Appendices

Appendix E Geotechnical Exploration Report

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GEOTECHNICAL EXPLORATION REPORT
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
1401 QUAIL STREET
NEWPORT BEACH, ORANGE COUNTY, CALIFORNIA

Prepared for **INTRACORP SOCAL 1, LLC**
895 DOVE STREET, SUITE 400
NEWPORT BEACH, CALIFORNIA 92660

Prepared by **LEIGHTON AND ASSOCIATES, INC.**
17781 COWAN
IRVINE, CALIFORNIA 92614

Project Number 13542.001

June 23, 2022

Leighton and Associates, Inc.
A Leighton Group Company

June 23, 2022

Project No. 13542.001

Intracorp SoCal 1, LLC
895 Dove Street, Suite 400
Newport Beach, California 92660

Attention: Mr. Rick Puffer, Vice President Development

Subject: Geotechnical Exploration Report
Proposed Multi-Family Residential Development
1401 Quail Street
Newport Beach, Orange County, California

In accordance with our April 29, 2022 proposal, authorized on May 25, 2022, Leighton and Associates, Inc. (Leighton) has completed geotechnical exploration for the subject project. We understand from review of BSB *Concept Site Plan* 30 Units Quail Street SFD, dated April 25, 2022, that one option for site development consists of 30 at grade multi-family, 3-story residential buildings with rooftop decks, surface parking and drive aisles connecting Quail Street and Spruce Avenue. Review of RHA *Quail Street Podium,* dated May 5, 2022 indicates a second option for site development consists of a podium concept with 70 units (5 over 2) with one level subterranean parking. In addition, we understand that drywells are being considered at the project site for stormwater BMPs. Ancillary improvements are anticipated to consist of utility infrastructure, flatwork, and landscaping.

The purpose of our geotechnical exploration was to evaluate the subsurface conditions at the site, identify potential geologic and seismic hazards that may impact the project, and provide geotechnical recommendations for design and construction of the proposed project as currently planned.

The project is considered feasible from a geotechnical standpoint. The results of our exploration, conclusions and recommendations are presented in this report. Once the design concept is identified and building loads are know they should be provided to Leighton for review to ensure the recommendations contained herein remain applicable for the proposed concept.

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

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

Joe A. Roe, PG, CEG 2456
Senior Principal Geologist
Extension 4263, jroe@leightongroup.com

Ronald A. Reed, PE, GE
Associate Engineer
Extension 4256, rreed@leightongroup.com

JAR/RAR/lr

Distribution: (1) Addressee

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located at 1401 Quail Street in Newport Beach, California. The site location (latitude 33.66035°, longitude -117.86847°) and immediate vicinity are shown on Figure 1, *Site Location Map.*

Site Description: The approximately 1.7-acre project site is rectangular in shape and contains a single-story commercial building (circa 1972) currently used as a Credit Union. Perimeter paved parking is located to the west and south of the structure with a solar panel parking cover in the southern portion. A landscape greenbelt borders the north and east sides of the structure.

Aerial Imagery Review: Based on review of available historical aerial photographs (NETR, 1938-1963), the site was vacant undeveloped land until approximately early 1970's. By 1972, we observe the existing building and perimeter roads were being constructed at the site with paved surface parking; by approximately 1980 the site is in the current configuration as observed today.

Proposed Development: We understand several concepts are being considered the layouts of which are presented on Figures 2a and 2b, *Geotechnical Maps.* Based on review of *Concept Site Plan 30 Units Quail Street SFD*, prepared by BSB, dated April 25, 2022, site development consists of 30 at grade multi-family, 3-story residential buildings with rooftop decks, surface parking and drive aisles connecting Quail Street and Spruce Avenue, see Figure 2a, *Geotechnical Map* for concept layout. Second consideration is known from review of *Quail Street Podium,* prepared by RHA, dated May 5, 2022, which indicates site development is considering a podium concept with 70 units (5 over 2) with one level subterranean parking, see Figure 2b, *Geotechnical Map*. In addition, we understand that drywells are being considered at the project site for stormwater BMPs. Ancillary improvements are anticipated to consist of utility infrastructure, flatwork, and landscaping. Preliminary structural loading information for both concepts was not yet available at the time this report was prepared. Once the concept is selected and the design progresses, building loads should be provided for our review to determine the settlement estimates provided herein remain applicable.

1.2 Purpose and Scope

Purpose of our geotechnical exploration was to evaluate the subsurface conditions at the site relative to the proposed concepts and provide preliminary geotechnical recommendations to aid in the design and construction for the projects as currently described above. In accordance with our April 29, 2022 proposal authorized on May 25, 2022, our Scope of Work included the following:

- *Research* – We reviewed readily available and provided literature aerial photographs, and maps relevant to the site. We evaluated geological hazards and potential geotechnical issues that may significantly impact the site. The documents reviewed are listed in Section 5.0 *References*.

- *Pre-Field Exploration Activities* – Reconnaissance of the site was performed by a certified engineering geologist to mark the proposed exploration locations. Underground Service Alert (USA) was notified to locate and mark existing underground utilities prior to our subsurface exploration.

- *Field Exploration* – Our subsurface exploration (soil borings) was performed on May 20, 2022, and included drilling, logging, and sampling of three (3) hollow-stem auger borings (designated LB-1, LB-2, and LB-3). LB-1 was drilled to a depth of approximately 51.5 feet, while LB-2 and LB-3 were both drilled to 26.5 feet below the existing ground surface (bgs). One (1) additional boring (designated LP-1) was drilled to an approximate depth of 21.5 feet bgs for subsequent percolation testing. Approximate location of these explorations are shown on Figures 2a and 2b, *Geotechnical Map* and corresponding boring logs are presented in Appendix A, *Exploration Logs.*

  During drilling of the hollow-stem auger borings both bulk and drive samples were obtained from the borings for geotechnical laboratory testing. Driven ring samples were collected from the borings using a Modified California ring-lined sampler conducted in accordance with ASTM Test Method D 3550. Standard Penetration Tests (SPTs) were also performed within the borings in accordance with ASTM Test Method D 1586. Samples were collected at 2½ and 5-foot intervals throughout the depth of exploration. In both test methods, the sampler is driven below the bottom of the borehole by a 140-pound weight (hammer) free-falling 30 inches. The drilling rig was equipped with an automatic hammer to provide greater consistency in the drop height and striking frequency. The number of blows to drive the sampler the final 12-inches of the 18-inch drive interval is termed the “blowcount” or SPT N-value. N-values provide a measure of relative density in granular (non-cohesive) soils and comparative consistency in cohesive soils. Number of blows per 6 inches of penetration was recorded on the boring logs included in Appendix A.

The borings were logged in the field by a geologist from our firm under supervision of a certified engineering geologist. Each soil sample collected was reviewed and described in accordance with the Unified Soil Classification System (USCS). The samples were sealed and packaged for transportation to our laboratory for testing. After completion of drilling, the borings were backfilled to the ground surface with soil cuttings and patched with cold-patch asphalt. Excess soil cuttings from the borings were spread onsite in planter areas.

- *Percolation Testing* – Boring LP-1 (Figures 2a and 2b) was converted to a temporary percolation test well upon completion of drilling and sampling. The test well consisted of 2-inch slotted (0.020-inch slots) PVC well casing surrounded by No. 3 Monterey Sand placed in the annulus of the well within the test zone. In-situ percolation testing was performed on May 20, 2022 in general accordance with the *Orange County Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Programs (WQMPs)* (OCPW, 2013). The results of the percolation testing are presented in Appendix B, *Percolation Test Data*. Refer to the discussion of infiltration rate presented in Section 2.3.1, *Infiltration*.

- *Laboratory Testing* – Selected relatively undisturbed and bulk soil samples obtained from our current hollow-stem-auger borings were tested at our in-house Irvine (DSA LEA 063) geotechnical laboratory. This laboratory testing program was designed to evaluate physical geotechnical characteristics of site soils including corrosion potential. Geotechnical test results are presented in Appendix C, *Geotechnical Laboratory Testing*. Tests performed during this investigation include:

  - In-situ Moisture Content and Dry Density (ASTM D 2216 and ASTM D 2937);
  - Expansion Index (ASTM D 4829);
  - Maximum Dry Density (ASTM D 1557);
  - Direct Shear (ASTM D 3080);
  - Particle Size Analysis (ASTM D 422);
  - Consolidation (ASTM D 2435); and

- Corrosivity Suite – pH, Sulfate, Chloride, and Resistivity (California Test Methods 417, 422, and 532/643).

Results of the in-situ moisture content and dry density testing are presented on the boring logs in Appendix A.

- *Engineering Analysis* – Data obtained from these borings and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and recommendations for proposed conceptual improvements described in Section 1.1 of this report.

- *Report Preparation* – This report presents our findings, conclusions, and preliminary recommendations for the proposed development.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

The project site is located on western edge of San Diego Creek drainage within Upper Newport Bay coastal estuary near the San Joaquin Hills. The Newport Bay estuary was originally formed as the lower reach of the Santa Ana River. However, due to extensive widespread flooding in 1915-1916, the Santa Ana River realigned its course to the west. The bay is currently fed only by San Diego Creek and its tributaries. The San Joaquin Hills lie within the northern part of the Peninsular Ranges geomorphic province which extends 900 miles southward from the Santa Monica Mountains to the tip of Baja California (Yerkes, et al., 1965). Regional tectonic activity has uplifted the San Joaquin Hills into an elongated arched fold (anticlinorium) trending to the northwest from San Juan Capistrano and Huntington Mesa. This anticlinal folding has occurred as this entire section of the southern California coast was uplifted by the San Joaquin Hills blind thrust fault (Grant et at., 1997, 1999, and 2002; Mueller et al., 1998). The geology in the vicinity of the project site is shown in Figure 3, *Regional Geology Map*.

2.2 Subsurface Conditions

Based on interpretation of samples recovered during the subsurface exploration (Figure 2a and 2b), the site is underlain by four to five feet of undocumented artificial fill overlying Quaternary-age old alluvial fan deposits and Quaternary-age old lacustrine, playa, and estuarine deposits. A general description of the earth materials as encountered are described below:

Undocumented Artificial Fill (Afu): The existing near-surface artificial fill soils encountered in our exploratory borings are considered undocumented, and unsuitable for foundation support due to the uncontrolled nature of these fill soils during placement. The undocumented artificial fill materials encountered in our borings (Figures 2a and 2b) range in thickness from approximately 4 to 5 feet bgs at the explored locations. These soils are characterized as reddish brown and yellowish brown, slightly moist, silty sand (SM). While the silty sand fill material was encountered at each location the overall deposit may not be continuous across the site. The composition of the earth material below the building is unknown.

Quaternary Age-Old Alluvial Fan Deposits (Qof): The alluvial fan deposits encountered beneath the fill materials in our exploratory borings generally consist of alternating beds of orange brown, reddish brown and gray brown, and slightly moist to moist very stiff carbonate impacted sandy clay (CL), silty sand (SM), and poorly graded sand (SP). Poorly-graded sand is notably thickest at 25 feet and deeper, with increased shell fragment percentage at depth.

Quaternary Old Lacustrine, Playa, and Estuarine Deposits (Qol): The Quaternary age lacustrine, playa, and estuarine deposits beneath the Qof in our exploratory boring LB-1 at 40.5 consists of a bluish gray fat clay, very moist, and containing trace shell fragments. This clay extends to at least 51.5 feet bgs.

The stratigraphy of the subsurface soils encountered in each soil boring is presented on the boring logs (Appendix A).

2.2.1 <u>Expansive Soil Characteristics</u>

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

One bulk soil sample obtained during our subsurface exploration from boring LB-1 (0-5 ft bgs) and one ring sample from boring LB-2 (15 ft bgs) were tested for expansion potential and Atterberg limit testing, respectively. The test results indicate an Expansion Index (EI) value of 1 from boring LB-1 indicate a very low" potential for expansion of the upper five feet of silty sand interpreted as undocumented fill.

The sample tested for Atterberg limits at the approximate foundation level for the subterranean option indicated (15 ft bgs) indicate fat clay will be exposed at the foundation bearing zone. This data suggests the clays at this depth fall into the High Expansion category. Laboratory test results are included in Appendix C of this report.

Additional testing should be performed upon completion of site grading and excavation to confirm the expansion potential presented in this report. For purposes of this report, and based upon visual characterization of the overall granular undocumented fill in the upper four (4) feet may be considered to support design of Concept 1 (BSB, April 2022) with a low expansion index. Highly expansive fat clays are expected to be encountered at the subterranean level for Concept 2 (RHA, May 2022).

2.2.2 Soil Corrosivity

One bulk soil sample obtained during our subsurface exploration from boring LB-1 was tested for corrosivity to assess corrosion potential to buried concrete. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix C of this report.

The test results indicate *Soluble Sulfate* concentration of 366 parts per million (ppm), *Chloride* content of 80 ppm, *pH* value of 7.65, and *Minimum Resistivity* value of 1,590 ohm-cm.

The results of the resistivity test indicate the underlying soil is severely to corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate contents from the soil sample of the near surface granular material, concrete in contact with the soil is expected to have negligible exposure to sulfate attack (S0) per ACI 318 (ACI, 2014). The samples tested for water-soluble chloride content indicate a low potential for corrosion of steel in concrete (C1) due to the chloride content of the soil.

Additional sampling of site soil upon completion of grading is recommended to confirm the values presented in this report.

2.2.3 Soil Compressibility

One (1) samples of the onsite soils recovered from the borings was subjected to consolidation testing to evaluate the compressibility of the materials under assumed loads representative of anticipated structural bearing stresses. The results of testing indicate the tested soil exhibit a low to moderate compressibility potential. The test result is presented in Appendix C.

2.2.4 Shear Strength

Evaluation of the shear strength characteristics of the soils included laboratory direct shear testing. The results of testing are included in

Appendix C as well as summary graphs that provide values of angle of internal friction (ø) and cohesion (c) for use in geotechnical analysis.

2.2.5 Excavation Characteristics

Based on our subsurface explorations performed at the site and our experience from grading jobs in the vicinity of the site, we anticipate the onsite artificial fill and native earth materials can generally be excavated using conventional excavation equipment in good operating condition.

2.3 Groundwater Conditions

Groundwater was encountered in our subsurface investigation at a depth of 25 feet bgs in borings LB-1, LB-2, and LB-3, and was present up to depths of 51.5 feet bgs. Clay soils expected at or near the foundation level (±15 feet bgs) for Concept 2 (RHA, 2022) display moisture contents ranging from 21 to 24 percent. These soils when subject to increase in moisture conditions may cause subgrade pumping during overexcavation.

Based on these findings, groundwater is not expected to pose a constraint during or after construction. Fluctuations of the groundwater level, localized zones of perched water, and an increase in soil moisture, should be anticipated during and following the rainy seasons or periods of locally intense rainfall or storm water runoff.

2.3.1 Infiltration

Percolation testing was performed within a temporary percolation test well installed within boring LP-1 (Figures 2a and 2b) to evaluate the infiltration characteristics of subsurface soils. The percolation test was conducted in general accordance with the *Orange County Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Programs (WQMPs)* (OCPW, 2013). Results of the percolation testing are presented in Appendix B, *Percolation Test Data*. The test location, test zone and corresponding infiltration rate are shown on Figure 2a and 2b, *Exploration Location Map*.

A boring percolation test is useful for field measurements of the infiltration rate of soils, and is suited for testing when the design depth of the infiltration device is deeper than current existing grades, especially in areas where it is difficult to dig test pits, or where the depths of these test pits would be considerably deep. At the subject site, testing consisted of advancing the boring to general depths anticipated for the invert of typical infiltration devices, approximately 15-20 feet below grade.

The test was performed using a constant-head method which records the approximate volume of water delivered to the test zone while maintaining a relatively constant height of water in the well over the testing period. Since the subsurface materials were generally favorable for percolation (sandy clay soils), a water source was used to deliver water to the well at a relatively constant rate while recording the water height in the well. The measured infiltration rate was calculated by dividing the total volume of water infiltrated by the total duration of the test and dividing by the percolation surface area. Detailed results of the field testing data and measured infiltration rate for the test wells are presented in Appendix B. The test result is summarized below:

Table 1 – Measured (Unfactored) Infiltration Rate

Test Well Designation	Approximate Depth of Test Zone (feet bgs)	Approximate Depth to Groundwater (feet bgs)	Measured Infiltration Rate (inches per hour)
LP-1	15 to 20	25	6.07

The results of the percolation tests indicate a favorable rate of infiltration at the specific location and depth tested. The measured infiltration rate is the result of small-scale test performed at specific location and depth, see Figures 2a and 2b, *Geotechnical Maps*. Static groundwater was measured at a depth of 25 feet in the three other borings (LB-1, LB-2 and LB-3). According to the *Orange County Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Programs (WQMPs)* (OCPW, 2013) infiltration must maintain a depth of at least 10 feet above static groundwater levels.

Based on the depth to groundwater (approx. 25 ft bgs) infiltration at these depths (15-20 feet) is not considered feasible. Clay soils of low permeability onsite extend from a depth of approximately 5 to 20 feet below grade with intermittent and laterally discontinuous interbeds of silty sand and while the *interbedded sand may support infiltration based on the lateral discontinuity, we do not recommend infiltration for this site*.

2.4 Surface Fault Rupture

Our review of available literature (geologic maps, aerial photos) indicates that no known active faults have been mapped across the site, and the site is **not** located within a currently established *Alquist-Priolo Earthquake Fault Zone* (Bryant and Hart, 2007). Therefore, the potential for surface fault rupture at the site is expected to be low and a surface fault rupture hazard evaluation is not mandated for this site.

The location of the closest active faults to the site was evaluated using the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008). The closest active fault to the site with the potential for surface fault rupture is the Newport-Inglewood Fault Zone (NIFZ), located approximately 4.9 miles from the site. The San Andreas fault, which is the largest active fault in California, is approximately 40 miles northeast of the site. Major regional faults with surface expression in proximity to the site are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

2.5 Strong Ground Shaking

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California (Figure 4). The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics.

Accordingly, design of the project should be performed 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). The 2019 edition of the California Building Code (CBC) is the current edition of the code. 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 code-based seismic parameters should be considered for design under the 2019 CBC:

Table 2 – 2019 CBC Based Ground Motion Parameters (Mapped Values)

Categorization Coefficient	Code-Based
Site Latitude	33.66035°
Site Longitude	-117.86847°
Site Class	D
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_S	1.297
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.463
Short Period (0.2 sec) Site Coefficient, F_a	1.0
Long Period (1 sec) Site Coefficient, F_v	1.837
Adjusted Spectral Response Acceleration at Short Period (0.2 sec), S_{MS}	1.297
Adjusted Spectral Response Acceleration at Long Period (1 sec), S_{M1}	0.851
Design Spectral Response Acceleration at Short Period (0.2 sec), S_{DS}	0.865
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	0.567
Site-adjusted geometric mean Peak Ground Acceleration, PGA_M	0.611

[1]Per Exception 2 in Section 11.4.8 of ASCE 7-16, seismic response coefficient C_S to be determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5T_s$ or Eq. 12.8-4 for $T > T_L$

2.6 Liquefaction Potential

The term liquefaction is generally referenced to loss of strength and stiffness in soils due to build-up of pore water pressure when subject to cyclic or monotonic loading. Both sandy and clayey soils are susceptible to loss of strength and stiffness. Because of the difference in strength characteristic and methods for evaluating strength loss potential for granular and clayey soils, the term liquefaction is used for granular soils while cyclic softening is used for fine-grained soils (i.e. clays and plastic silts).

In general, adverse effects of liquefaction or cyclic softening include excessive ground settlement, loss of bearing support for structural foundations, and seismically-induced lateral ground deformations such as lateral spreading. Depending upon the relative thickness of the liquefied strata with respect to overlying non-liquefiable soils, other potentially adverse effects such as ground oscillation and ground fissuring may occur.

