION (PGA) = 0.75g

6 MEAN MOMENT MAGNITUDE (M) = 7.10

FIGURE 5

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ACCELERATION RESPONSE SPECTRUM

ALMOND STREET IMPROVEMENT PROJECT

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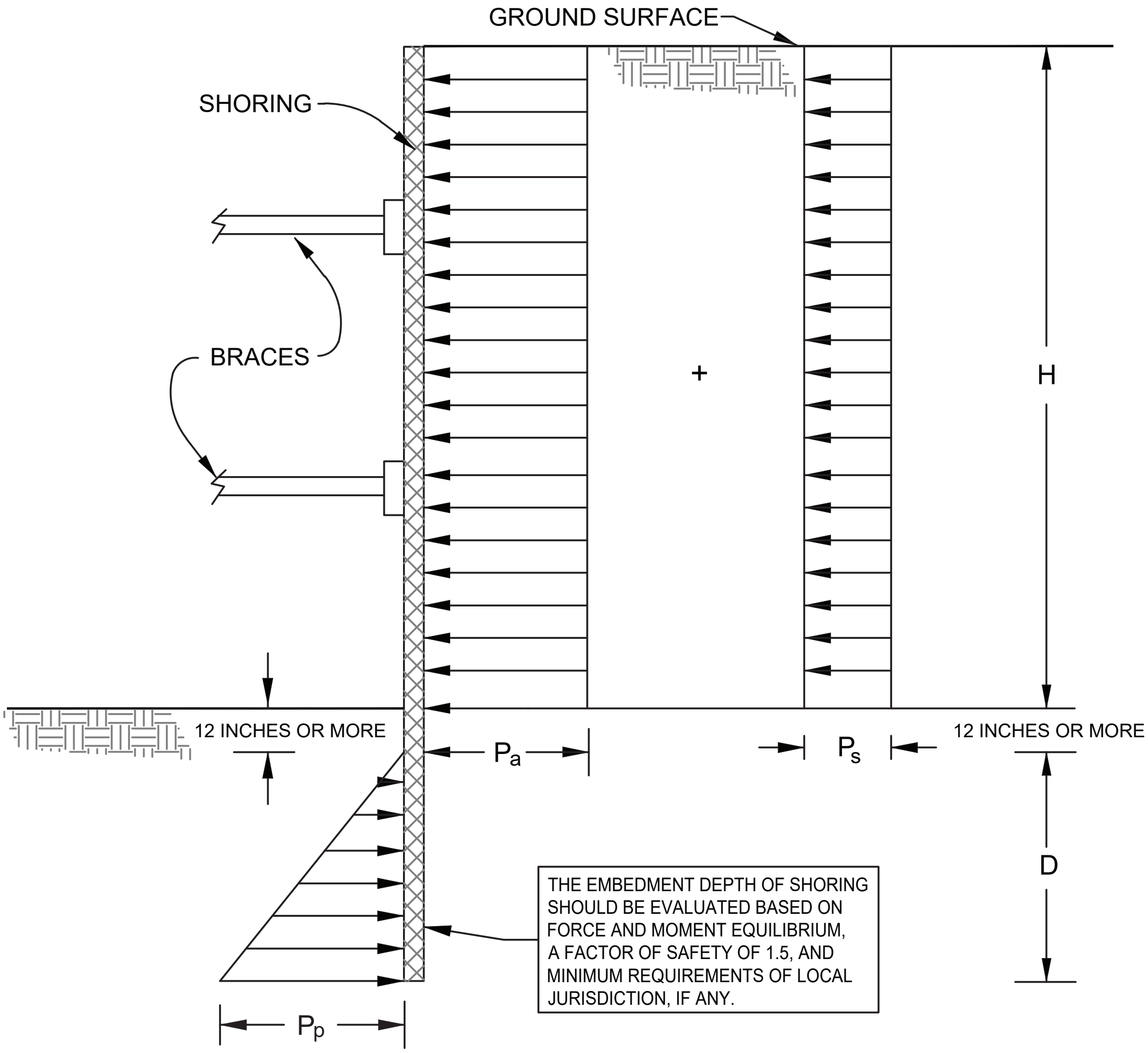

NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   P_a = 24H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 350D psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

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FIGURE 6

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION

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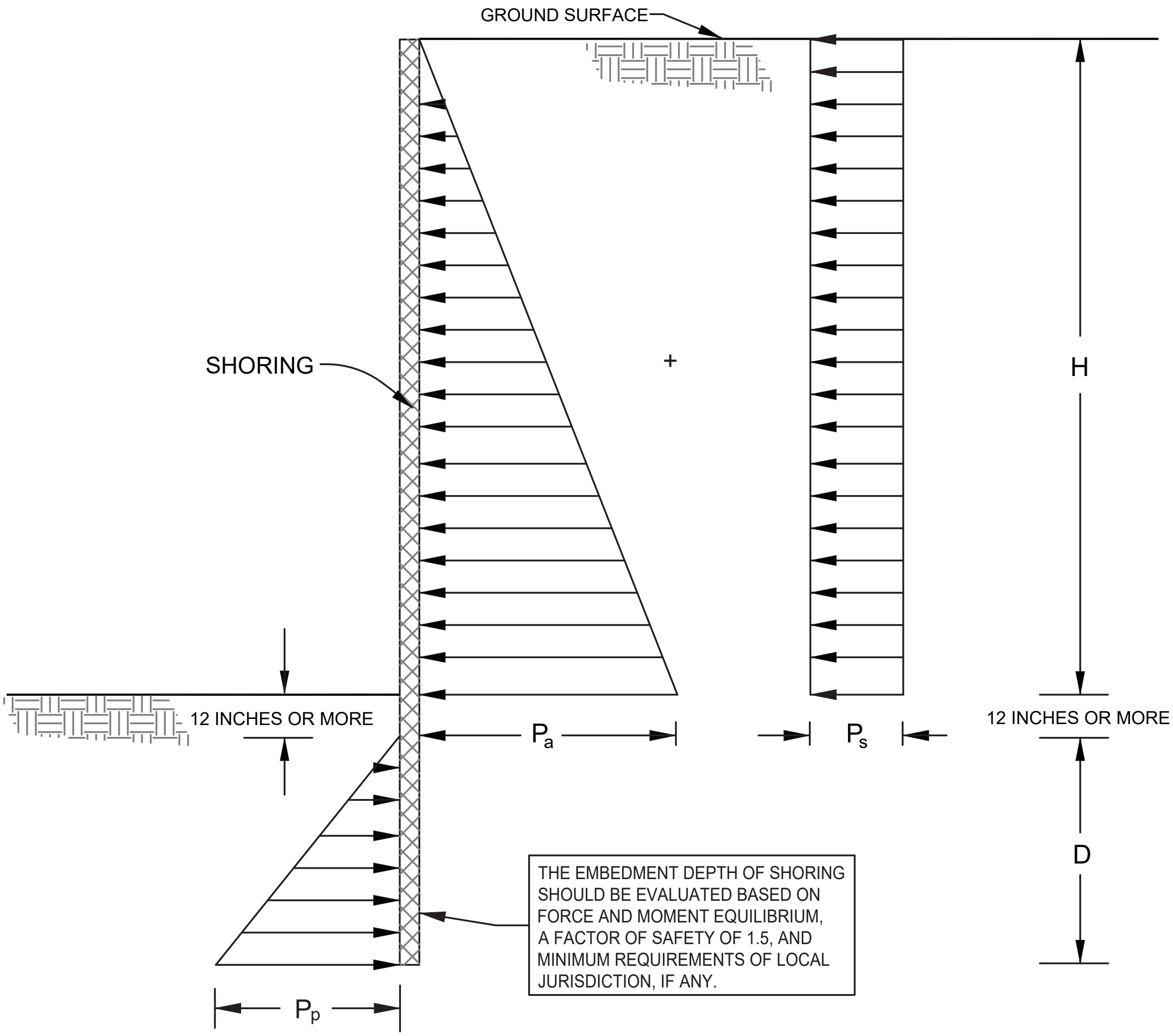

NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   P_a = 37H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 72 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 350D psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. H AND D ARE IN FEET

NOT TO SCALE

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FIGURE 7

LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING

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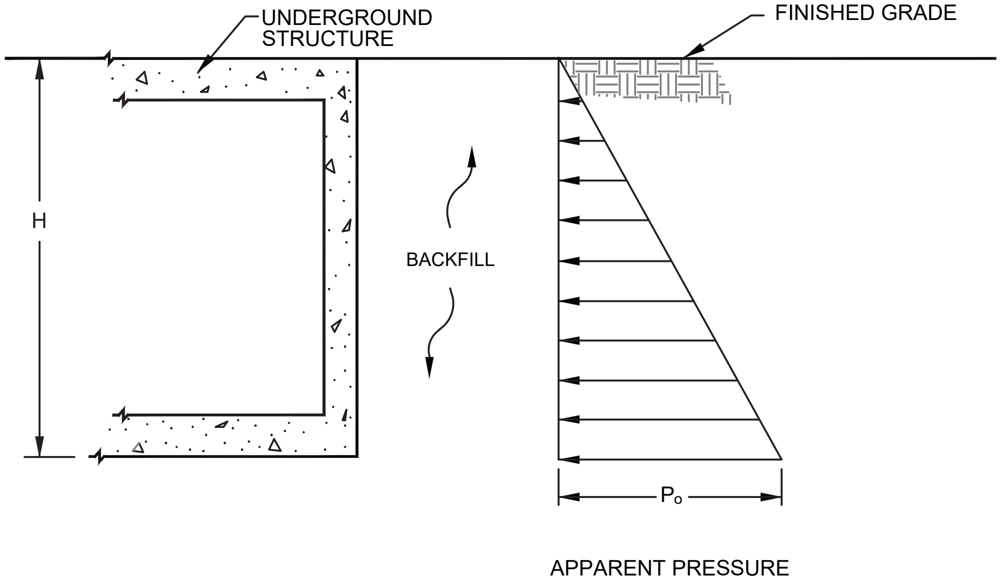

NOTES:

1. APPARENT LATERAL EARTH PRESSURES, P_o

LEVEL BACKFILL	10H:1V SLOPING BACKFILL	2H:1V SLOPING BACKFILL
P_o = 53H psf	P_o = 58H psf	P_o = 77H psf

2. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED

3. H IS IN FEET

4. ASSUMES GROUNDWATER IS NOT PRESENT

NOT TO SCALE

FIGURE 8

LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES

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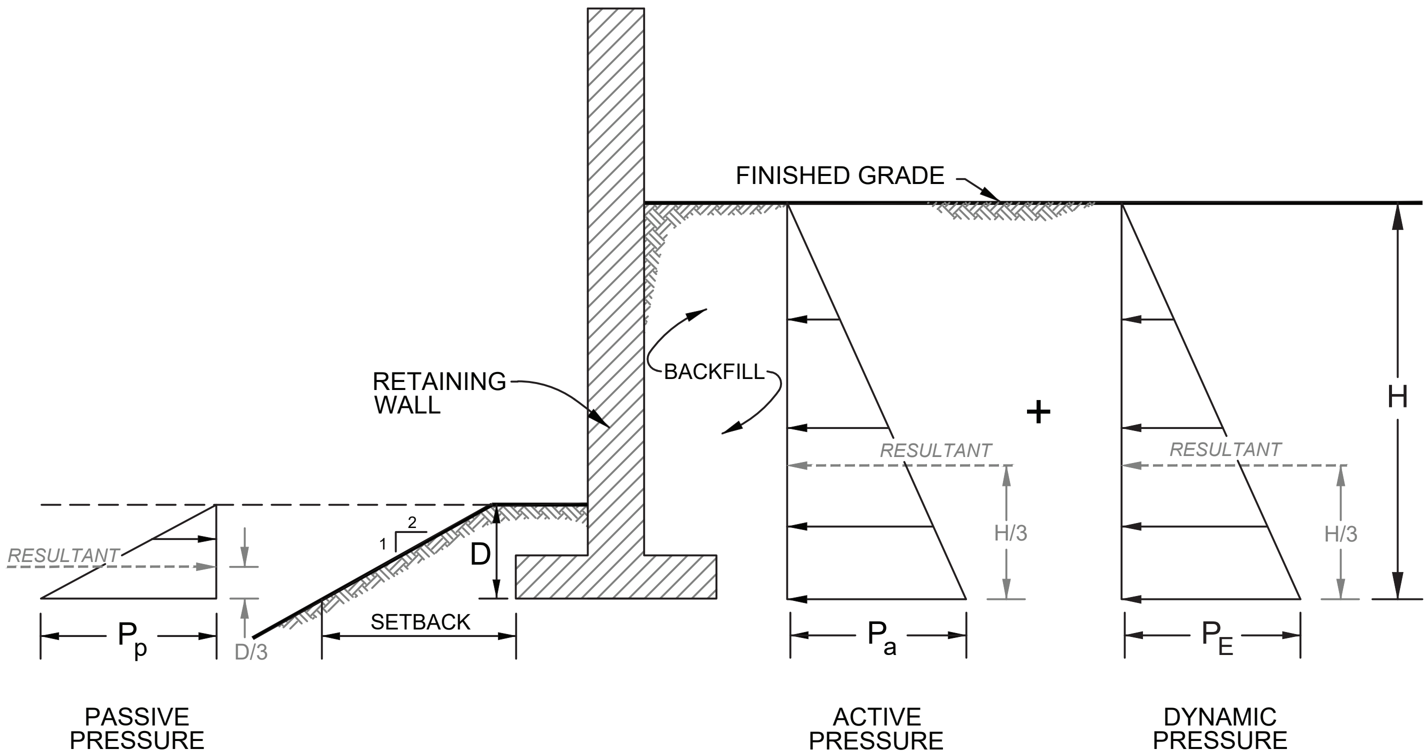

NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN CALTRANS STANDARD SPECIFICATIONS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
5. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
6. H AND D ARE IN FEET
7. SETBACK SHOULD BE IN ACCORDANCE WITH THE CURRENT VERSION OF THE APPLICABLE BUILDING CODE

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)		
P_a	Level Backfill with Granular Soils (2)	10H:1V Sloping Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	34H	36H	50H
P_E	19H	22H	27H
P_p	Level Ground	10H:1V Descending Ground	2H:1V Descending Ground
	350D	320D	140D