As shown on the *Seismic Hazard Zones* map for the Anaheim Quadrangle (CGS, 1998), the project site is **not** located within an area that has been identified by the State of California as being potentially susceptible to liquefaction (Figure 5, *Seismic Hazard Map*). Pleistocene age low plasticity clayey soil below the site is very stiff and impacted with carbonate, soils of this type are generally not subject to the adverse effects of liquefaction. Based on these findings and blow counts of encountered soils, liquefaction is not considered a hazard at the site.

2.7 Seismically-Induced Settlement

Seismically-induced settlement consists of dynamic settlement of unsaturated soil (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within low density sandy soil due to reduction in volume during and shortly after an earthquake event.

Based on our evaluation of the site soils, the total seismically-induced settlement is estimated to be less than ½ inch. The differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet.

2.8 Lateral Spreading

Liquefaction may also cause lateral spreading. For lateral spreading to occur, the liquefiable zone must be continuous, unconstrained laterally, and free to move along gently sloping ground toward an unconfined area. Since the site is relatively flat and constrained laterally, earthquake-induced lateral spreading is not considered a hazard at the site.

2.9 Earthquake-Induced Landsliding

As shown on Figure 5, the site is **not** mapped within a seismically-induced landslide hazard zone identified by the State of California (CGS, 1998). In addition, due to project site being relatively flat, it is our opinion that the potential for seismically-induced landslide hazard at the site is negligible.

2.10 Storm-Induced Flooding

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map 06059C02861 dated December 3, 2009 (FEMA, 2009), the project site is not located within a flood hazard area.

3.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

Based on this study, we conclude that the proposed development concepts as described in Section 1.1 of this report are considered feasible from a geotechnical standpoint, provided that the recommendations presented in this report are properly incorporated in design and construction.

All existing undocumented fill is recommended to be removed from the proposed building/structure footprint areas to expose suitable native soils then reworked (moisture conditioned) prior to placement as compacted engineered fill. Undocumented fill is considered suitable for reuse from a geotechnical standpoint provided it is properly moisture conditioned and placed under engineering-controlled conditions.

The recommendations below are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. The 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 determine the effect upon the recommendations subsequently presented. These recommendations are considered minimal and may be superseded by more restrictive requirements of the civil and structural engineers, the City of Newport Beach and other governing agencies.

Once the concept is selected and plans developed, Leighton should review the grading plans, foundation plans and project specifications as they become available to verify that the recommendations presented in this report have been incorporated into the plans for this project.

3.1 Site Grading

Earthwork guide specifications are presented in Appendix D, *Earthwork and Grading Guide Specifications*. Earthwork for Concept 1 (BSB, April 2022) is expected to include overexcavation and recompaction of low expansion undocumented fill soils below new improvement footprints. Earthwork for Concept 2 (RHA, May 2022) expected to consist of excavation of the subterranean parking level, foundation excavation, backfill of the basement walls, and other site improvement work. Recommendations for site earthwork are provided in the following sections.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of any vegetation, trash, and/or debris within the area of proposed grading. These materials should be removed from the site. After the site is cleared, the soils should be carefully observed for the removal of all unsuitable deposits. All undocumented fill or man-made debris, should be removed, reworked and replaced as engineered fill.

3.1.2 Removals and Overexcavations

To provide uniform foundation support and reduce the potential for excessive static settlement, all existing undocumented fill and any unsuitable soil, as deemed by the geotechnical engineer, should be removed to expose suitable native soils and replaced as engineered fill below the proposed building and parking structure footprints and other structural improvements. Based on our field explorations, we estimate removals of existing undocumented fill will be approximately 4 to 5 feet below existing grade across most of the site, with localized areas that may require deeper removals.

The lateral extent of removals and overexcavations beyond foundations should be equal to the depth of removals and overexcavations below the proposed foundations where practical. Localized areas in the unexplored portions of the site should be anticipated to possibly require deeper removals depending on observed subsurface conditions and thickness of undocumented fill evaluated during grading of the site.

Any underground obstructions encountered should be removed. Efforts should be made to locate any existing utility lines. Those lines should be removed or rerouted were interfering with proposed new foundations. Should the subterranean concept be chosen all fill is expected to be removed during excavation for the subterranean garage. The silty sand (SM) that makes up the very low expansion fill may be stockpiled and reused onsite as fill below flatwork and other slab on grade.

3.1.3 Excavation Bottom Preparation

Resulting removal excavation bottom-surfaces should be observed by Leighton prior to placement of any backfill or new construction. After these over-excavations are completed, and prior to fill placement, exposed surfaces should be scarified to a minimum depth of 6 inches, moisture-conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction as determined by ASTM D1557 standard test method (modified Proctor compaction curve).

3.1.4 Subgrade Stabilization

It is important to recognize instability conditions may vary laterally from where observations are made during field implementation or may change from repetitive loading by heavy vehicles. Based on our findings from the boring explorations (Appendix A) saturated subgrade conditions (moisture contents ranging from 21 to 24 percent) are expected to exist across the entire footprint of the planned subterranean structure (RHA, May 2022).. Adjustment to the limits should be anticipated based on observed performance during stabilization. The stabilization methodology may vary and it is the contractor's responsibility to achieve a non-yielding compacted subgrade prior to fill placement or foundation construction. While the laboratory-indicated moisture contents alone may not cause subgrade instability, the exposed moisture conditions may vary from what is currently reported. As such, we provide this information for planning purposes. The following proven geotechnical solution may be considered should subgrade instability occur during grading.

***Rock Stabilization**:* After removal of alluvial soils if saturated subgrade conditions exits, a 3- to 4-inch layer of 2- to 3-inch crushed rock should be placed in the excavation. Rock should be mechanically compacted under the weight of the equipment to push the rock into the underlying clay soils. Vibratory equipment should not be used to work in the rock blanket as the vibrations may aggravate locally soft saturated clays causing pumping conditions to expand laterally and destabilize the subgrade further. Clay soils removed from the excavation will require drying prior to reuse and are not considered suitable for use behind retaining walls.

Depending upon the degree of subgrade instability, should it occur, the initial lift may completely penetrate the subgrade, and additional lifts will be necessary. Alternatively, the quantity of material may be reduced if a geogrid or geotextile fabric is considered to provide additional reinforcement effect after the placement of the initial lift. Geogrid or geotextile reinforcement should be placed with a minimum 3 feet of overlap between adjacent panels extending a distance of at least 5 feet beyond the footprint on all sides.

3.1.5 Fill Materials

On-site soil that is free of construction debris, organics, or rock larger than 4 inches in largest dimension is suitable to be used as fill for support of structures. Onsite clayey soils with an expansion index of 21 or greater (EI≥21) should not be used within 2 feet of concrete slabs-on-grade to avoid potential for lightly loaded concrete slabs to heave. Any imported fill soil should be approved by the geotechnical engineer prior to import or use onsite. Import soils should be uncontaminated, granular in nature, free of organic material (loss on ignition less than 2 percent), have a very low expansion potential (with an Expansion Index less than 21) and have a low corrosion impact to the proposed improvements.

Because of the expansive nature of the onsite clay soils observed and documented in the borings (Appendix A) below the granular fill, precautions should be taken to reduce the potential heaving of concrete slabs on grade if clay soil is exposed in the subgrade. A layer of relatively non-expansive, predominantly granular soils is recommended immediately beneath concrete walks and slabs on grade, including Portland cement concrete paving. This select, non-expansive granular soil should contain sufficient fines as to be relatively impermeable when compacted. Material of this type was observed onsite within the undocumented fill encountered at the boring locations. This granular undocumented fill material may be reused onsite since the expansion potential of the undocumented fill observed at the boring locations had an Expansion Index (EI) of one (1), essentially non-expansive.

3.1.6 Fill Placement and Compaction

Fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned to at least 2 percent above optimum moisture content, and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D 1557 (modified Proctor compaction curve). Aggregate base should also be compacted to a minimum of 95 percent relative compaction.

3.1.7 Shrinkage

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. Field and laboratory data used in our calculations included laboratory-measured maximum dry density for the general soil type encountered at the subject site, the measured in-place densities of near surface soils encountered and our experience.

Based upon the results of the in-place density of native alluvial soils and engineered fill and the moisture-density relationship exhibited by representative bulk samples of the near surface soils, recompaction of the soils is anticipated to result in volume shrinkage in the range of 5 to 10 percent. The estimated shrinkage does not include material losses due to removal of organic material or other unsuitable bearing materials (debris, rubble, oversize material greater than 6-inches) and the actual shrinkage that occurs during grading may vary throughout the site.

3.2 Foundation Design

The proposed three-story residential buildings (BSB, April 2022) can be supported on Post Tension (PT) foundations. Maximum column loading and wall loading is not available at the time of this report. We have anticipated that the proposed residential buildings will be wood-framed and lightly loaded. We assume a maximum column load of 45 kips and maximum wall load of 2.5 kips per lineal foot are generally applicable for the relatively light residential structural loads. Structural loading information should be provided to us when available for review.

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

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment per code requirements, with a minimum width of 24 and 18 inches for isolated and continuous footings, respectively.

3.2.2 Post-Tension Foundation Design Parameters

Post-tensioned foundations founded on reconditioned undocumented fill of very low expansion should be designed by a qualified structural engineer in accordance with the 2019 CBC using the minimum geotechnical parameters provided below for soils with a "low" Expansion Index. Expansion index should be confirmed upon completion of grading.

Post-tensioned Foundation Design Recommendations		
		Low
Edge Moisture Variation, e_m	Center Lift	9.0
	Edge Lift	5.1
Differential Swell Y_m	Center Lift	0.15
	Edge Lift	0.36
Modulus of Subgrade Reaction		120 pci

For post-tension slab foundations, exterior footings (thickened edges) should have a minimum depth of 18 inches below the lowest adjacent soil grade and a minimum width of 12 inches. These footings may be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot. The allowable bearing pressure may be increased by one-third for short-term loading. The structural engineer should provide the slab with adequate stiffness to minimize potential cracking due to expansive forces. The design of post-tensioned slab foundations should follow the procedures described in the latest edition of the Design of Post-Tensioned Slabs-on-Ground by the Post-Tensioning Institute.

To provide more uniform moisture in the subgrade, the top 18 inches of the prepared subgrade should be moisture conditioned to 1 to 2 percent of the optimum moisture prior to placement of concrete.

The soil-moisture around the immediate perimeter of the slab should be maintained to near-optimum moisture content (or above) during construction and up to occupancy of the homes.

The Post-Tensioning Institute (PTI) has recommended the following guidelines for residential development:

- Initial landscaping should be done on all sides adjacent to the foundation. Positive drainage away from the foundation should be implemented and maintained.

- Irrigation watering should be done in a uniform manner as equally as possible on all sides of the foundation to maintain constant soil moisture content. Ponding of irrigation or rainfall water adjacent to the foundation slab can cause differential soil moisture levels potentially causing differential movements.

  Planting trees closer to the structure than a distance equal to one-half the mature height of the tree could allow the root system to enter under the foundation. The root system could alter the soil moisture content within the root system and cause soil shrinkage which may result in differential movements of the foundation. A landscape architect should be consulted and made aware of these recommendations.

We recommend additional Expansion Index tests be conducted prior to the residential construction phase. The above recommended design criteria may be subject to change if the expansion potential of the subgrade soil is found to be different than assumed herein.

3.2.3 Conventional Spread Footings

It is anticipated that conventional spread footings can be used to support the proposed subterranean parking structure and other isolated columns. Footings should be embedded at a minimum 18 inches below the lowest adjacent grade. For footings in the upper silty sand materials, (i.e. the upper 5 feet) an allowable soil bearing pressure of 3,000 psf may be used for footings with a minimum width of 12 inches. Footings for the proposed subterranean parking structure would likely be located in the clayey materials that were encountered at depth in the borings. For footings in the clayey materials, an allowable soil bearing pressure of 2,500 psf may be used for footings with a minimum width of 12 inches. These bearing capacities may be increased by one-third for wind or seismic loading.

The total static settlement induced by a column load of 800 kips is estimated to be 1½ inch. For a column load of 150 kips, the total static settlement is estimated to be ½ inch. Differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet. Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. Leighton should review the settlement estimates when final foundation plans and loads for the proposed structures become available. If upon review, the settlement estimates exceed tolerable values, then the use of structural slab system with grade beams connecting the footings or a mat foundation may be recommended. Additional investigation may also be considered.

Resistance to lateral loads will be provided by a combination of friction between the soil and structure interface and passive pressure acting against the vertical portion of the footings structures. For calculating lateral resistance of footings located in the upper silty sand materials (i.e. the upper 5 feet), a passive pressure of 300 psf per foot of depth to a maximum of 3,000 psf and a frictional coefficient of 0.30 may be used. For footings located in the deeper clayey materials, a passive pressure of 200 psf per foot of depth to a maximum of 2,000 psf and a frictional coefficient of 0.20 may be used. Note that the passive and frictional coefficients do not include a factor of safety. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

3.3 Slabs-on-Grade

Concrete slabs may be designed using a modulus of subgrade reaction of 100 pci provided the subgrade is prepared as described in Section 3.1. From a geotechnical standpoint, we recommend slab-on-grade be a minimum 4 inches thick with No. 3 rebar placed at the center of the slab at 24 inches on center in each direction for *Concept Site Plan 30 Units Quail Street SFD*, prepared by BSB, dated April 25, 2022. For the subterranean concept*, Quail Street Podium,* prepared by RHA, dated May 5, 2022 and in consideration of the highly expansive clay anticipated to be exposed at the bearing elevation of the parking structure we recommend slab-on-grade be a minimum 8 inches thick. The structural engineer should design the actual thickness and reinforcement based on anticipated loading conditions. Where moisture-sensitive floor coverings or equipment is planned, the slabs should be protected by a minimum 10-mil-thick vapor barrier between the slab and subgrade. A coefficient of friction of 0.35 can be used between the floor slab and the vapor barrier.

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected; however, concrete 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. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking. Additionally, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential but not eliminate for concrete cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panels.

3.4 Sulfate Attack and Ferrous Corrosion Protection

3.4.1 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 2019 California Building Code (CBC) defers to the American Concrete Institute's (ACI's) ACI 318-14 for concrete durability requirements. Table 19.3.1.1 of ACI 318-14 lists "*Exposure categories and classes*," including sulfate exposure as follows:

Table 3 - Sulfate Concentration and Exposure

Water-Soluble Sulfate (SO_4) in soil (percentage by weight)	ACI 318-14 Sulfate Class
0.00 - 0.10	S0 (negligible)
0.10 - 0.20	S1 (moderate*)
0.20 - 2.00	S2 (severe)
>2.00	S3 (very severe)

*or seawater

3.4.2 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 4 - 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.

3.4.3 Corrosivity Test Results

To evaluate corrosion potential of near surface 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 5 - Results of Corrosivity Testing

Boring Number	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
LB-1	0-5	366	80	7.65	1,590

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

These results are discussed as follows:

- **Sulfate Exposure**: Based on test results and Table 19.3.1.1 of ACI 318-14, in our opinion, sulfate exposure should be considered "**moderate**" with an **Exposure Class S0**. *Additional testing of subgrade materials recommended upon completion of grading*.
- **Ferrous Corrosivity**: As shown above, minimum soil resistivity of **1,590 ohm-centimeters** was measured in our laboratory test. In our opinion, based on resistivity correlation presented in Table 4 Section 3.4.2, it appears for site soils that corrosion potential to buried steel may be characterized as "**severely corrosive**" at the site.

As standard design concepts, ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Or ferrous pipe can be protected by polyethylene bags, tape or coatings, di-electric fittings or other means to separate the pipe from on-site earth materials. *Additional testing of subgrade materials recommended upon completion of grading to confirm these reported values once rough grade mixing of onsite soils is complete.*

3.5 Retaining Walls

Recommended lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft. or pcf. These 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.

On-site clay soils are not considered suitable to be used as retaining wall backfill due to its expansion potential. Should shallow site soil in upper five characterized as silty sand (SM) of very low expansion potential be considered or available for reuse behind basement retaining walls, it should be tested to ensure Expansion potential is less than 20 (EI<20) after it is stockpiled. Recommended lateral earth pressures for retaining walls backfilled with sandy soils with drained conditions as shown on Figure 6, *Retaining Wall Backfill and Subdrain Detail* are as follows:

Table 6 – Retaining Wall Design Earth Pressures

Retaining Wall Condition (Level Backfill)	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active (cantilever)	35
At-Rest (braced)	60
Passive Resistance (compacted fill)	300
Seismic Increment (add to active pressure)	20

Walls that are free to rotate or deflect may be designed using active earth pressure. For basement walls or walls that are fixed against rotation, the at-rest pressure should be used. For the seismic condition, the pressure should be distributed as a triangular distribution and the dynamic thrust should be applied at a height of 1/3H above the base of the wall.

3.5.1 Sliding and Overturning

Total depth of retained earth for design of walls and for uplift resistance, should be measured as the vertical height of the stem below the ground surface at the wall face for stem design, or measured at the heel of the footing for overturning and sliding. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing, if drained, or 60 pcf if submerged, for properly compacted backfill.

3.5.2 Drainage

Adequate drainage may be provided by a subdrain system positioned behind any earth retaining walls. Typically, this system consists of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with pervious backfill material described in Section 300-3.5.2 of the *Standard Specifications for Public Works Construction* (Green Book), current edition. This pervious backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in Section 300-8.1 of the *Standard Specifications for Public Works Construction* (Green Book), current edition. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage geocomposites, or similar, may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill, particularly where horizontal space is limited adjacent to shoring (where walls are cast against shoring if considered). These drainage panels should be connected to the perforated drainpipe at the base of the wall.

3.6 Pavements

To provide support for paving, the subgrade soils should be prepared as recommended in the Section 3.1. Compaction of the subgrade, including trench backfills, to at least 90 percent of the maximum dry density as determined by ASTM Test Method D 1557, and achieving a firm, hard, and unyielding surface will be important for paving support. The preparation of the paving area subgrade should be performed immediately prior to placement of the base course.

Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet. Cut-off walls achieved by deepening curb sections or grade beams around planters or other comparable barriers are also recommended to minimize lateral flow of irrigation water beneath the adjacent subgrade soils.

Excessive over-irrigation will have an adverse impact on adjacent pavements. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from paving, will result in premature pavement failure.

3.6.1 Asphalt Concrete

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of engineered fill with an R-value of at least 15, compacted to at least 90 percent as recommended, the minimum recommended paving thicknesses are presented in the following table.

Table 7 – Asphalt Concrete Pavement Sections

Design Traffic Index (TI)	Asphalt Concrete (inches)	Base Course (inches)
5	4.0	7.0
6	5.0	9.0
7	6.0	11.0

Traffic Indexes (TIs) used in our pavement design are considered reasonable values for proposed auto parking lots, and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Higher TIs should be used in heavy truck traffic areas or high-volume lanes.

3.6.2 Portland Cement Concrete Paving

For light axle loads and average daily truck traffic (ADT) less than (<) 500, fire lanes subject to outrigger loads, trash corral aprons, or other areas where point loads are possible, should be paved with Portland Cement Concrete (PCC) with a minimum thickness of 6-inches over properly compacted fill. However, for medium/heavy axle loads and an ADT of (≥) 500 or more over properly compacted fill subgrades, a minimum PCC thickness of 7-inches should be used. All PCC pavements should have a minimum 28-day concrete compressive strength of 3,250 pounds-per-square-inch (psi), and have appropriate joints and saw cuts in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. PCC subgrades supporting axle loads are recommended to be compacted to 95 percent relative compaction in the upper 12 inches.