NOT TO SCALE

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FIGURE 9

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

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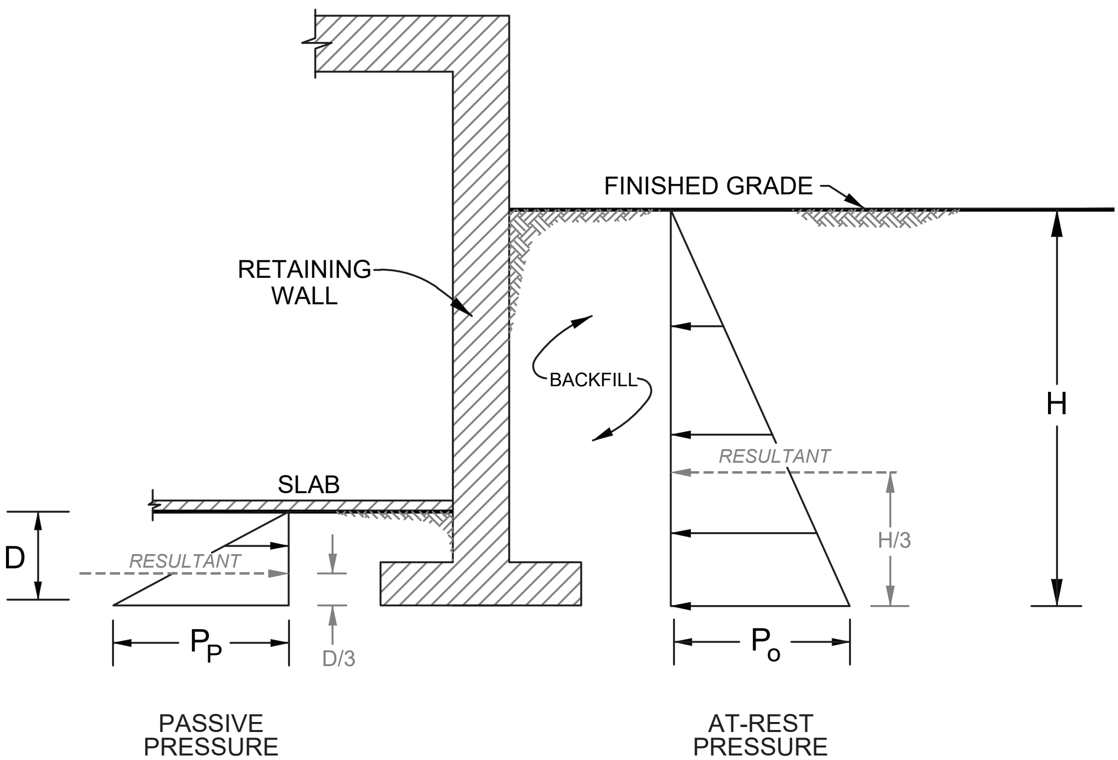

NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN CALTRANS STANDARD SPECIFICATIONS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS IGNORED AND DEEMED INAPPLICABLE DUE TO THE AT-REST CONDITION OF THE RETAINING WALL
5. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
6. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)[1]		
P_O	Level Backfill with Granular Soils [2]	10H:1V Sloping Backfill with Granular Soils [2]	2H:1V Sloping Backfill with Granular Soils [2]
	53H	58H	77H
P_P	Level Ground	10H:1V Descending Ground	2H:1V Descending Ground
	350D	320D	140D

NOT TO SCALE

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FIGURE 10

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

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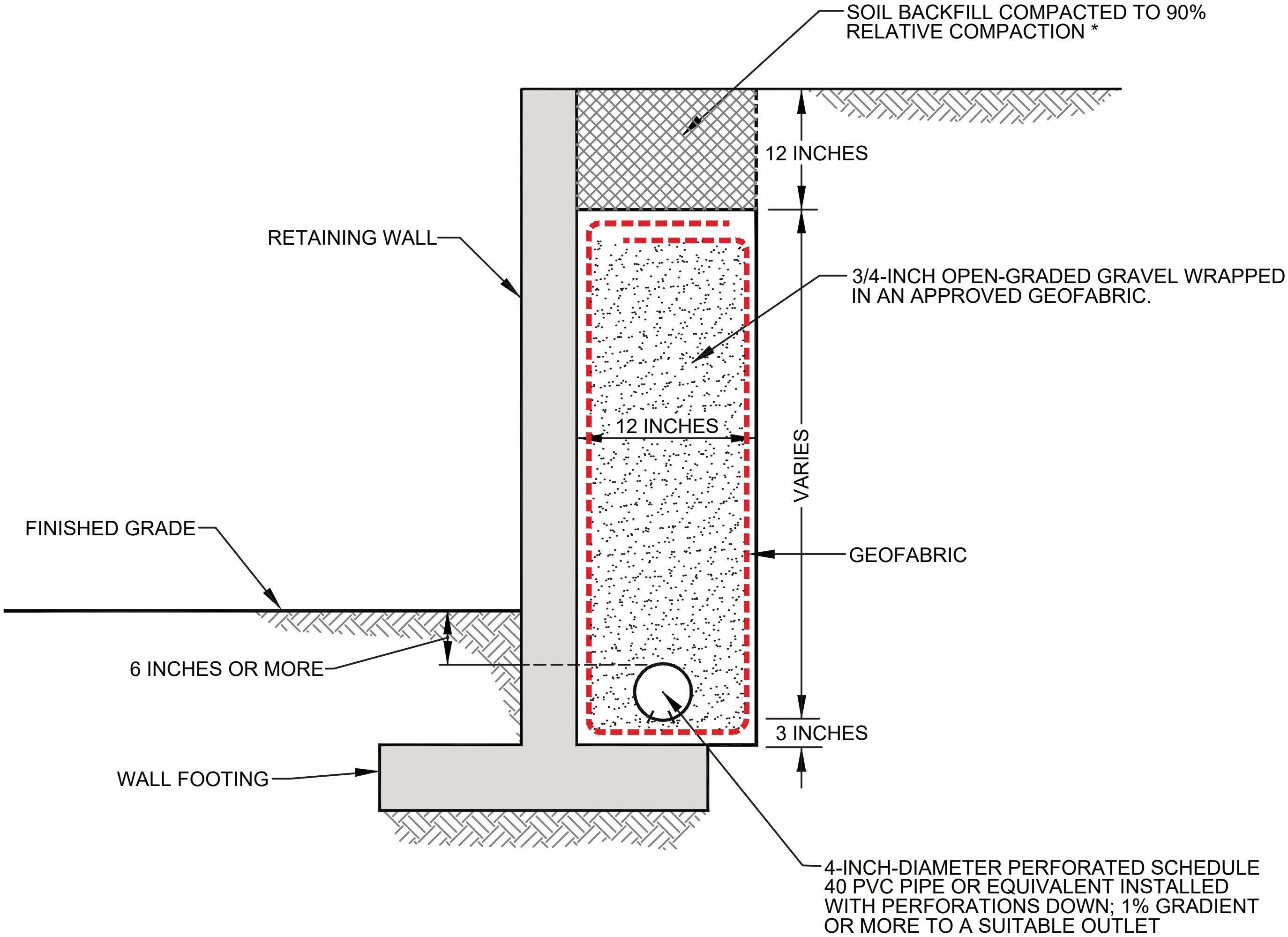
ALMOND STREET IMPROVEMENT PROJECT
RANCHO CUCAMONGA, CALIFORNIA



*BASED ON ASTM D1557

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NOT TO SCALE

FIGURE 11

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RETAINING WALL DRAINAGE DETAIL

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213220001 I 1/26

APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

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

Bulk Samples

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

Field Procedure for the Collection of Relatively Undisturbed Samples

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

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sampler barrel in the brass rings, sealed, and transported to the laboratory for testing.

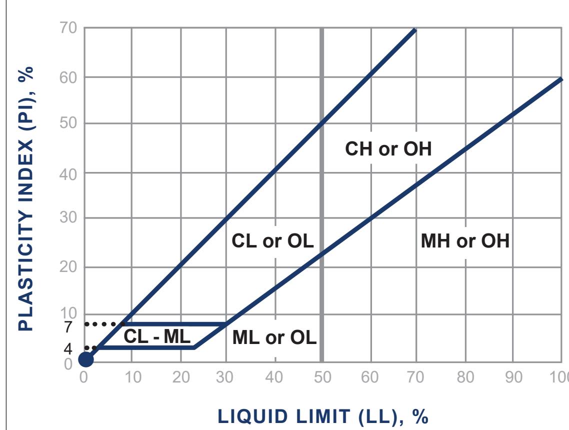
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consistency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
20								The total depth line is a solid line that is drawn at the bottom of the boring.