A 4-inch layer of Class 2 aggregate may be used beneath areas of PCC pavement to improve performance. Additional details should be added to plans indicating pavement thickness transitions, pavement joint dowels, expansion joints and saw cut joints. Use of concrete cutoff or edge barriers should be considered at the perimeter of common parking or driveway areas when abutting either open (unfinished) or landscaped areas.

3.6.3 Paving Materials

Asphalt concrete, aggregate base and Portland Cement Concrete (PCC) should conform to *Caltrans Standard Specifications* (current Edition):

https://dot.ca.gov/-/media/dot-media/programs/design/documents/f00203402018stdspecsa11y.pdf

Recommended structural pavement materials should conform to the specified provisions in the 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.
- **Class 2 Aggregate Base (AB)** should conform to Section 26 of the *Standard Specifications*.
- **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*.

As an alternative, asphalt concrete can conform to Section 203-6 of the *Standard Specifications for Public Works Construction* (Green Book), 2018 Edition. Crushed aggregate base or crushed miscellaneous base can conform to Sections 200-2.2 and 200-2.4 of the *Standard Specifications for Public Works Construction* (Green Book), current edition, respectively.

3.7 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations, and foundation excavations should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 4 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

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

Temporary excavations should be treated in accordance with the State of California version of OSHA excavation regulations, Construction Safety Orders for Excavation General Requirements, Article 6, Section 1541, effective October 1, 1995. The sides of excavations should be shored or sloped in accordance with OSHA regulations. OSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a ¾H:1V (horizontal:vertical) slope for Type A soils, 1H:1V for Type B soils, and 1½H:1V for Type C soils. Near-surface onsite soils are to be considered Type B soils.

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

3.7.1 Temporary Shoring

Vertical cuts may be supported by several methods including cross-braced hydraulic shoring or conventional shields in utility trenches, or sheet piles, soldier piles and wood lagging, and/or soil nailing for basement excavations. These choices should be left to the contractor’s judgment since economic considerations and/or the individual contractor’s construction experience may determine which method is more economical and/or appropriate. However, shoring systems should be designed by a California licensed civil or structural engineer. The contractor and shoring designer should also perform additional geotechnical studies as necessary to refine means and methods of shoring construction. The contractor should forward temporary excavation support system plans to us for pre-construction review.

Hollow-stem-auger borings were drilled, which avoid empirical observation of caving soils in drilled shafts. Therefore, this report does not have any empirical information regarding the potential for caving in drilled holes (e.g. shear wall piles, soldier piles and/or tie-backs), which should be considered by the contractor. The contractor may therefore choose to evaluate the potential for difficult drilling conditions and caving of piles, soldier pile and tie-back shafts by drilling pilot holes with the intended production drilling equipment. We expect some clean sand layers at this site are prone to caving.

Support of all adjacent existing structures and infrastructure without distress is the contractor's responsibility. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structure. In addition, it should be the contractor's responsibility to undertake a pre-construction survey including (1) establishing surface survey monuments adjacent to existing sensitive structures and infrastructure to measure ground movement adjacent to excavations, and (2) photographing and otherwise documenting adjacent property conditions prior to excavation. Surface monuments should be established and read by a California licensed Professional Land Surveyor (PLS), with an accuracy on the order of 1/10th. of an inch.

As preliminary guidelines, the following geotechnical parameters can be used for shoring design:

- **Supported Earth Pressures**: For site undocumented fill, an active equivalent fluid earth pressure of 35 pounds-per-cubic-foot (pcf) should be used for <u>**temporary**</u> deflecting cantilever shoring, or 50 pcf as an at-rest pressure for <u>**temporary**</u> braced shoring, only for drained shoring above groundwater with level backfill. Braced shoring can also be designed using a uniform rectangular soil pressure of 35H psf, where H is equal to the depth of the excavation being shored, in feet. Braces, tie-backs or soil nails should be installed and pre-loaded as the excavation progresses to reduce shoring deflections. Determination of appropriate design conditions (active or at-rest) depends on shoring flexibility. If a rotation of more than 0.001 radian (0.06 degrees) at the base or at the top is allowed, active pressure conditions apply; otherwise, at-rest condition governs.

- **Surcharge Loads**: Surcharge loads (dead or live) should be added to the indicated lateral earth pressures and should be applied uniformly, if such loads are within a horizontal distance that is less-than the exposed shoring height. The corresponding lateral earth pressure will approximately be 33-percent of the vertical surcharge for active conditions, and 50-percent for at-rest conditions. Surcharge pressures from concentrated loads should be evaluated after geometric constraints and loading conditions are determined on individual basis.

- **Soldier Piles:** Soldier piles typically consist of steel H-beams set in predrilled holes and backfilled with structural concrete below the proposed excavation level and then with lean-mix concrete to the ground surface. Lagging between the soldier piles is expected to be required. Soldier piles may be assumed to have a passive resistance below the lowest adjacent excavation (bottom of pile caps) of 350 pounds-per-square-foot (psf), per foot of embedment of the soldier pile encased in concrete in firm contact with the native soil. This passive pressure should not exceed 3,500 psf, and is based on the assumption that soldier piles will be spaced at least three diameters on center. Soldier piles can be problematic in some zones of the sand material that may make drilling and lagging installation difficult. Due to the potential for sand caving, drilled shafts should be poured the same day as drilled, and under no circumstances should be left open overnight. If water is encountered then it should be pumped out, and the "tremmie" method used in pouring concrete, which should be designed for an additional compressive strength of 1,000 psi above the dry shaft design strength. Casing should be made available during drilling.

3.8 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1 and 306-6 of the Standard Specifications for Public Works Construction, ("Greenbook"), current edition. Utility trenches can be backfilled with onsite sandy 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) **Sand:** A uniform, sand material that has a Sand Equivalent (SE) greater-than-or-equal-to (≥) 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer), water densified in place, or

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the *Standard Specifications for Public Works Construction,* ("Greenbook"), current Edition. CLSM should not be jetted.

Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. Native and clean fill soils can be used as backfill over the pipe bedding zone, and should be placed in thin lifts, moisture conditioned above optimum, and mechanically compacted to at least 95 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

Utility trenches excavated into site clays should not have sand bedding placed within 3 feet of the trench as it exits from under the slabs to prevent water migration along bedding sand into and under the foundation. A bentonite plug or clayey soil can be placed to interrupt bedding sand and prevent water migration.

3.9 Drainage and Landscaping

Building walls below grade should be waterproofed or at least damp proofed, depending upon the degree of moisture protection desired. Surface drainage should be designed to direct water away from foundations and toward approved drainage devices. Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without overwatering.

3.10 Additional Geotechnical Services

Leighton should review the grading plans, foundation plans, and specifications when they are available to verify that the recommendations presented in this report have been properly interpreted and incorporated.

Geotechnical observation and testing should be provided during the following activities:

- Grading and excavation of the site;
- During installation of shoring,
- Subgrade Preparation;
- Compaction of all fill materials;
- Utility trench backfilling and compaction;
- Footing excavation and slab-on-grade preparation;
- Pavement subgrade and base preparation;
- Placement of asphalt concrete and/or concrete; and
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This geotechnical exploration does not address the potential for encountering hazardous soil at this site. In addition, 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, by necessity, incomplete. Please also refer GBA's *Important Information About Your Geotechnical Report* (included at the rear of the text), presenting additional information and limitations regarding geotechnical engineering studies and reports. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if Leighton and Associates, Inc. has the opportunity to observe subsurface conditions during grading and construction, to confirm that our data are representative for the site. Leighton and Associates, Inc. should also review the construction plans and project specifications, when available, to comment on the geotechnical aspects.

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing at this time in Orange County. We do not make any warranty, either expressed or implied.

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Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

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

Do not rely on this report if your geotechnical engineer prepared it:
- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are 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 provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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

FIGURES

Project: 13542.001	Eng/Geol: RAR/JAR
Scale:1 " = 2,000 '	Date: June 2022

Reference: © 2022 Microsoft Corporation © 2022 Maxar ©CNES (2022) Distribution Airbus

SITE LOCATION MAP

Proposed Residential Development
1401 Quail Street
Newport Beach, California

FIGURE 1

Leighton

Map Saved as J:\Drafting\13542\001\Maps\13542-001_F01_SLM_2022-06-03.mxd on 5/26/2022 5:01:54 PM Author: KVM (btran)

APN : N/A
EXISTING ZONE Light Industrial
PC11 Newport Place Planned Community

PROJECT SUMMARY

GROSS AREA	+/-1.7 ac
TOTAL UNITS	30 units
DENSITY	17.64 du/ac

PARKING SUMMARY

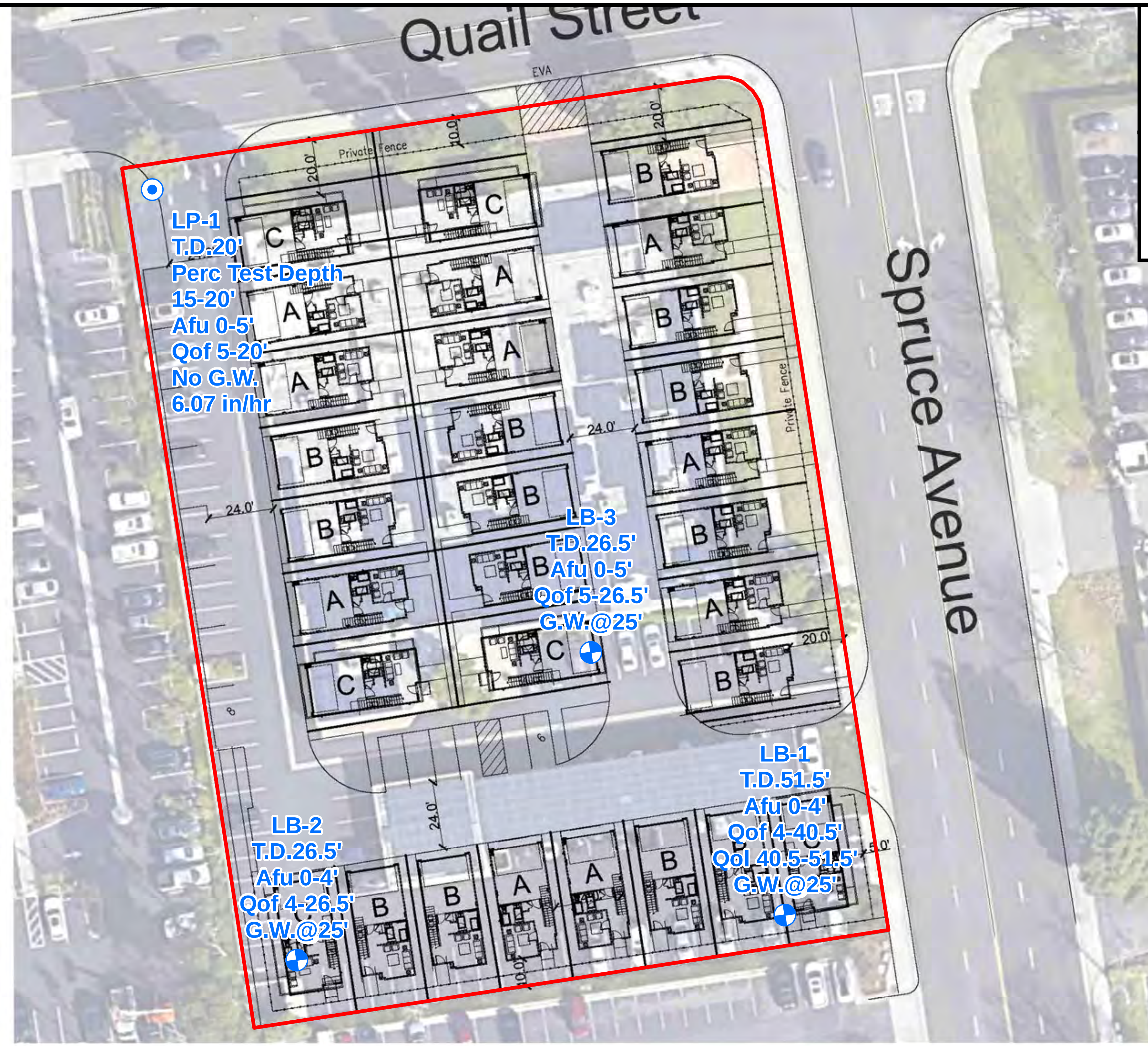
PARKING REQUIRED:	n/a
PARKING PROVIDED:	74 spaces (+/-2.46 ratio)
- SURFACE PARKING	14 spaces
- GARAGES	60 spaces

PRODUCT MIX:

PLAN A	10 units	(33.3%)
PLAN B	14 units	(46.7%)
PLAN C	6 units	(20.0%)
TOTAL	30 units	(100%)

PROPOSED ZONE CHANGE:

PROPOSED ZONE: RM (Multi-Unit Residential)
SETBACKS:
- Front: 20'
- Side: 3'
- Rear: 10'



LEGEND

LB-3 Approximate location of hollow-stem auger boring showing proposed depth (T.D.) in feet below adjacent grade.

LP-1 Approximate location of proposed infiltration test boring shown with test depth between 15-20 feet

Approximate Site Boundary

N

0 50 100
Feet

North
0 20' 40' 80'
Scale: 1" = 20' (on 24x36 sheet)

INTRACORP
Newport Beach, Ca

CONCEPT SITE PLAN - 30 UNITS
(THE PLACE FOOTPRINTS)

QUAIL STREET SFD
Newport Beach, CA.

BSB DESIGN
BSBDESIGN.COM

The drawings presented are illustrative of character and design intent only, and are subject to change based upon final design considerations (i.e. applicable codes, structural, and MEP design requirements, unit plan / floor plan changes, etc.)

April 25, 2022 | SF220XXX.00

Project: 13542.001	Eng/Geol: RAR/JAR
Scale:1 " = 50 '	Date: June 2022
Reference: Base Map As Shown	

GEOTECHNICAL MAP

Proposed Residential Development
1401 Quail Street
Newport Beach, California

FIGURE 2a

Leighton

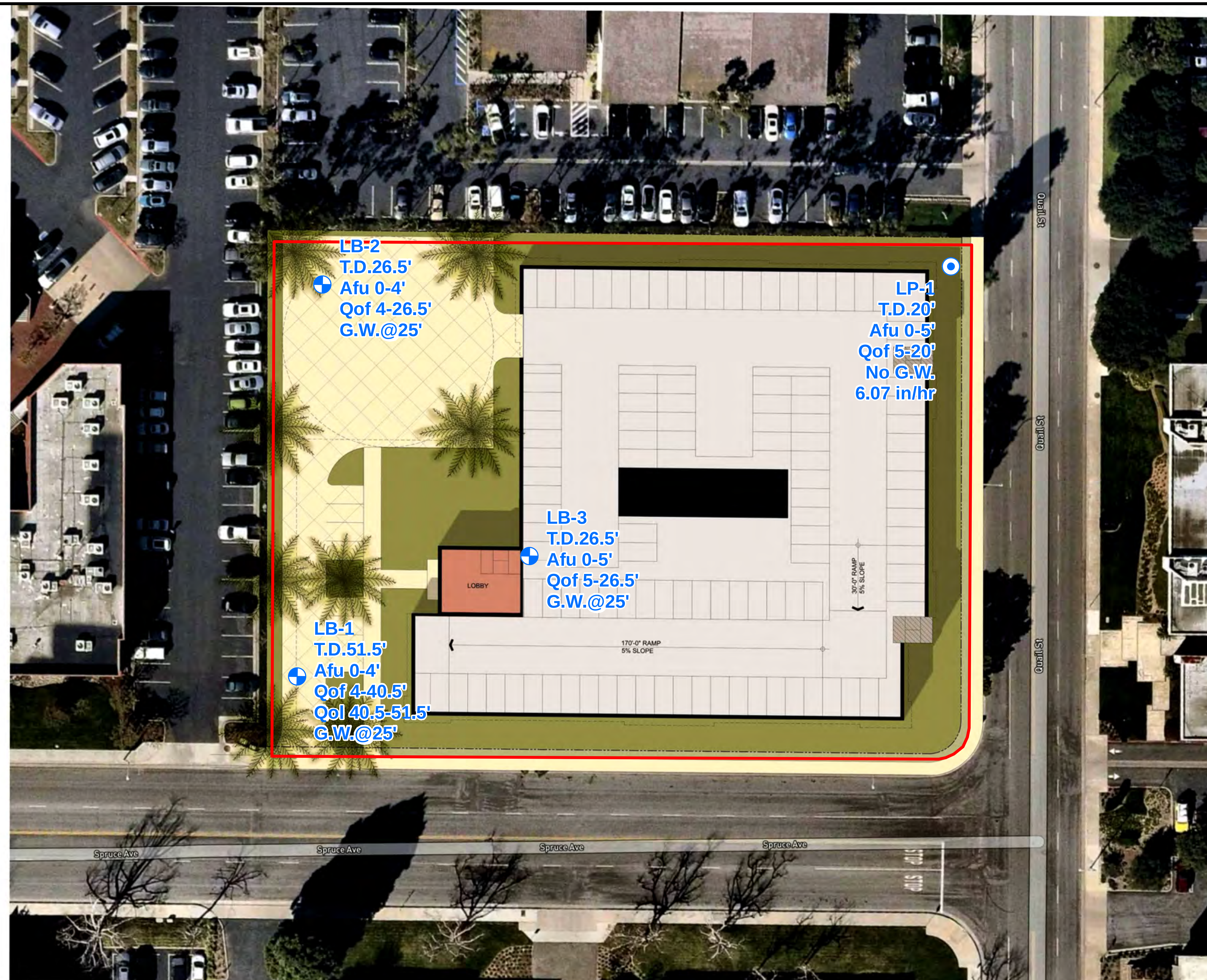
Map Saved as J:\Drafting\13542\001\Maps\13542_001_F02a_GM_2022-06-10.mxd on 6/10/2022 10:35:31 AM Author: KVM (btran)

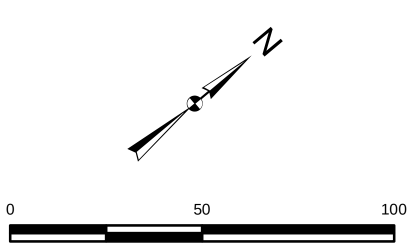
LEGEND

LB-3 — Approximate location of hollow-stem auger boring showing proposed depth (T.D.) in feet below adjacent grade.

LP-1 — Approximate location of proposed infiltration test boring shown with test depth between 15-20 feet

Approximate Site Boundary





Project: 13542.001	Eng/Geol: RAR/JAR
Scale: 1 " = 50 '	Date: June 2022

Reference: Quail Podium, Ground Floor
Intracorp, Dated: 05/02/2022

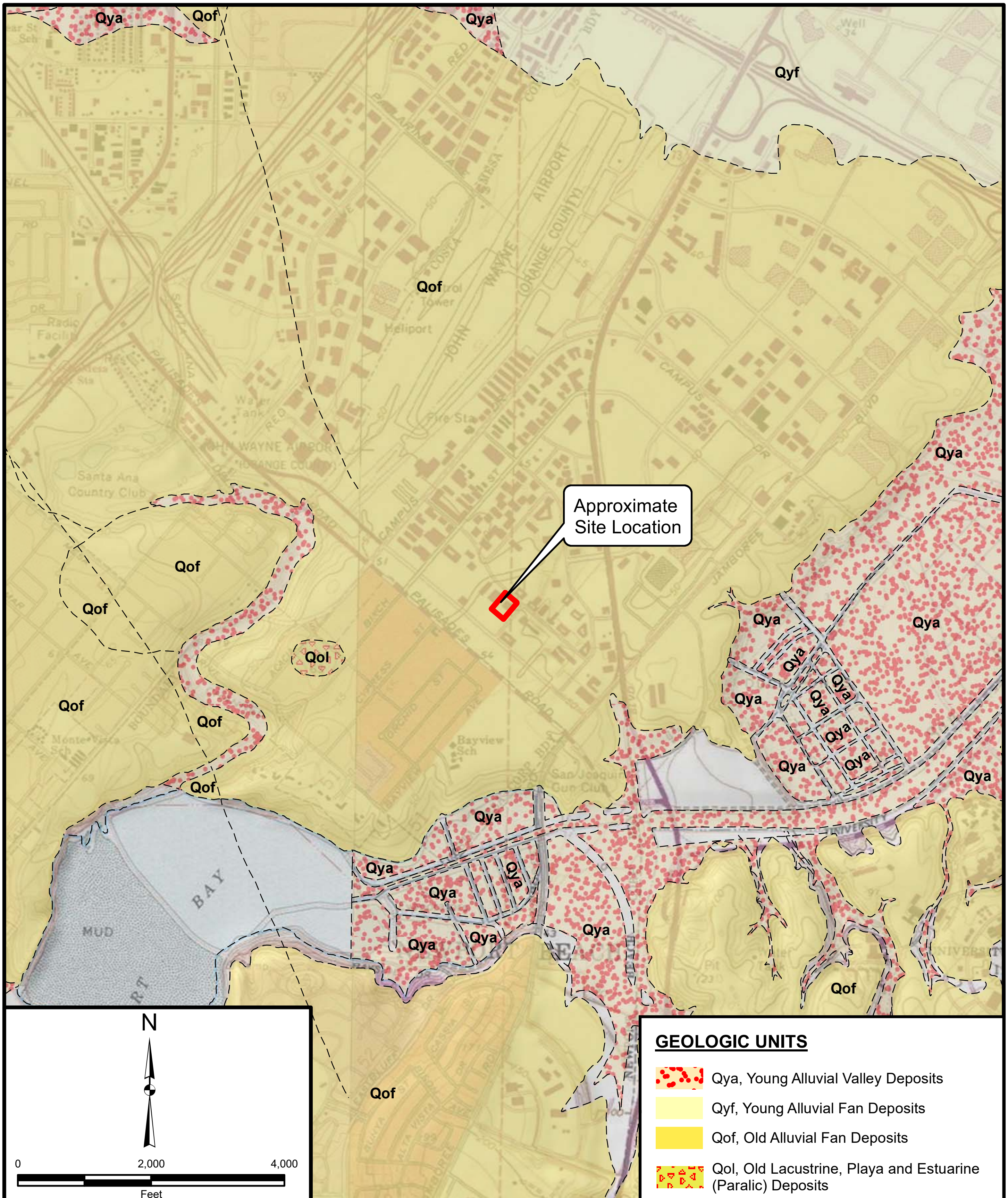
GEOTECHNICAL MAP

Proposed Residential Development
1401 Quail Street
Newport Beach, California

FIGURE 2b

Leighton

Map Saved as J:\Drafting\13542\001\Maps\13542_001_F02b_GM_2022-06-10.mxd on 6/10/2022 10:37:18 AM Author: KVM (btran)



Project: 13542.001	Eng/Geol: RAR/JAR
Scale:1 " = 2,000 '	Date: June 2022

Reference: Geologic Compilation of Quaternary Surficial Deposits in Southern California, Orange County, Bedrossian et. al., 2010

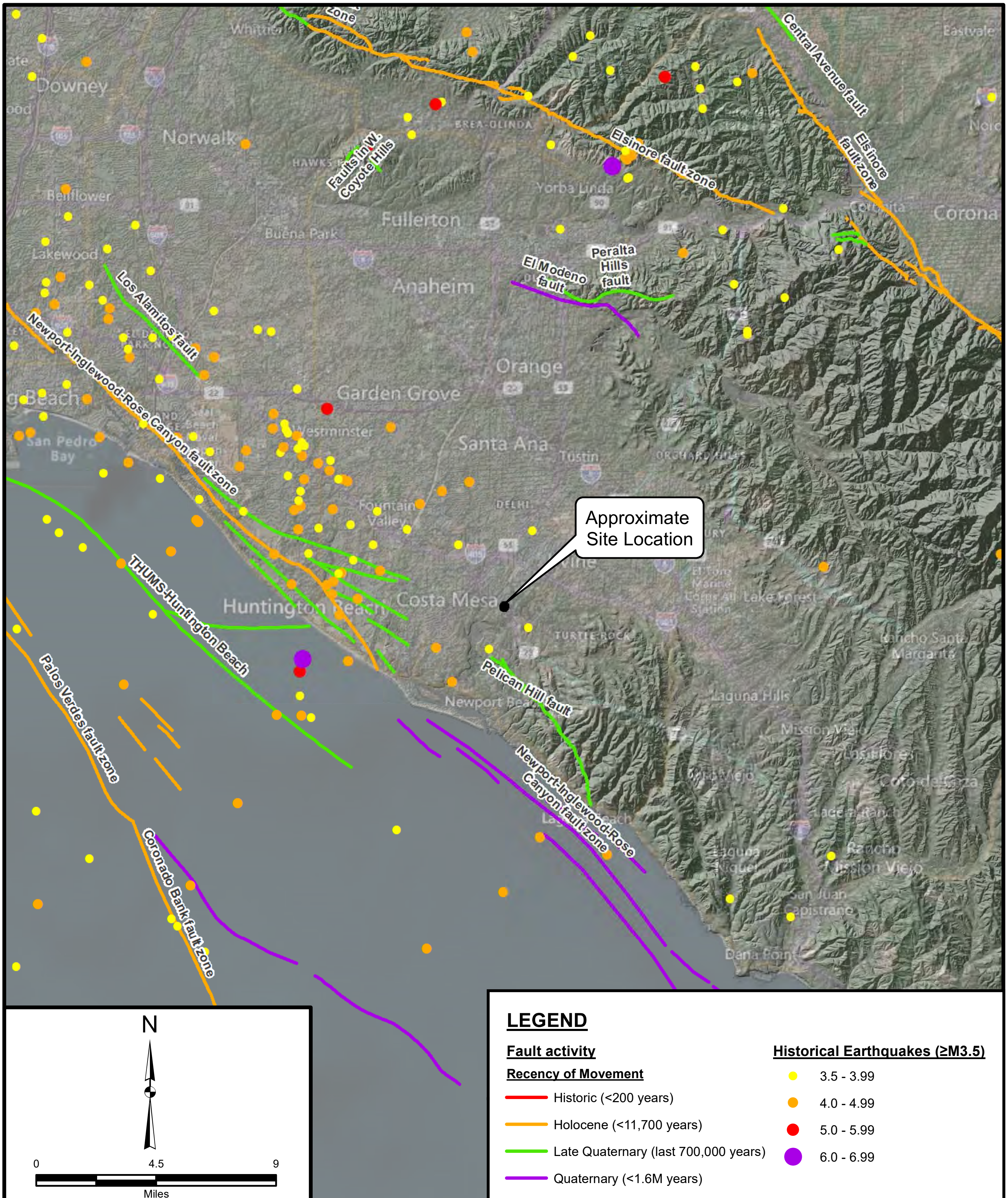
REGIONAL GEOLOGY MAP

Proposed Residential Development
1401 Quail Street
Newport Beach, California

FIGURE 3

Leighton

Map Saved as J:\Drafting\13542\001\Maps\13542-001_F03_RGM_2022-06-03.mxd on 5/26/2022 5:13:19 PM Author: KVM (btran)



Project: 13542.001	Eng/Geol: RAR/JAR
Scale: 1 " = 5 miles	Date: June 2022

Basemap Reference: © 2022 Microsoft Corporation
Earthstar Geographics SIO © 2022 TomTom
Seismicity Data Reference: maps.conservation.ca.gov

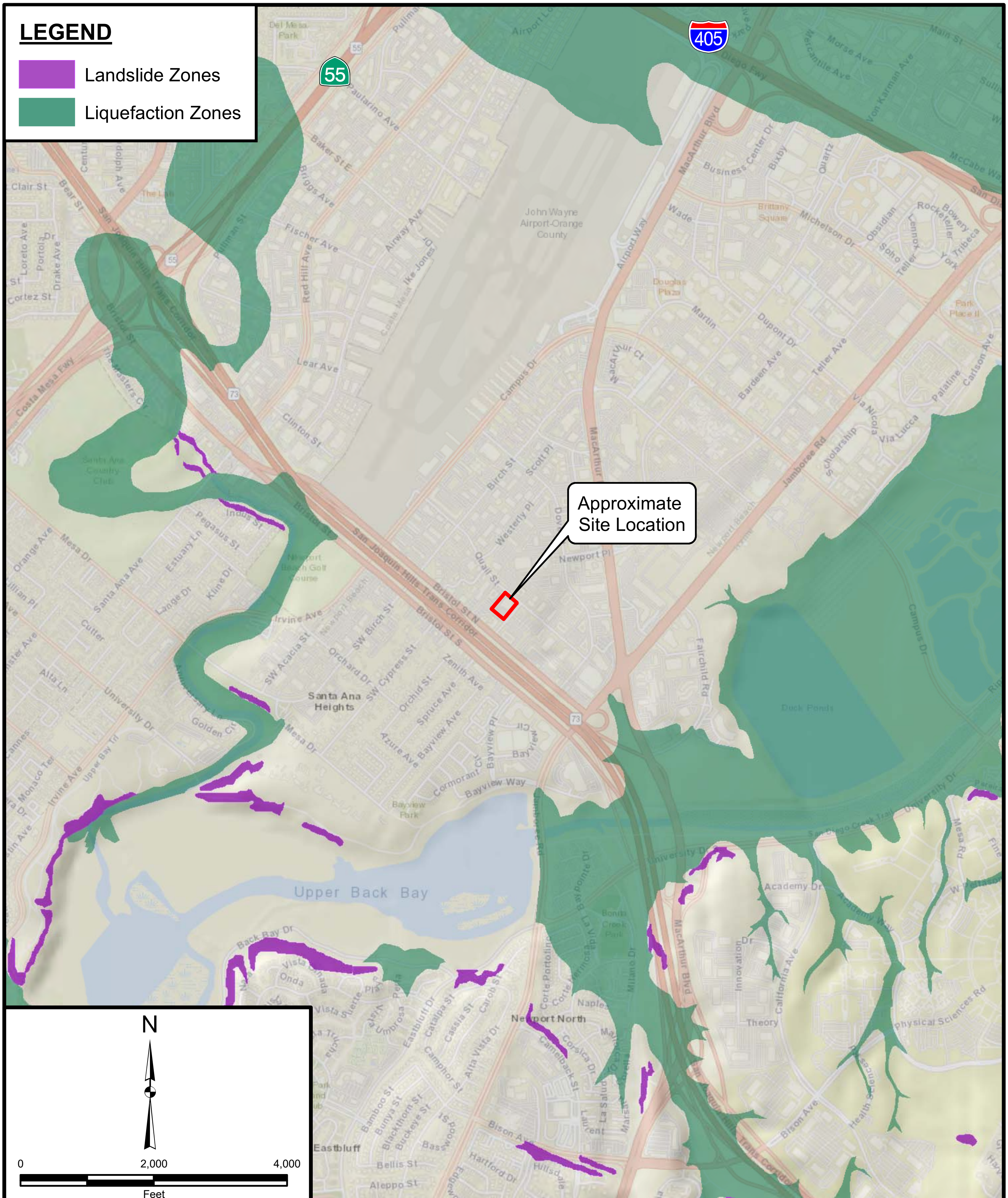
REGIONAL FAULTS AND HISTORIC SEISMICITY MAP

Proposed Residential Development
1401 Quail Street
Newport Beach, California

FIGURE 4

Leighton

Map Saved as J:\Drafting\13542\001\Maps\13542-001_F04_RFHSM_2022-06-03.mxd on 5/26/2022 5:15:06 PM Author: KVM (btran)



Project: 13542.001	Eng/Geol: RAR/JAR
Scale:1 " = 2,000 '	Date: June 2022

Service Layer Credits: 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
Seismic Hazards Program, California Geological Survey, California

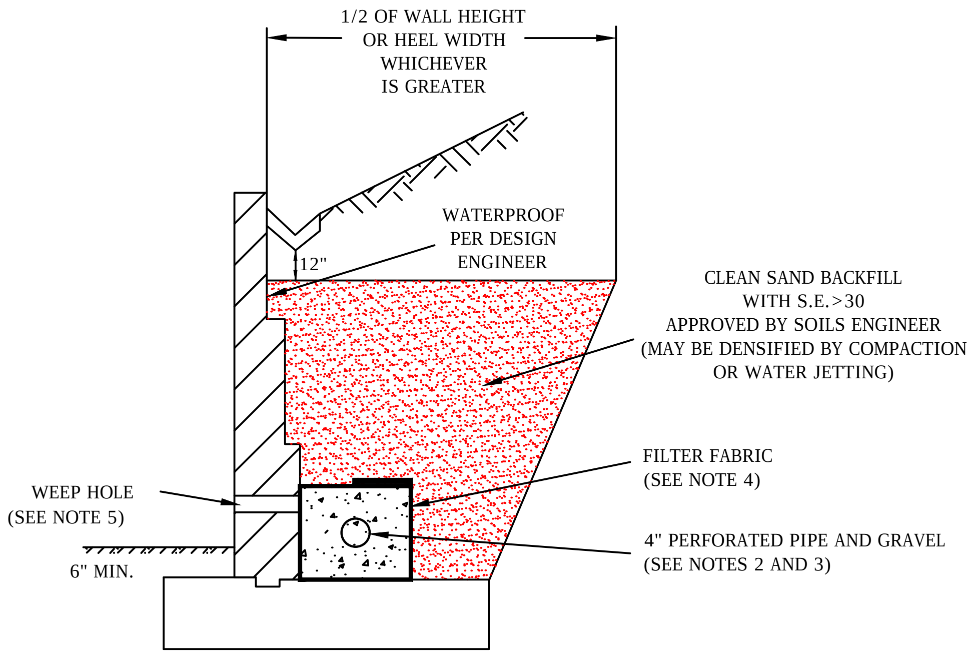
SEISMIC HAZARD MAP

Proposed Residential Development
1401 Quail Street
Newport Beach, California

FIGURE 5

Leighton

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



NOTE: AS AN ALTERNATE TO CLEAN SAND BACKFILL, CLEAN GRAVEL MAY BE UTILIZED WITH APPROVED FILTER FABRIC. A SECOND ALTERNATE IS TO UTILIZE AN AGGREGATE BASE MATERIAL COMPACTED TO 90% RELATIVE COMPACTION. A SAMPLE OF THE PROPOSED BASE MUST BE APPROVED BY THE GEOTECHNICAL CONSULTANT PRIOR TO BACKFILL FOR SUITABILITY. COMPACTION SHOULD BE ACHIEVED WITHOUT DAMAGING THE WALL.

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

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50

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FIGURE 6

APPENDIX A

EXPLORATION LOGS

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GEOTECHNICAL BORING LOG LB-1

Project No.	13542.001	**Date Drilled**	5-20-22
Project	Intracorp Quail St	**Logged By**	LFO
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	44'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	LFO

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *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.*	Type of Tests
	0			BB-1					@Surface: 3" asphalt over 8" CLAY w/ Sand base	MD, DS, EI, CR
									Undocumented Artificial Fill (Afu)	
				R-1	12 9 9			SM	@2': Silty SAND, yellowish brown, medium dense, predominantly fine sand, trace medium and coarse sand, some rootlets, slightly moist	
40									**Quaternary Old Alluvial Fan Deposits (Qof)**	
				R-2	4 9 11	103.7	20.1	CL	@4': Sandy CLAY, dark yellow brown, very stiff, fine sand, low plasticity, weakly laminated, moist	
	5			R-3	5 11 12	107.0	8.3	CL	@6': Sandy CLAY, reddish brown, very stiff, fine sand, low to medium plasticity, laminated, moist	
35				R-4	6 8 13	113.4	17.0	CL	@8': CLAY, reddish brown mottled gray, very stiff, some fine sand, low to medium plasticity, trace MnO, moist	
	10			R-5	8 13 21	110.4	18.8	CL	@10': CLAY, reddish brown mottled gray, very stiff, some fine sand, low to medium plasticity, some MnO and carbonate blebs, moist	
30										
	15			S-6	3 5 10		24.0	CH	@15': Fat CLAY, light yellowish brown, very stiff, trace fine sand, low plasticity, large carbonate blebs, moist	
25										
	20			R-7	14 20 28	98.6	8.9	ML	@20': Sandy SILT, gray brown mottled yellow and orange (heavily Fe-stained), dense, predominantly fine sand, trace medium sand, friable, very moist	
20										
	25			S-8	4 8 16		19.1	SP	@25': Poorly-graded SAND, gray brown, dense, predominantly medium sand, trace fine and coarse sand, Fe-stained, micaceous, friable, wet	
15										
	30									

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

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

Leighton

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

GEOTECHNICAL BORING LOG LB-1

Project No.	13542.001	**Date Drilled**	5-20-22
Project	Intracorp Quail St	**Logged By**	LFO
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	44'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	LFO

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	30			R-9	25 50/2"			SP	@30': Poorly-graded SAND, gray brown, very dense, predominantly medium sand, trace fine and coarse sand, Fe-stained, micaceous, friable, wet	
10										
	35			S-10	9 24 30		18.4	SP	@35': Poorly-graded SAND, gray brown, very dense, predominantly medium sand, trace fine and coarse sand, Fe-stained, micaceous, friable, abundant white shell fragments, wet	
5										
	40			S-11	1 2 2		32.7	SP CH	@40': Same as above **Quaternary Old Lacustrine, Playa, and Estuarine Deposits (Qol)** @40.5': Fat CLAY, bluish gray, medium stiff, high plasticity, moist	
0										
	45			S-12	Push Push 2		39.6	CH	@45': Fat CLAY, bluish gray, soft, high plasticity, trace white shell fragments, wet	
-5										
	50			S-13	Push Push 3		41.0	CH	@50': Same as above	
									T.D. 51.5 feet bgs **Groundwater encountered at 25 feet bgs.** **Borehole backfilled with soil cuttings and patched with cold-patch asphalt.**	
-10										
	55									
-15										
	60									

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

TYPE OF TESTS:

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

Leighton

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

GEOTECHNICAL BORING LOG LB-2

Project No.	13542.001	**Date Drilled**	5-20-22
Project	Intracorp Quail St	**Logged By**	LFO
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	43'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	LFO

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BB-1					@Surface: 8" asphalt over 4" CLAY w/ Sand base	
									Undocumented Artificial Fill (Afu)	
40				R-1	6 8 10			SM	@2': Silty SAND, reddish brown, medium dense, fine sand, trace clay, slightly moist	
									Quaternary Old Alluvial Fan Deposits (Qof)	
	5			R-2	10 12 14	109.9	17.5	CL	@4': Sandy CLAY, reddish brown mottled gray, very stiff, fine sand, trace MnO, moist	
				R-3	7 14 23	113.9	6.7	SM	@6': Silty SAND, orange brown, dense, predominantly fine sand, trace medium sand, friable, moist	
35				R-4	8 16 23	117.0	15.7	CL	@8': Lean CLAY, reddish brown, hard, low plasticity, some MnO stringers, moist	
	10			R-5	7 13 21	114.1	17.3	CL	@10': Lean CLAY, reddish brown, very stiff, low plasticity, some MnO blebs, moist	
30										
	15			S-6	3 6 12		21.1	CH	@15': Fat CLAY, light yellowish brown, very stiff, trace fine sand, low plasticity, abundant carbonate blebs, moist	
25										
	20			R-7	13 20 25	106.6	9.6	ML	@20': Sandy SILT, gray brown mottled orange from Fe-staining, dense, fine sand, weakly laminated, friable, moist	
20										
	25			S-8	5 13 20		23.5	SP	@25': Poorly-graded SAND, graybrown mottled orange from Fe-staining, dense, predominantly fine to medium sand, trace coarse sand, very moist	
									T.D. 26.5 feet bgs **Groundwater encountered at 25 feet bgs.** **Borehole backfilled with soil cuttings and patched with cold-patch asphalt.**	
15										
	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	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