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 9/26/25 BORING NO. B-1 GROUND ELEVATION 2144' ±(MSL) SHEET 1 OF 1 METHOD OF DRILLING 8" Hollow Stem Auger (2R Drilling) DRIVE WEIGHT 140 lbs. DROP 30-inches SAMPLED BY ARB LOGGED BY ARB REVIEWED BY MLP **DESCRIPTION/INTERPRETATION**
0	X						SM	FILL: Reddish yellow, moist, medium dense, silty SAND with gravel.
	X						SM	ALLUVIUM: Reddish yellow, moist, medium dense, silty SAND with gravel.
		X	23	6.8	107.5			
							SC	Reddish brown, moist, medium dense, clayey SAND; few to little gravel.
10		X	16	11.2	122.7			
								Total Depth = 11.5 feet. Groundwater not encountered during drilling. Backfilled with on-site soils on 9/26/25. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								
30								
40								

FIGURE A- 1

DATE DRILLED 9/26/25 BORING NO. B-2

GROUND ELEVATION 2152' ±(MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow Stem Auger (2R Drilling)

DRIVE WEIGHT 140 lbs. DROP 30-inches

SAMPLED BY ARB LOGGED BY ARB REVIEWED BY MLP

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X						SM	FILL: Reddish brown, moist, medium dense, silty SAND with gravel.
	X						SM	ALLUVIUM: Reddish brown, moist, medium dense, silty SAND with gravel.
		X	24	12.5	107.2		SC-SM	Reddish brown, moist, medium dense, silty clayey SAND; trace gravel.
10		X	52	7.2	126.7		SM	Reddish brown, moist, dense, silty SAND with gravel; trace clay; intensely weathered igneous/metamorphic gravel.
		X	66	6.1	124.5			Reddish yellow.
		X	65	5.8	126.5			Few cobbles.
20								Total Depth = 20.0 feet. Groundwater not encountered during drilling. Backfilled with on-site soils on 9/26/25. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

FIGURE A- 2

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ALMOND STREET IMPROVEMENT PROJECT
RANCHO CUCAMONGA, CALIFORNIA

DATE DRILLED	9/26/25	BORING NO.	B-3
GROUND ELEVATION	2147' ±(MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Hollow Stem Auger (2R Drilling)		
DRIVE WEIGHT	140 lbs.	DROP	30-inches
SAMPLED BY	ARB	LOGGED BY ARB	REVIEWED BY MLP

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X						SM	FILL: Yellowish brown, moist, medium dense, silty SAND with gravel.
5		X	50/3"					Very dense.
								Encountered unmarked water pipeline and terminated drilling.
								Total Depth = 7.0 feet. Groundwater not encountered during drilling. Water pipeline owner performed utility repair and backfilled on 9/26/25.
10								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								
30								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 9/26/25 BORING NO. B-4 GROUND ELEVATION 2149' ±(MSL) SHEET 1 OF 1 METHOD OF DRILLING 4" Hand Auger (2R Drilling) DRIVE WEIGHT N/A DROP N/A SAMPLED BY ARB LOGGED BY ARB REVIEWED BY MLP DESCRIPTION/INTERPRETATION
0							SM	FILL: Yellowish brown, moist, medium dense, silty SAND with gravel; few cobbles. Refusal on cobbles/boulders.
								Total Depth = 6.0 feet. Groundwater not encountered during drilling. Backfilled with on-site soils on 9/26/25. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
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APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

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

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

200 Wash
An evaluation of the percentage of particles finer than the No. 200 sieve on selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-1.

Direct Shear Test
Direct shear testing was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected sample. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-2.

R-Value
The resistance value, or R-value, for site soils was evaluated in general accordance with CT 301. A sample was prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test result is summarized on Figure B-3.

Soil Corrosivity Tests
Soil pH and resistivity tests were performed on a representative sample in general accordance with CT 643. The soluble sulfate content and chloride content of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-4.