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GEOTECHNICAL BORING LOG LB-3

Project No.	13542.001	**Date Drilled**	5-20-22
Project	Intracorp Quail St	**Logged By**	LFO
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	42'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	LFO

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *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.*	Type of Tests
	0			BB-1					@Surface: 4" asphalt over 4" CLAY base	
									Undocumented Artificial Fill (Afu)	
40										
				R-1	5 13 22	110.8	158	SM	@3': Silty SAND, yellow brown, dense, predominantly fine sand, trace medium sand, trace clay, weakly laminated, moist	
	5			R-2	9 14 21	106.4	19.0	SC	**Quaternary Old Alluvial Fan Deposits (Qof)** @5': Clayey SAND, gray yellow brown, medium dense, fine sand, low plasticity, moist	
35				R-3	7 13 19	115.4	16.1	CL	@7': Sandy CLAY, orange brown, very stiff, fine sand, low plasticity, trace MnO, some carbonate blebs, slightly moist	
	10			R-4	6 14 20	116.5	15.0	CL	@10': Sandy CLAY, reddish brown mottled gray brown, very stiff, fine sand, low plasticity, slightly moist	
30										
	15			R-5	6 10 15	104.3	23.2	CH	@15': Fat CLAY, light yellowish brown, very stiff, fine sand, low plasticity, abundant carbonate blebs, slightly moist	AL, CN
25								SM	@17': Changes to yellow Silty SAND in tailings	
	20			S-6	4 8 9		14.4	SM	@20': Silty SAND, gray mottled yellow and orange from Fe-staining, medium dense, fine sand, trace clay, friable, very moist	
20										
	25 ▽			S-7	7 17 21		23.4	SP	@25': Poorly-graded SAND, gray brown, very dense, fine sand, friable, micaceous, very moist	
15									**T.D. 26.5 feet bgs** **Groundwater encountered at 25 feet bgs.** **Borehole backfilled with soil cuttings and patched with cold-patch asphalt.**	
	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	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

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GEOTECHNICAL BORING LOG LP-1

Project No.	13542.001	**Date Drilled**	5-20-22
Project	Intracorp Quail St	**Logged By**	LFO
Drilling Co.	Martini Drilling Corp	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	41'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	LFO

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
								This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0		BB-1					@Surface: 4" asphalt over 5" CLAY base	
40								**Undocumented Artificial Fill (Afu)** Silty SAND w/ Clay, yellow brown, fine sand, low plasticity, friable, slightly moist	
	5		R-1	6 14 22	105.8	14.1	SP-SM	**Quaternary Old Alluvial Fan Deposits (Qof)** @5': Poorly-graded SAND w/ Silt and Clay, yellow brown, dense, fine sand, low plasticity, friable, slightly moist	
35			R-2	7 11 15	97.2	2.2	SP-SM	@7': Poorly-graded SAND w/ Silt, olive yellow, medium dense, predominantly fine sand, trace medium sand, trace clay, slightly moist	SA
	10		R-3	8 13 19	110.9	19.8	SP	@10': Poorly-graded SAND, orange brown, predominantly fine to medium sand, medium dense, micaceous, friable, slightly moist	
30								@11': Lean CLAY, orange brown mottled gray brown, low plasticity, trace MnO staining, moist	
	15		R-4 BB-2	7 12 18	102.1	24.8	CH	@15': Fat CLAY, light yellowish brown, very stiff, low plasticity, abundant carbonate blebs, moist	
25									
	20		S-5	4 9 10		10.5	SP	@20': Poorly-graded SAND, mustard yellow, medium dense, predominantly fine sand, friable, micaceous, moist	
20								**T.D. 21.5 feet bgs** **No groundwater encountered during drilling.** **Borehole backfilled with soil cuttings and patched with cold-patch asphalt.**	
	25								
15									
	30								

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

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

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APPENDIX B

PERCOLATION TEST DATA

Boring Percolation Test Data Sheet

Project Number:	13542.001	**Test Hole Number:**	LP-1	
Project Name:	IntraCorp Quail Street	**Date Excavated:**	5/20/2022	
Earth Description:	Alluvium	**Date Tested:**	5/20/2022	
Liquid Description:	Tap water	**Depth of boring (ft):**	20	
Tested By:	BTM/LFO	**Radius of boring (in):**	4	
Time Interval Standard		**Radius of casing (in):**	1	
Start Time for Pre-Soak:	8:21 AM	**Length of slotted of casing (ft):**		5
Start Time for Standard:	8:59 AM	**Depth to Initial Water Depth (ft):**		
Standard Time Interval	13	**Porosity of Annulus Material, *n* :**		0.35
Between Readings, mins:	5	**Bentonite Plug at Bottom:**		No

Field Percolation Data - Falling Head Test

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
P1	8:21	13	15.00	60.0	54.0	5.56
	8:34		19.50	6.0		
P2	8:39	15	15.00	60.0	54.0	4.99
	8:55		19.50	6.0		
1	8:59	5	15.00	60.0	27.8	5.43
	9:04		17.32	32.2		
2	9:06	5	15.00	60.0	28.3	5.55
	9:11		17.36	31.7		
3	9:12	5	15.00	60.0	28.9	5.70
	9:17		17.41	31.1		
4	9:19	5	15.00	60.0	28.7	5.64
	9:24		17.39	31.3		
5	9:25	5	15.00	60.0	29.0	5.73
	9:30		17.42	31.0		
6	9:32	5	15.00	60.0	28.6	5.61
	9:37		17.38	31.4		
7	9:38	5	15.00	60.0	28.6	5.61
	9:43		17.38	31.4		
8	9:45	5	15.00	60.0	29.0	5.73
	9:50		17.42	31.0		
9	9:54	5	15.00	60.0	28.8	5.67
	9:59		17.40	31.2		
10	10:02	5	15.00	60.0	25.4	4.84
	10:07		17.12	34.6		
11	10:09	5	15.00	60.0	29.8	5.92
	10:14		17.48	30.2		
12	10:16	5	15.00	60.0	29.3	5.80
	10:21		17.44	30.7		
13	10:24	5	15.00	60.0	29.4	5.83
	10:29		17.45	30.6		
14	10:30	5	15.00	60.0	30.1	6.02
	10:35		17.51	29.9		
15	10:38	5	15.00	60.0	29.9	5.95
	10:43		17.49	30.1		
16	10:44	5	15.00	60.0	29.4	5.83
	10:49		17.45	30.6		
17	10:52	5	15.00	60.0	30.0	5.98
	10:57		17.50	30.0		
18	10:58	5	15.00	60.0	28.9	5.70
	11:03		17.41	31.1		
19	10:07	5	15.00	60.0	28.9	5.70
	11:12		17.41	31.1		
20	11:13	5	15.00	60.0	28.9	5.70
	11:18		17.41	31.1		
17	11:19	5	15.00	60.0	30.5	6.11
	11:24		17.54	29.5		
18	11:27	5	15.00	60.0	30.4	6.08
	11:32		17.53	29.6		
19	11:35	5	15.00	60.0	30.8	6.21
	11:40		17.57	29.2		
20	11:41	5	15.00	60.0	30.8	6.21
	11:46		17.57	29.2		
21	11:48	5	15.00	60.0	30.0	5.98
	11:53		17.50	30.0		
22	11:54	5	15.00	60.0	30.1	6.02
	11:59		17.51	29.9		

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

Measured Infiltration Rate, I (Average of Last 3 Readings) = 6.07 in./hr.

APPENDIX C

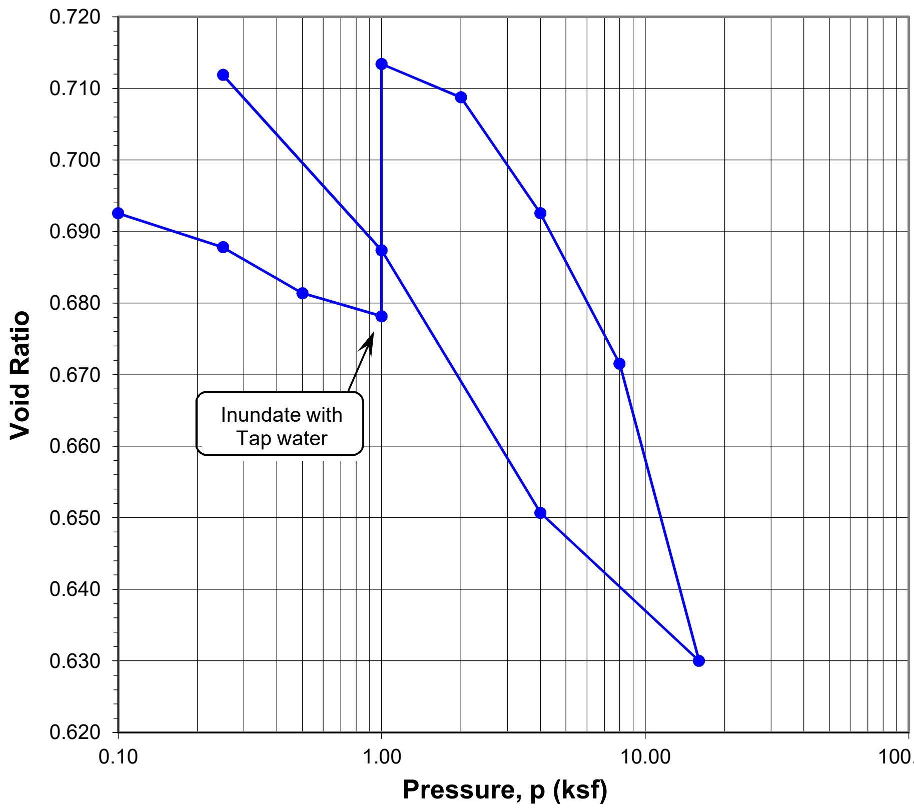
LABORATORY TEST RESULTS

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	IntraCorp Quail Street	Tested By: GB/JD	Date: 06/01/22
Project No.:	13542.001	Checked By: J. Ward	Date: 06/13/22
Boring No.:	LB-3	Depth (ft.): 15.0	
Sample No.:	R-5	Sample Type: Ring	
Soil Identification:	Light yellowish brown fat clay (CH)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	195.35
Weight of Ring (g):	45.29
Height after consol. (in.):	1.0103
Before Test	
Wt. of Wet Sample+Cont. (g):	278.38
Wt. of Dry Sample+Cont. (g):	238.40
Weight of Container (g):	65.85
Initial Moisture Content (%)	23.2
Initial Dry Density (pcf)	101.3
Initial Saturation (%):	92
Initial Vertical Reading (in.)	0.0776
After Test	
Wt. of Wet Sample+Cont. (g):	236.97
Wt. of Dry Sample+Cont. (g):	204.86
Weight of Container (g):	39.01
Final Moisture Content (%)	26.63
Final Dry Density (pcf):	99.2
Final Saturation (%):	100
Final Vertical Reading (in.)	0.0720
Specific Gravity (assumed):	2.75
Water Density (pcf):	62.43

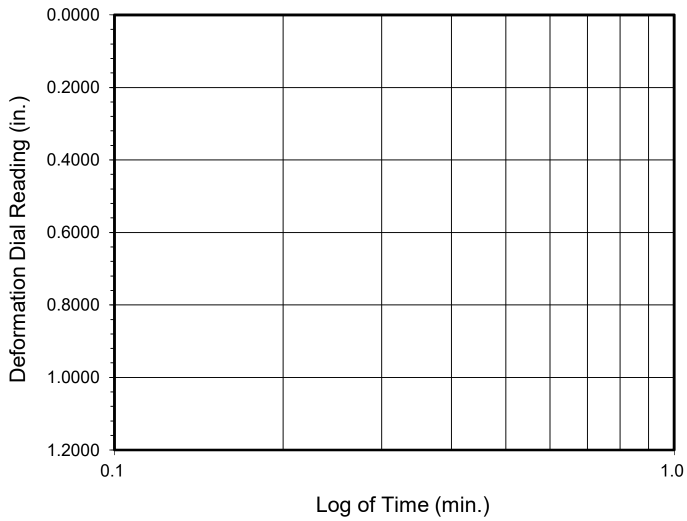


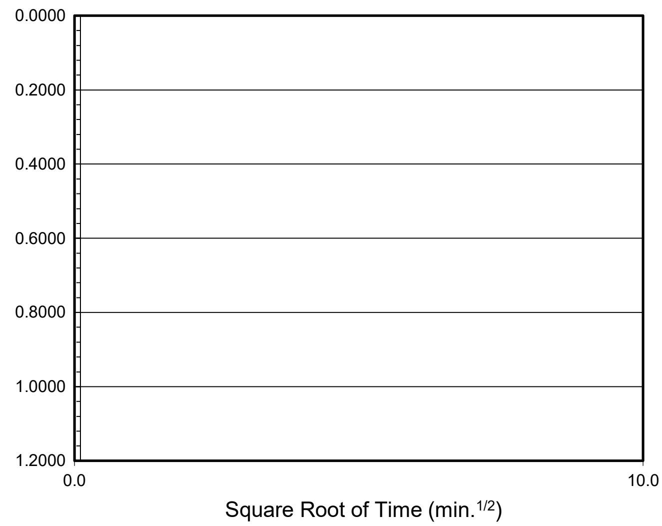
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.0787	0.9989	0.00	0.11	0.693	0.11
0.25	0.0826	0.9950	0.11	0.50	0.688	0.39
0.50	0.0878	0.9898	0.25	1.02	0.681	0.77
1.00	0.0915	0.9861	0.43	1.39	0.678	0.96
1.00	0.0707	1.0069	0.43	-0.69	0.713	-1.12
2.00	0.0750	1.0027	0.58	-0.27	0.709	-0.85
4.00	0.0859	0.9917	0.72	0.83	0.693	0.11
8.00	0.0999	0.9777	0.88	2.23	0.672	1.35
16.00	0.1263	0.9513	1.07	4.87	0.630	3.80
4.00	0.1123	0.9653	0.89	3.47	0.651	2.58
1.00	0.0890	0.9887	0.72	1.14	0.687	0.41
0.25	0.0720	1.0056	0.47	-0.56	0.712	-1.03

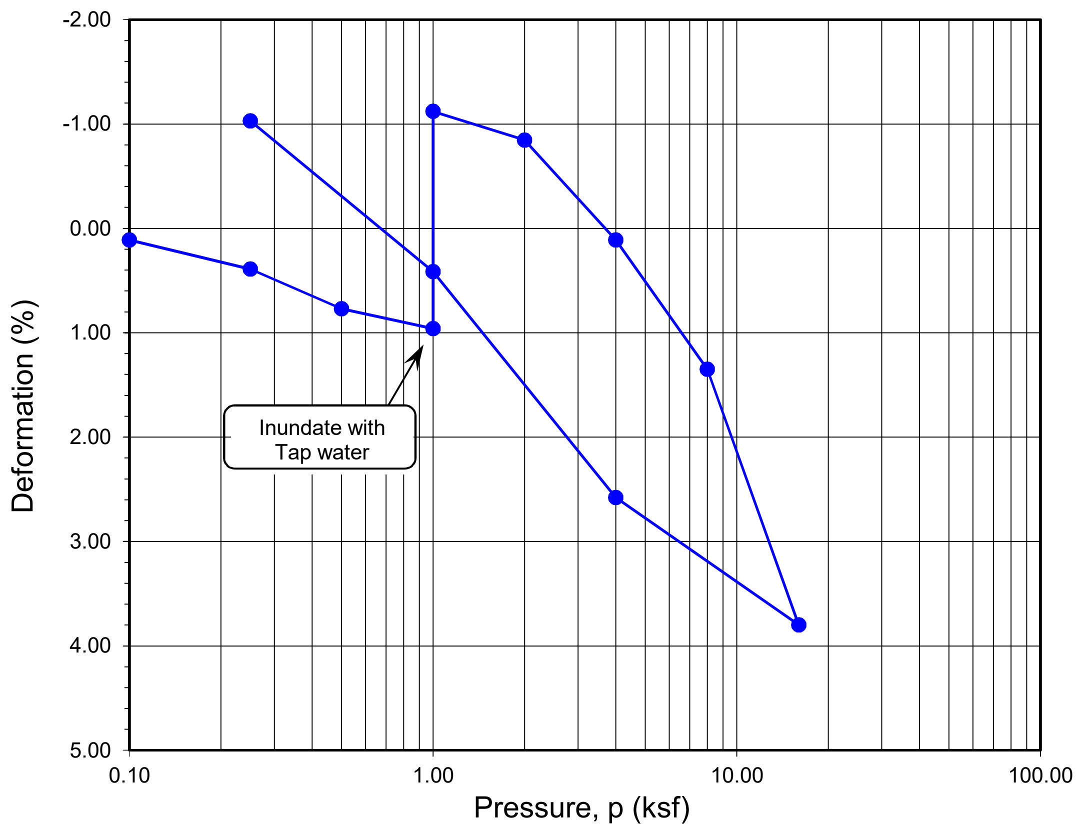
Time Readings

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

Time Readings







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-3	**R-5**	**15**	**23.2**	**26.6**	**101.3**	**99.2**	**0.694**	**0.712**	**92**	**100**

Soil Identification: Light yellowish brown fat clay (CH)

Leighton	**ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS** **ASTM D 2435**	Project No.: 13542.001 IntraCorp Quail Street 06-22

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TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name:	IntraCorp Quail Street	Tested By :	GEB/JD	Date:	06/07/22
Project No. :	13542.001	Checked By:	J. Ward	Date:	06/13/22

Boring No.	LB-1			
Sample No.	BB-1			
Sample Depth (ft)	0-5			
Soil Identification:	Yellowish brown SM			
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)	100.12			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	5			
Crucible No.	4			
Furnace Temperature (°C)	860			
Time In / Time Out	13:00/13:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	21.6441			
Wt. of Crucible (g)	21.6352			
Wt. of Residue (g) (A)	0.0089			
PPM of Sulfate (A) x 41150	366.24			
PPM of Sulfate, Dry Weight Basis	**366**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.6			
PPM of Chloride (C -0.2) * 100 * 30 / B	80			
PPM of Chloride, Dry Wt. Basis	**80**			

pH TEST, DOT California Test 643

pH Value	**7.65**			
Temperature °C	21.5			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	IntraCorp Quail Street	Tested By :	J. Domingo Date: 06/11/22
Project No. :	13542.001	Checked By:	J. Ward Date: 06/13/22
Boring No.:	LB-1	Depth (ft.) :	0-5
Sample No. :	BB-1		

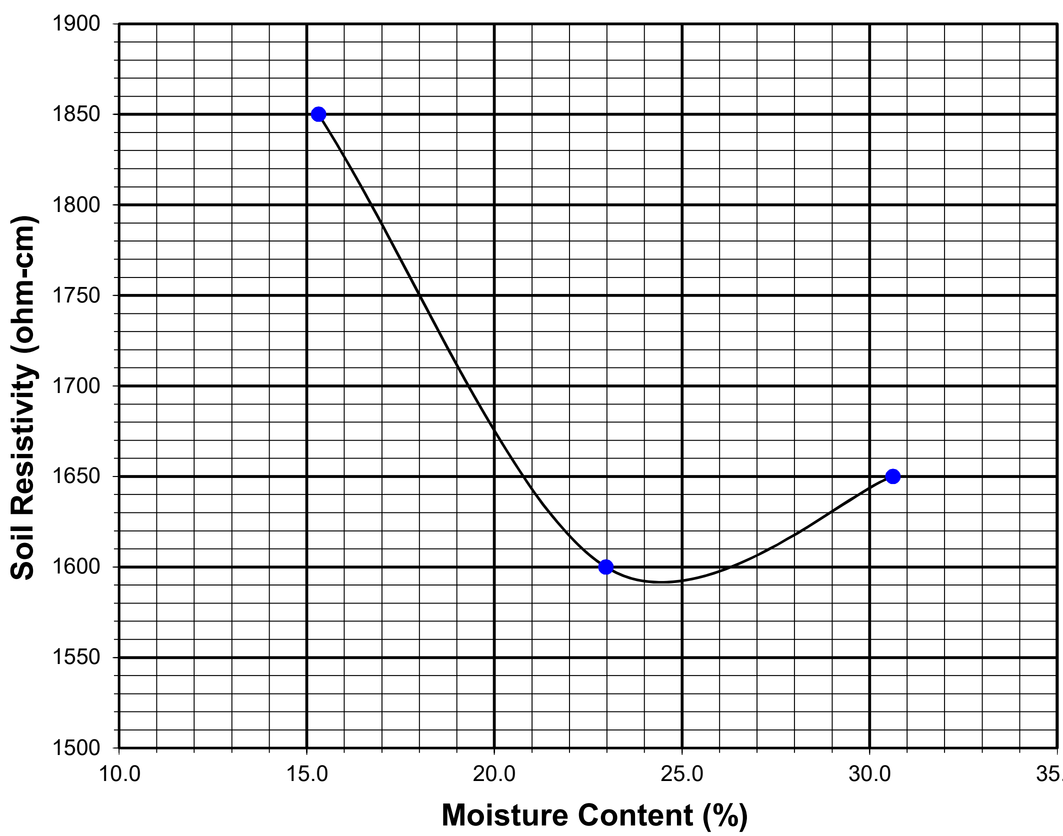
Soil Identification:* Yellowish brown SM

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	15.31	1850	1850
2	30	22.97	1600	1600
3	40	30.63	1650	1650
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)	130.60
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1590	**24.5**	**366**	**80**	**7.65**	**21.5**



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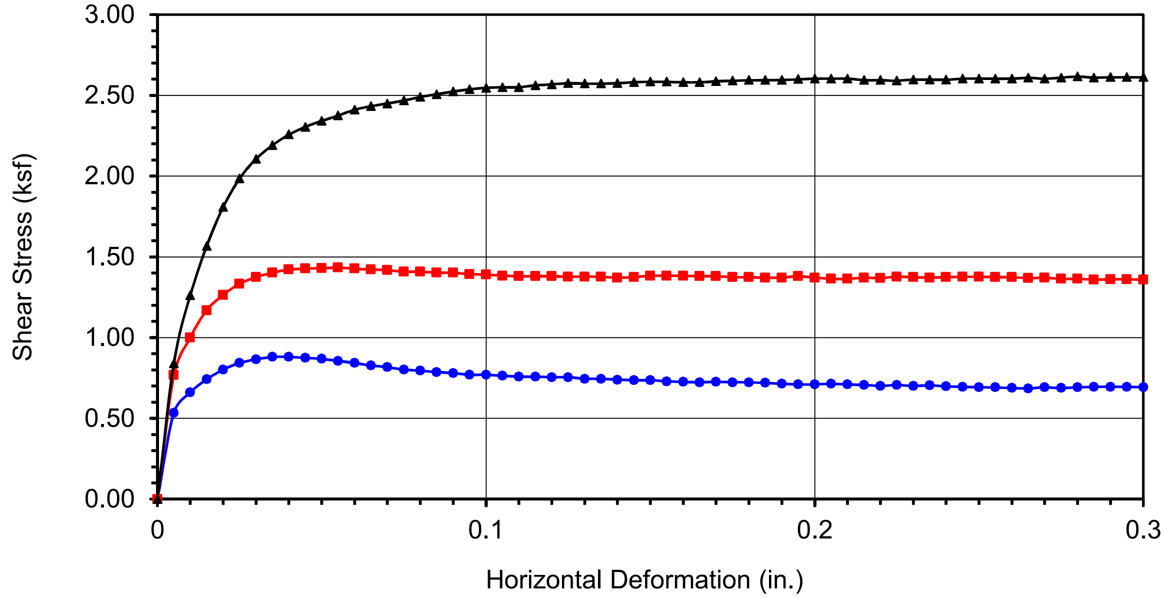
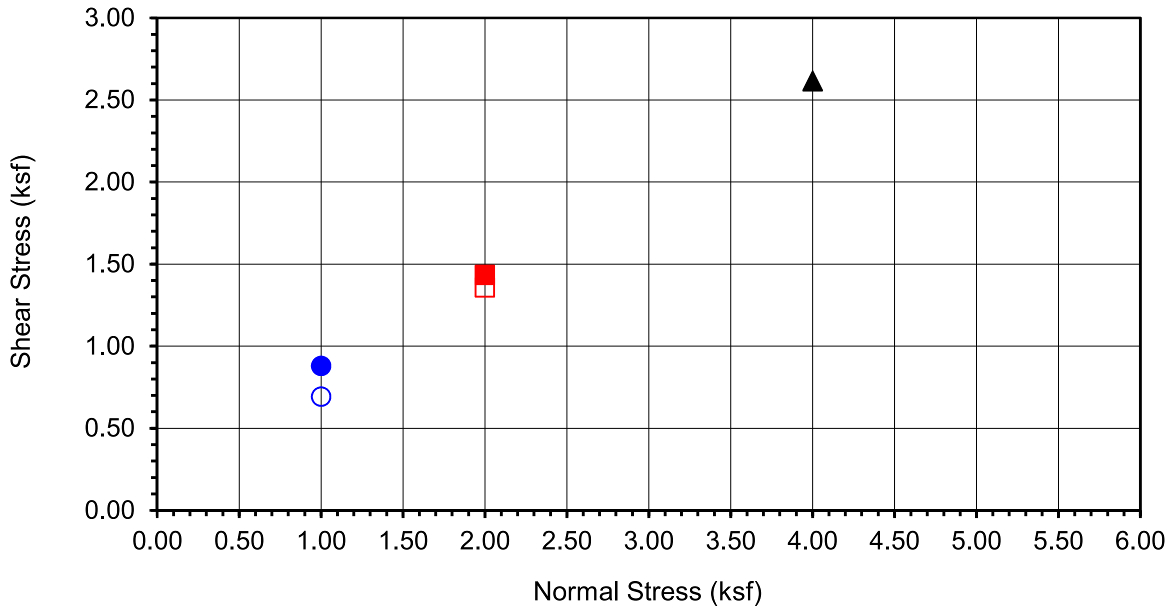
DIRECT SHEAR TEST

Consolidated Undrained

Project Name:	IntraCorp Quail Street	Tested By:	G. Bathala	Date:	06/06/22
Project No.:	13542.001	Checked By:	ACS/JHW	Date:	06/13/22
Boring No.:	LB-1	Sample Type:	90% Remold		
Sample No.:	BB-1	Depth (ft.):	0-5		
Soil Identification:	Yellowish brown silty sand (SM)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	199.25	199.79	199.64
Weight of Ring(gm):	45.41	45.59	45.35
Before Shearing			
Weight of Wet Sample+Cont.(gm):	161.97	161.97	161.97
Weight of Dry Sample+Cont.(gm):	153.80	153.80	153.80
Weight of Container(gm):	57.48	57.48	57.48
Vertical Rdg.(in): Initial	0.2760	0.2477	0.0000
Vertical Rdg.(in): Final	0.2858	0.2613	-0.0187
After Shearing			
Weight of Wet Sample+Cont.(gm):	225.36	197.66	219.71
Weight of Dry Sample+Cont.(gm):	207.17	179.95	202.57
Weight of Container(gm):	67.69	39.55	61.74
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-1**
Sample No.	**BB-1**
Depth (ft)	**0-5**

Sample Type:

90% Remold

Soil Identification:
Yellowish brown silty sand (SM)

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.880	■ 1.434	▲ 2.616
Shear Stress @ End of Test (ksf)	○ 0.692	□ 1.358	△ 2.612
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.48	8.48	8.48
Dry Density (pcf)	117.9	118.2	118.3
Saturation (%)	53.4	53.8	53.9
Soil Height Before Shearing (in.)	0.9902	0.9864	0.9813
Final Moisture Content (%)	13.0	12.6	12.2

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DIRECT SHEAR TEST RESULTS

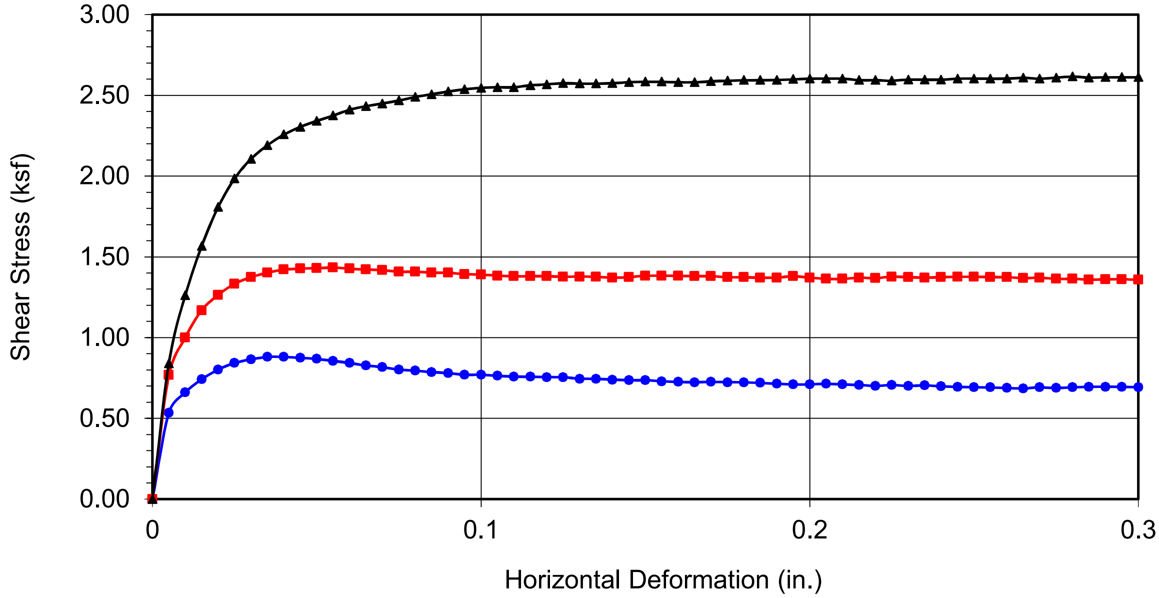
Consolidated Undrained

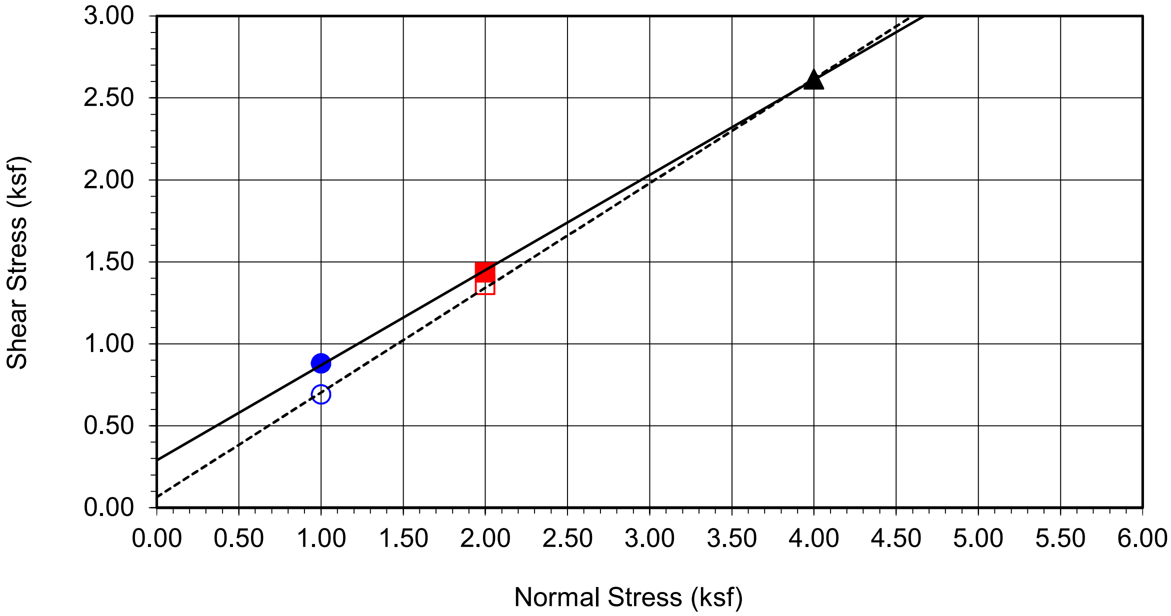
Project No.: 13542.001

IntraCorp Quail Street

06-22

DS LB-1, BB-1 @ 0-5





Boring No.	**LB-1**
Sample No.	**BB-1**
Depth (ft)	**0-5**
Sample Type:	90% Remold
Soil Identification: Yellowish brown silty sand (SM)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	289	30
Ultimate	65	33

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.880	■ 1.434	▲ 2.616
Shear Stress @ End of Test (ksf)	○ 0.692	□ 1.358	△ 2.612
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.48	8.48	8.48
Dry Density (pcf)	117.9	118.2	118.3
Saturation (%)	53.4	53.8	53.9
Soil Height Before Shearing (in.)	0.9902	0.9864	0.9813
Final Moisture Content (%)	13.0	12.6	12.2

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DIRECT SHEAR TEST RESULTS

Consolidated Undrained

Project No.: 13542.001

IntraCorp Quail Street

06-22

EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	IntraCorp Quail Street	Tested By:	G. Berdy	Date:	06/07/22
Project No.:	13542.001	Checked By:	ACS/JHW	Date:	06/13/22
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Yellowish brown silty sand (SM)				

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

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0010
Wt. Comp. Soil + Mold	(g)	639.10	453.70
Wt. of Mold	(g)	201.30	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	867.60	655.00
Dry Wt. of Soil + Cont.	(g)	810.80	610.46
Wt. of Container	(g)	0.00	201.30
Moisture Content	(%)	7.01	10.89
Wet Density	(pcf)	132.1	136.7
Dry Density	(pcf)	123.4	123.3
Void Ratio		0.366	0.367
Total Porosity		0.268	0.269
Pore Volume	(cc)	55.5	55.7
Degree of Saturation (%) [S_{meas}]		**51.7**	80.0

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.)
06/07/22	14:13	1.0	0	0.5560
06/07/22	14:23	1.0	10	0.5560
Add Distilled Water to the Specimen				
06/07/22	14:50	1.0	27	0.5570
06/08/22	5:07	1.0	884	0.5570
06/08/22	6:43	1.0	980	0.5570

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

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
LB-1	4.0							20.1	103.7		
LB-1	6.0							8.3	107.0		
LB-1	8.0							17.0	113.4		
LB-1	10.0							18.8	110.4		
LB-1	15.0							24.0			
LB-1	20.0							8.9	98.6		
LB-1	25.0							19.1			
LB-1	35.0							18.4			
LB-1	40.0							32.7			
LB-1	45.0							39.6			
LB-1	50.0							41.0			
LB-2	4.0							17.5	109.9		
LB-2	6.0							6.7	113.9		
LB-2	8.0							15.7	117.0		
LB-2	10.0							17.3	114.1		
LB-2	15.0							21.1			
LB-2	20.0							9.6	106.6		
LB-2	25.0							23.5			
LB-3	3.0							15.8	110.8		
LB-3	5.0							19.0	106.4		
LB-3	7.0							16.1	115.4		
LB-3	10.0							15.0	116.5		
LB-3	15.0							23.2	104.3		
LB-3	20.0							14.4			
LB-3	25.0							23.4			
LP-1	5.0							14.1	105.8		
LP-1	7.0							2.2	97.2		
LP-1	10.0							19.8	110.9		
LP-1	15.0							24.8	102.1		
LP-1	20.0							10.5			

US LAB SUMMARY 13542.001 INTRACORP QUAIL STREET.GPJ ROCKLOG2012.GDT 6/13/22

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Summary of Laboratory Results

Project Name: Intracorp Quail St

Project Number: 13542.001

Date: 6/13/2022 2:13:22 PM

Figure No. 1

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	IntraCorp Quail Street	Tested By:	J. Gonzalez	Date:	06/03/22
Project No.:	13542.001	Checked By:	A. Santos	Date:	06/06/22
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Yellowish brown silty sand (SM)				

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

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

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3813	3970	3918			
Weight of Mold (g)	1826	1826	1826			
Net Weight of Soil (g)	1987	2144	2092			
Wet Weight of Soil + Cont. (g)	427.0	402.6	443.2			
Dry Weight of Soil + Cont. (g)	407.0	374.0	402.8			
Weight of Container (g)	39.3	39.7	37.9			
Moisture Content (%)	5.44	8.56	11.07			
Wet Density (pcf)	131.5	141.9	138.5			
Dry Density (pcf)	124.8	130.8	124.7			

Maximum Dry Density (pcf) **130.9** **Optimum Moisture Content (%)** **8.3**

PROCEDURE USED

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

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

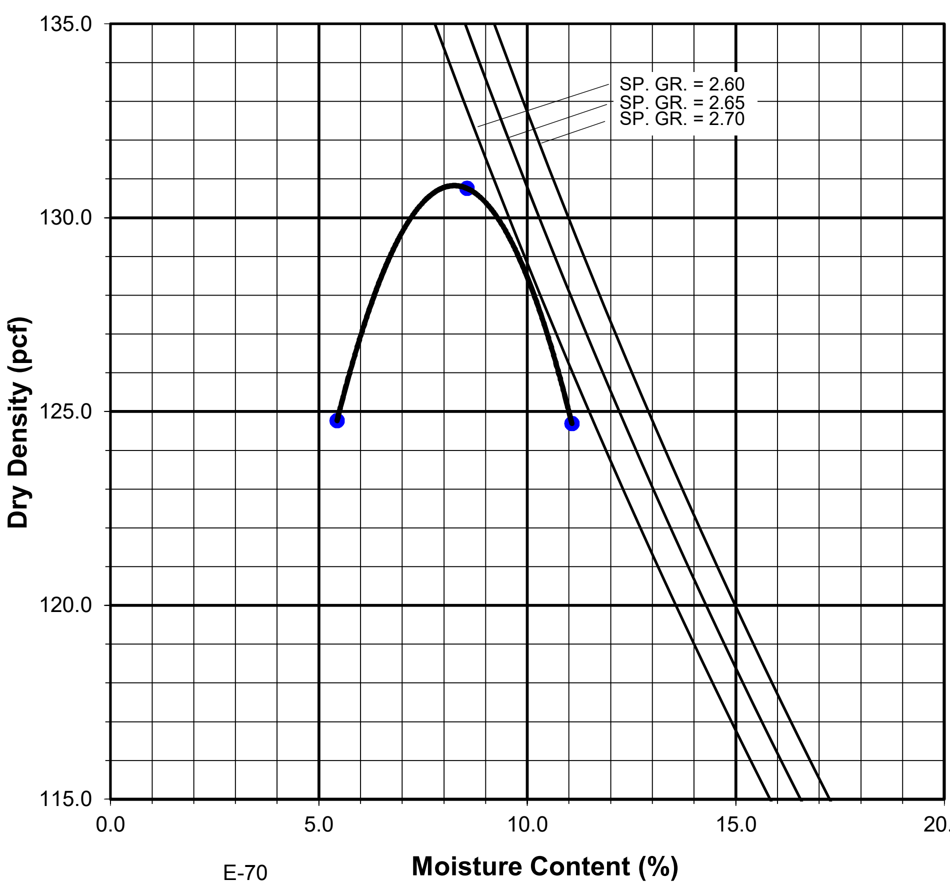
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