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	0.0-5.0	SILTY SAND WITH GRAVEL	80	17	SM
B-1	10.0-11.5	CLAYEY SAND	86	34	SC
B-2	0.0-4.5	SILTY SAND WITH GRAVEL	78	23	SM
B-2	5.0-6.5	SILTY, CLAYEY SAND	97	43	SC-SM
B-2	10.0-11.5	SILTY SAND WITH GRAVEL	75	23	SM
B-3	0.0-5.0	SILTY SAND WITH GRAVEL	66	14	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-1

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NO. 200 SIEVE ANALYSIS TEST RESULTS

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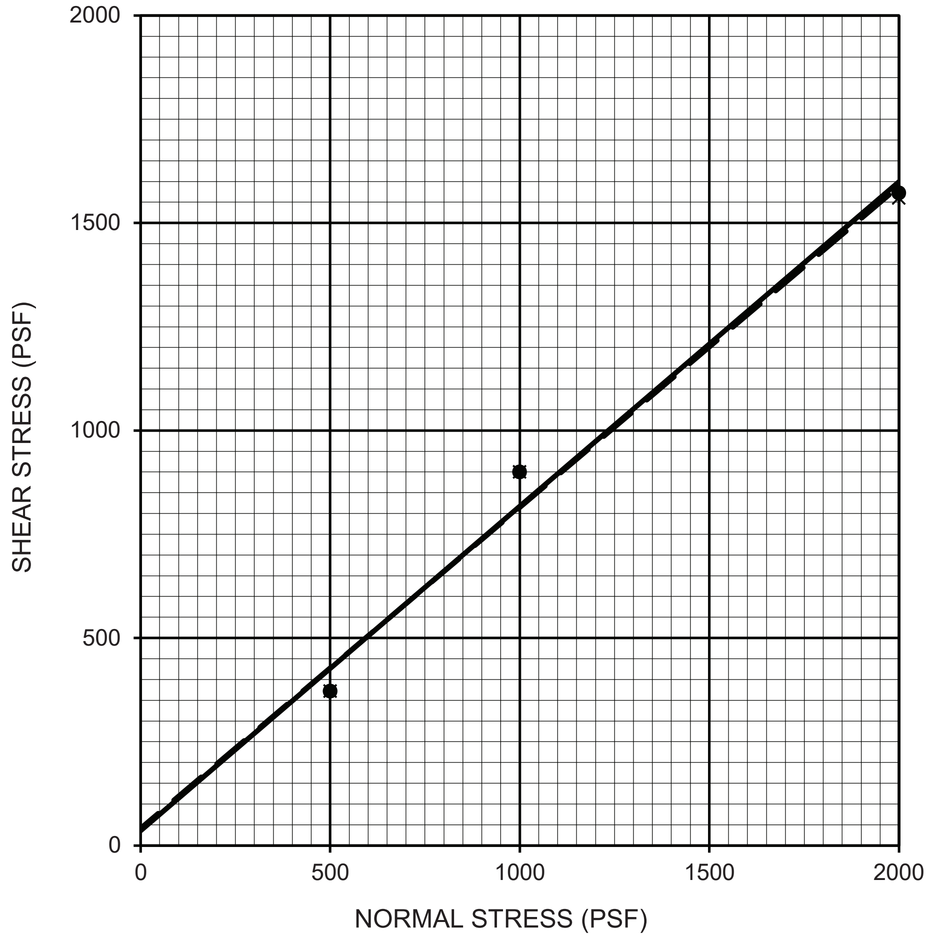

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SILTY, CLAYEY SAND	——●——	B-2	5.0-6.5	Peak	36	38	SC-SM
SILTY, CLAYEY SAND	– – X – –	B-2	5.0-6.5	Ultimate	42	38	SC-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-2

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DIRECT SHEAR TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-2	0.0-4.5	SM	73

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-3

R-VALUE TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-2	0.0-4.5	7.5	5,175	10	0.001	20

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE B-4

CORROSIVITY TEST RESULTS

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