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ATTERBERG LIMITS
ASTM D 4318

Project Name:	IntraCorp Quail Street	Tested By:	J. Domingo	Date:	06/02/22
Project No. :	13542.001	Input By:	G. Bathala	Date:	06/07/22
Boring No.:	LB-3	Checked By:	ACS/JHW		
Sample No.:	R-5	Depth (ft.)	15.0		
Soil Identification:	Light yellowish brown fat clay (CH)				

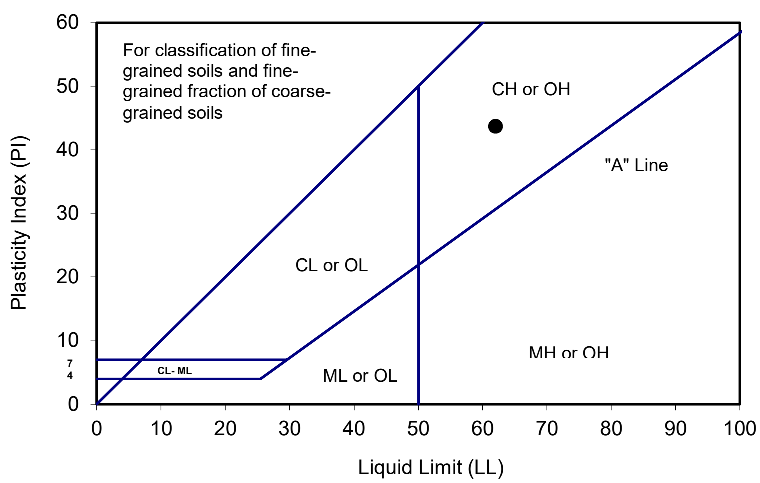
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			31	23	16	
Wet Wt. of Soil + Cont. (g)	9.13	9.10	20.64	20.94	20.41	
Dry Wt. of Soil + Cont. (g)	7.88	7.84	13.38	13.25	12.61	
Wt. of Container (g)	1.01	1.01	1.04	1.05	1.00	
Moisture Content (%) [Wn]	18.20	18.45	58.83	63.03	67.18	

Liquid Limit	**62**
Plastic Limit	**18**
Plasticity Index	**44**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) | 30.66

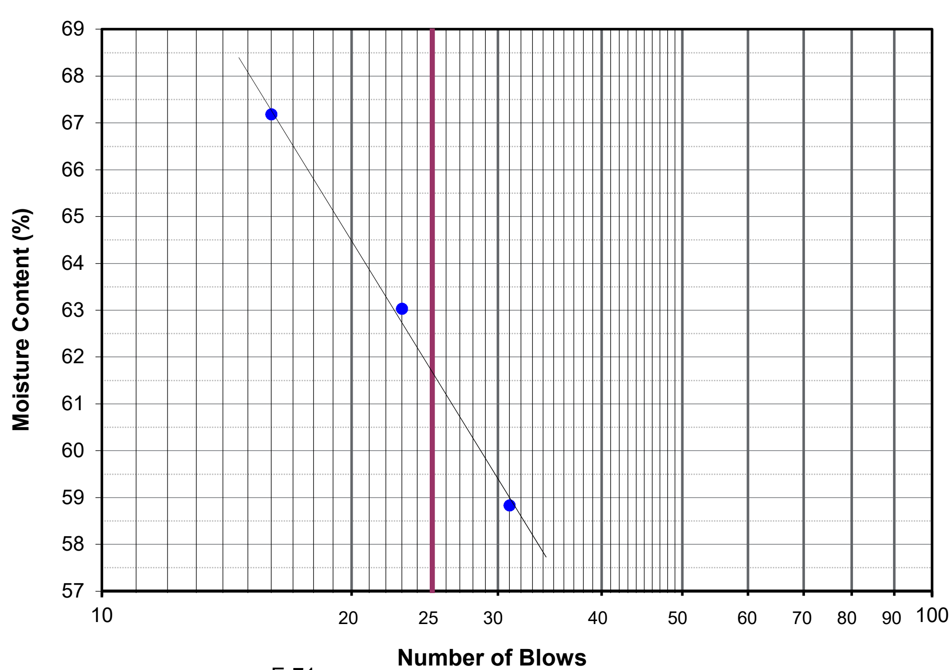
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



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PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 7928 & D 6913

Project Name: IntraCorp Quail Street | Tested By: GEB/JD | Date: 06/07/22

Project No.: 13542.001 | Checked By: J. Ward | Date: 06/13/22

Boring No.: LP-1

Sample No.: R-2 | Depth (feet): 7.0

Soil Identification: Olive yellow poorly-graded sand with silt (SP-SM)

		Soil Type
% Gravel	0	SP-SM
% Sand	94	
% Fines	6	

			Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(g)	0.00	84.95	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	84.90	169.63
Wt.of Air-Dry Soil + Cont. (g)	570.62	Wt. of Container No.___ (g)	1.00	66.67	75.84
Wt. of Container	96.83	Moisture Content (%)	0.00	0.27	
Dry Wt. of Soil (g)	473.79	Wt. of Dry Soil (g)			93.79

Coarse Sieve

U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
3/4"	0.00	100.0
3/8"	0.00	100.0
No. 4	2.23	99.5
No. 10	23.15	95.1
Pan		

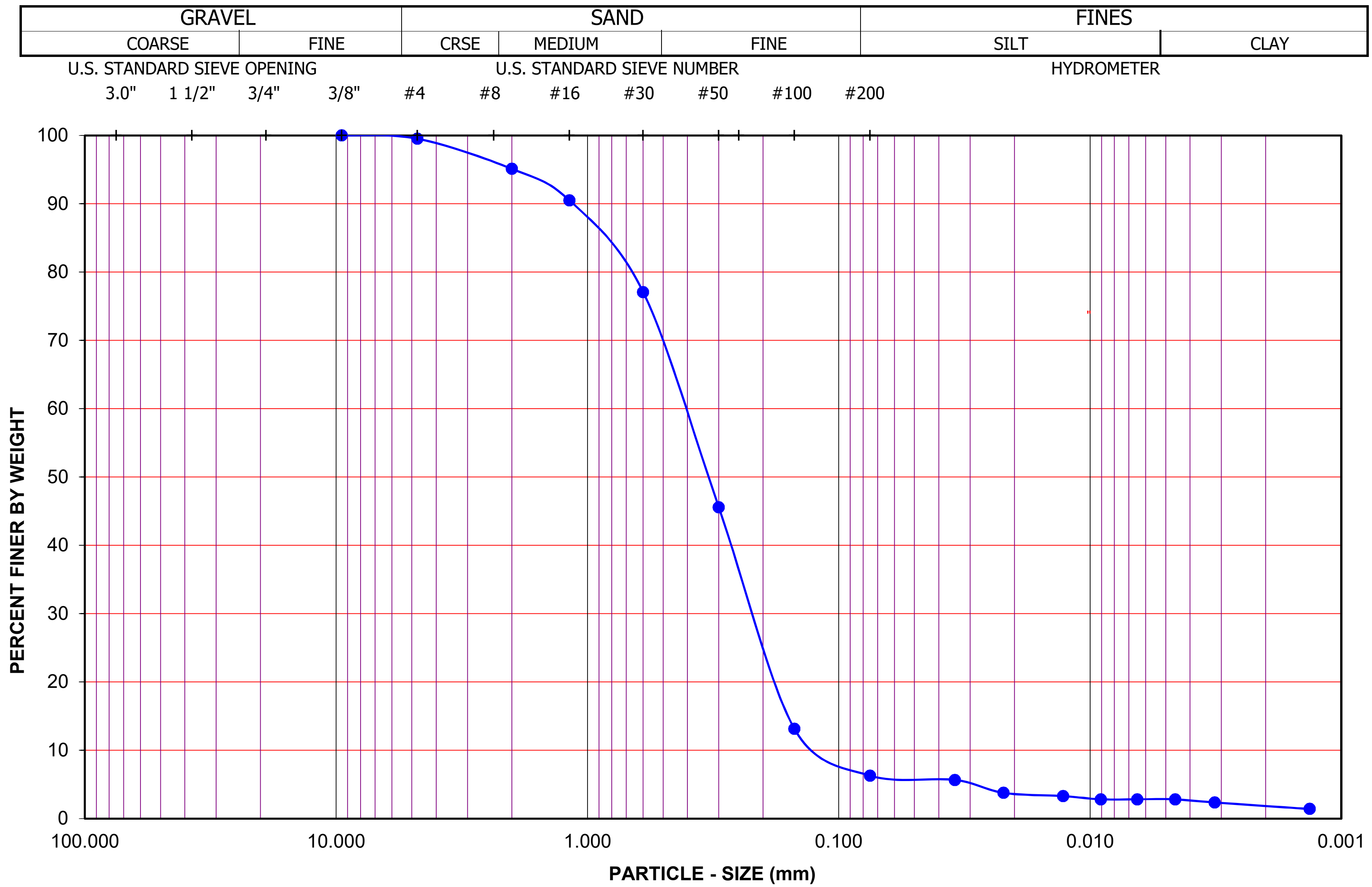
Sieve after Hydrometer & Wet Sieve

U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	95.1
No. 16	4.84	95.2	90.5
No. 30	18.95	81.0	77.1
No. 50	52.04	47.9	45.6
No. 100	86.11	13.8	13.1
No. 200	93.29	6.6	6.3
Pan			

Hydrometer Wt. of Air-Dry Soil (g) 100.18 Wt. of Dry Soil (g) 99.91

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
08-Jun-22	6:10	0		9.0			
	6:12	2	22.2	9.0	15.0	5.7	0.0345
	6:15	5	22.2	9.0	13.0	3.8	0.0221
	6:25	15	22.1	9.0	12.5	3.3	0.0128
	6:40	30	22.1	9.0	12.0	2.8	0.0091
	7:10	60	21.8	9.0	12.0	2.8	0.0065
	8:10	120	21.8	9.0	12.0	2.8	0.0046
	10:20	250	21.4	9.0	11.5	2.4	0.0032
09-Jun-22	6:10	1440	21.6	9.0	10.5	1.4	0.0013



Project Name: IntraCorp Quail Street

Project No.: 13542.001

Boring No.: LP-1

Sample No.: R-2

Depth (feet): 7.0

Soil Type : SP-SM

Soil Identification: Olive yellow poorly-graded sand with silt (SP-SM)

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PARTICLE - SIZE DISTRIBUTION
ASTM D 7928 & D 6913

GR:SA:FI : (%) 0 : 94 : 6

Jun-22

SA & Hyd LP-1, R-2 @ 7

APPENDIX D

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

Leighton

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

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT'D)

ATTACHMENTS

Standard Details

1.0 GENERAL

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

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

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

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction.

The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 <u>The Earthwork Contractor</u>

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 <u>Clearing and Grubbing</u>

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

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

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

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

2.2 Processing

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

2.3 Overexcavation

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

2.4 Benching

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

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

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 FILL MATERIAL

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 FILL PLACEMENT AND COMPACTION

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 SUBDRAIN INSTALLATION

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

6.0 EXCAVATION

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

7.0 TRENCH BACKFILLS

7.1 Safety

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

7.2 Bedding and Backfill

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

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

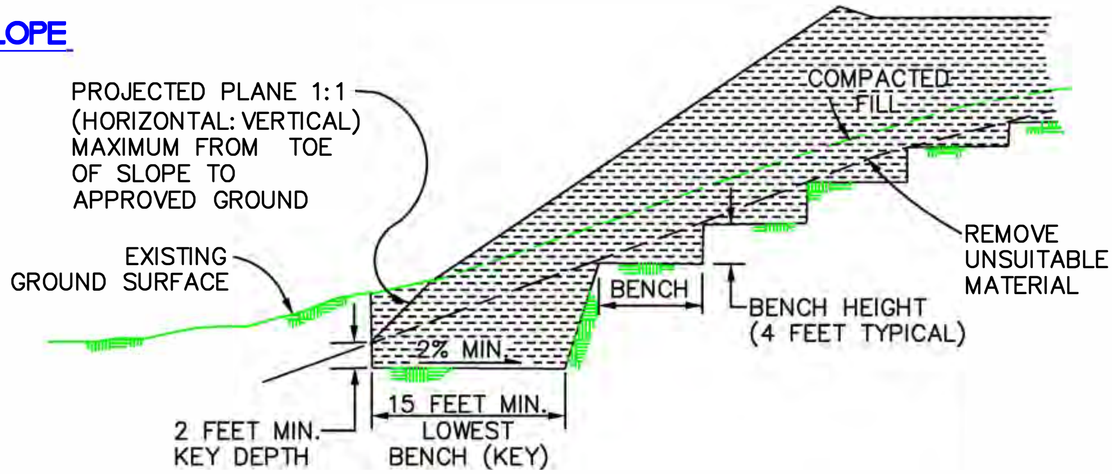
7.3 Lift Thickness

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

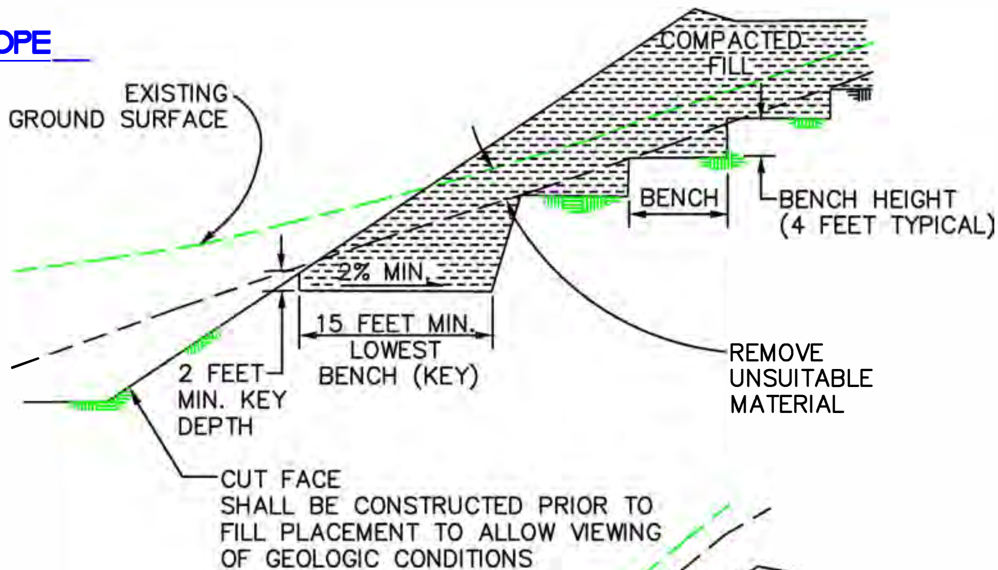
7.4 Observation and Testing

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

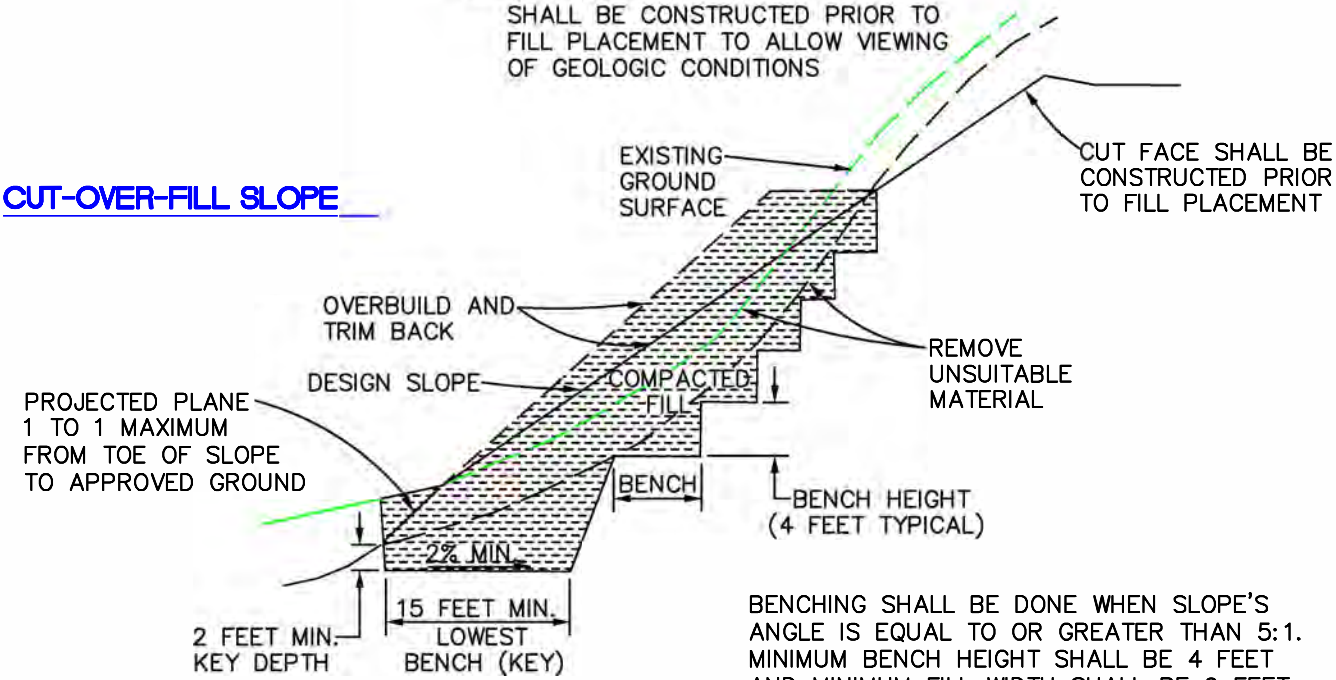
FILL SLOPE



FILL-OVER-CUT SLOPE

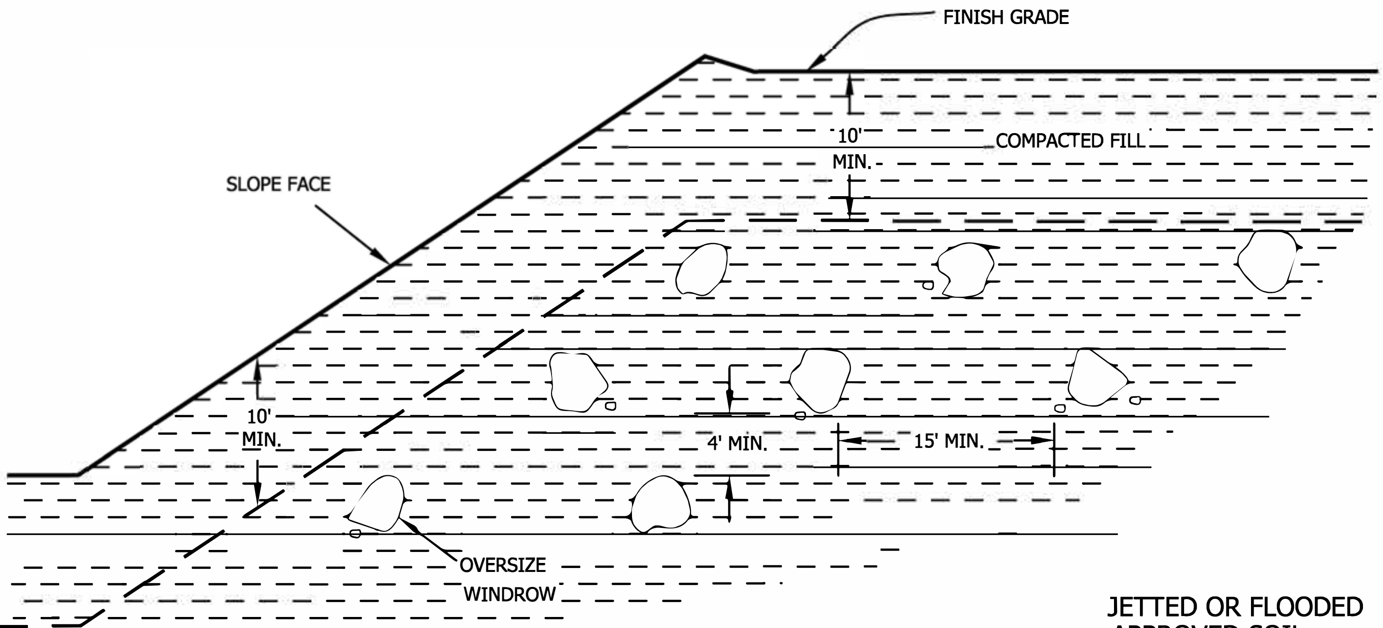


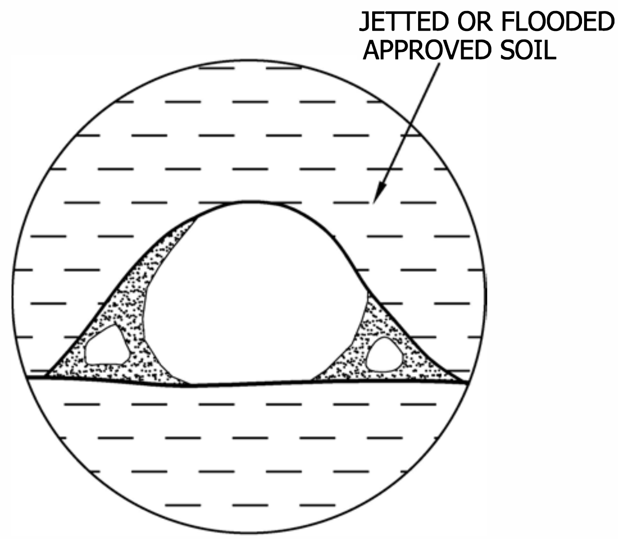
CUT-OVER-FILL SLOPE



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

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

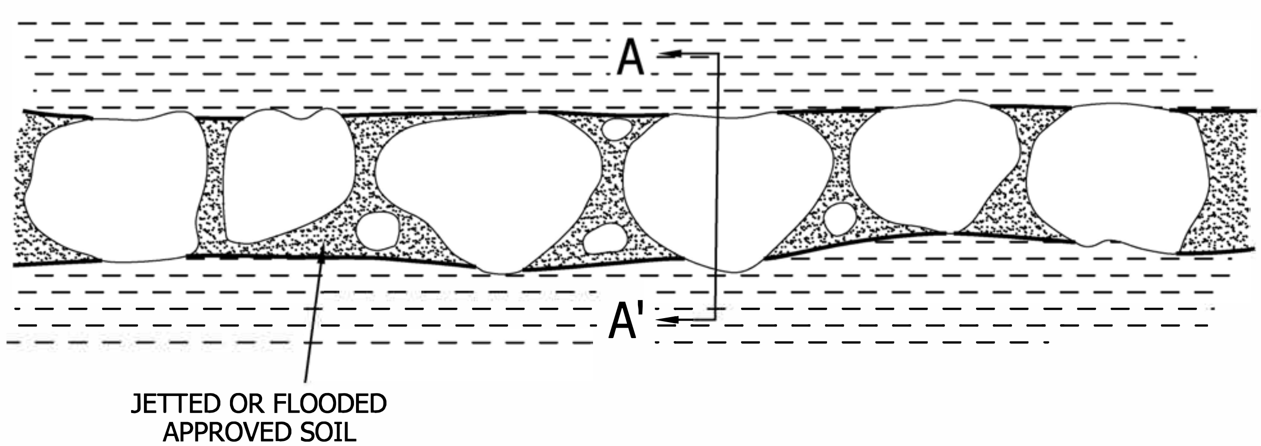




SECTION A-A'

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

PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B E-85	Leighton

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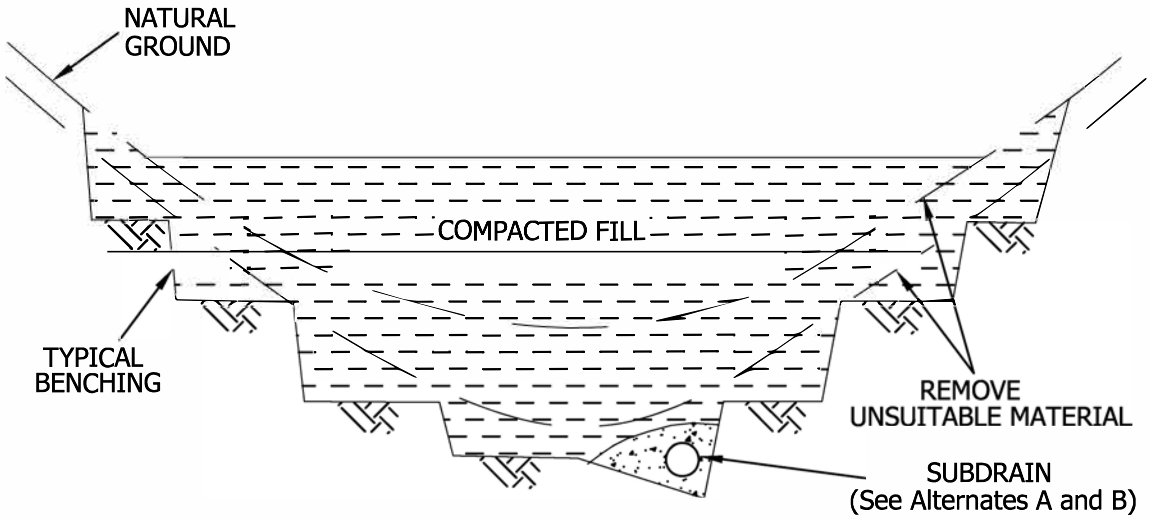
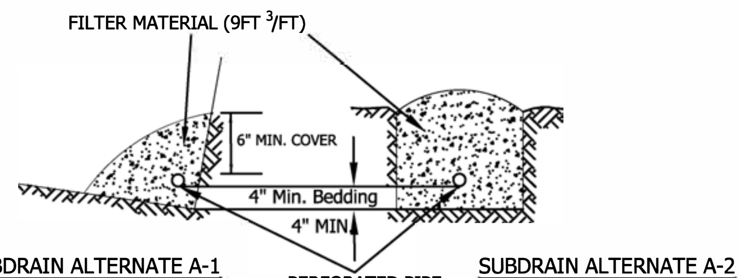


SUBDRAIN ALTERNATE A

PERFORATED PIPE SURROUNDED WITH FILTER MATERIAL



FILTER MATERIAL

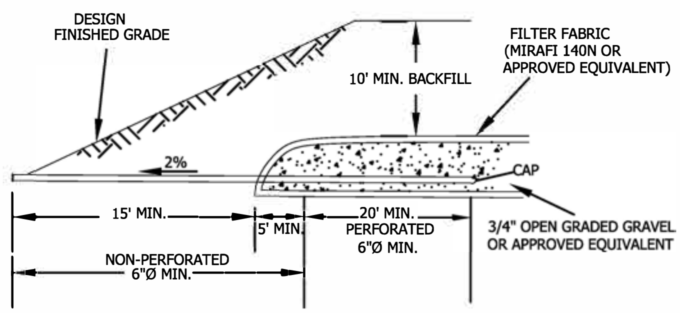
FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE. CLASS 2 GRADING AS FOLLOWS:

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

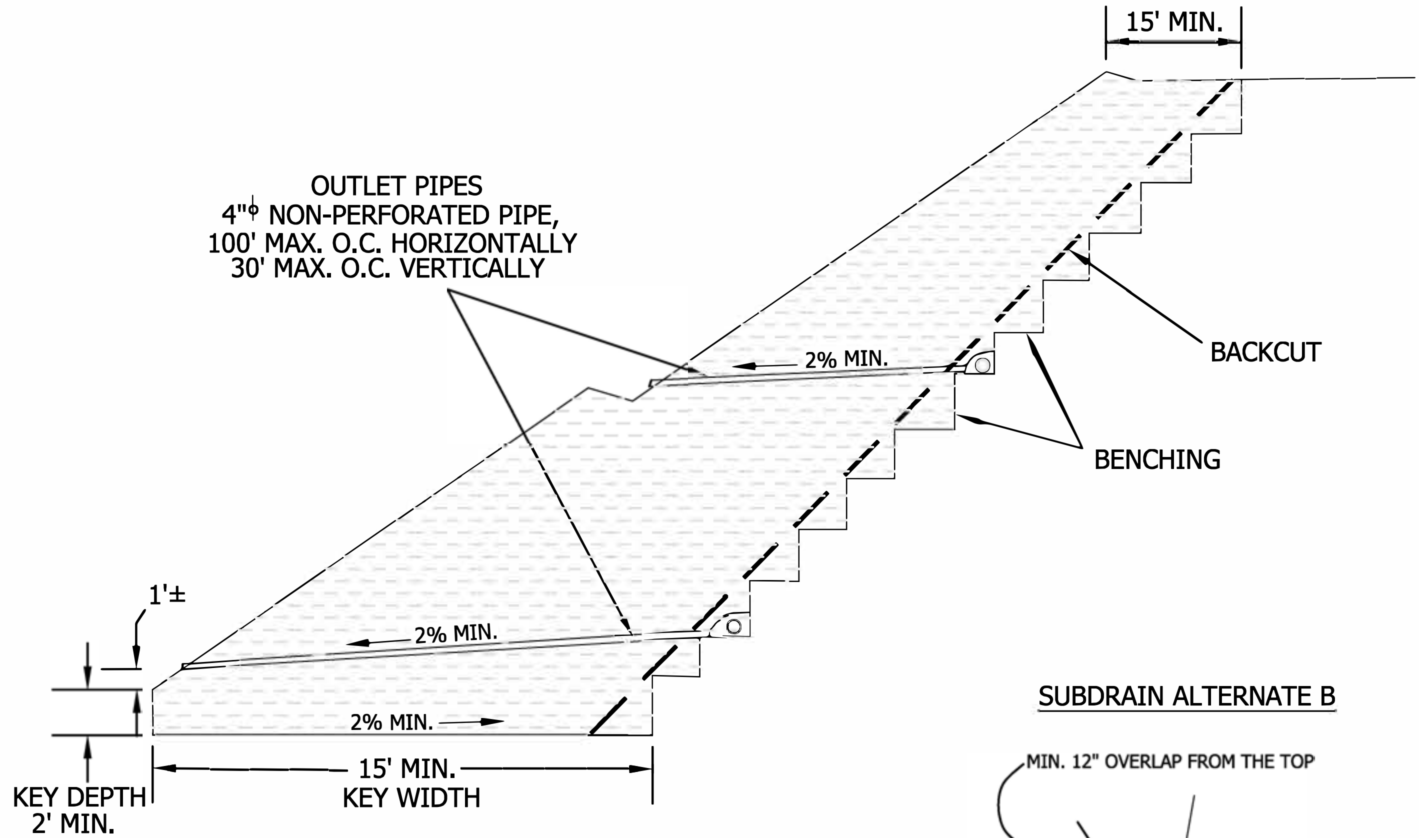
SUBDRAIN ALTERNATE B

○ PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

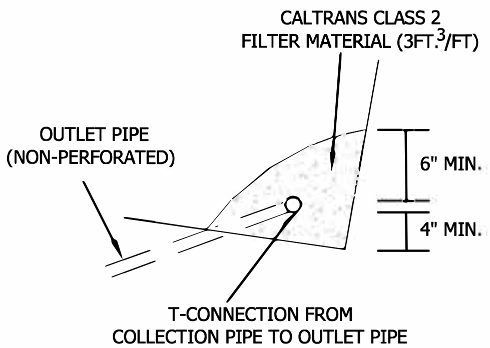
DETAIL OF CANYON SUBDRAIN TERMINAL



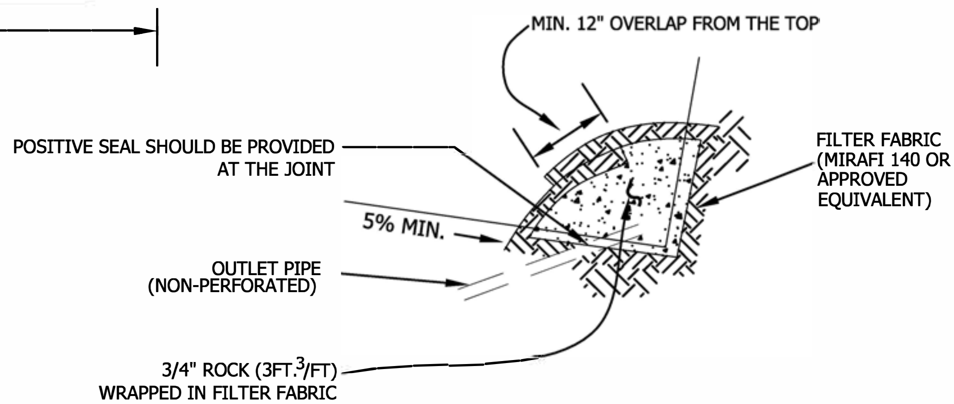
CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	Leighton



SUBDRAIN ALTERNATE A



SUBDRAIN ALTERNATE B



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

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

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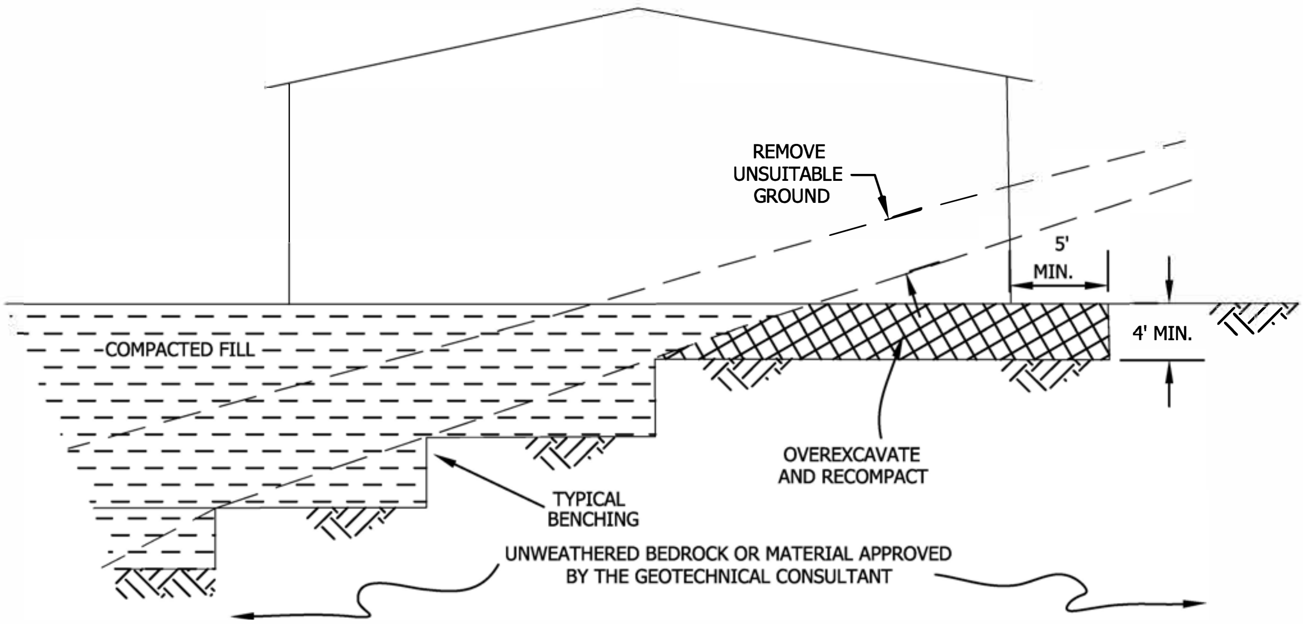
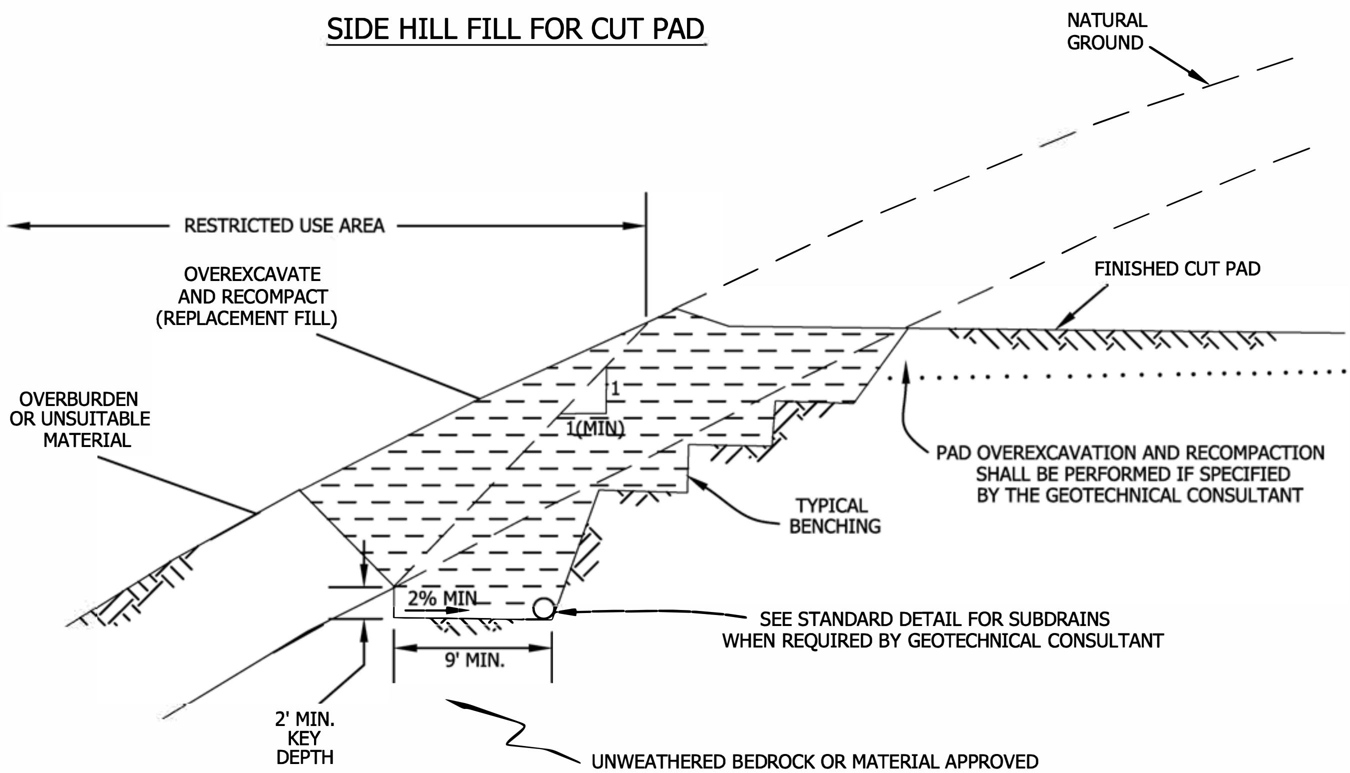
CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAILS E

Leighton