February 26, 2025

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

IDI Logistics
840 Apollo Street
Suite 343
El Segundo, CA 90245

Attention: Mr. Aaron Scherer
Vice President of Construction

Project No.: **17G199-6**

Subject: **Additional Subsurface Exploration**
Rider 2 - Proposed Site Improvements
NEC Redlands Avenue at Rider Street
Perris, California

Reference: Geotechnical Investigation, Rider 2 – Proposed Commercial/Industrial Building, NEC Redlands Avenue at Rider Street, Perris, California, prepared by Southern California Geotechnical, Inc. (SCG) for IDI Logistics, SCG Project No. 17G199-1R2, revised date March 3, 2021.

Mr. Scherer:

In accordance with your request, we have conducted a limited geotechnical investigation within the eastern portion of the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

This report should be distributed to all consultants and contractors associated with this project along with a copy of the referenced report.

## Scope of Services

The scope of services performed for this project was in general accordance with our Change Order No. 17G199-CO5 dated February 4, 2025. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide recommendations for preparing the design of the proposed screen walls and new pavements within the eastern portion of the subject site.

## Site and Project Description

The overall site is approximately 38.3 acres in size, located at the northeast corner of Redlands Avenue and Rider Street in Perris, California. The site is bounded to the north by a vacant lot, to the west by Redlands Avenue, to the south by Rider Street and to the east by the Perris Valley Flood Control Channel.

The overall site is developed with an 820,676± ft$^2$ single-story commercial/industrial building. The building is of concrete tilt-up construction, supported on a conventional shallow foundation system with a concrete slab-on-grade floor. The building is generally surrounded by Portland cement concrete (PCC) pavements and hardscape. The subject site of this report is the area located east of the existing building. The general location of the subject site is illustrated on the Boring Location Plan, enclosed as Plate 1 of this report.

The client has indicated that the existing PCC pavements will be replaced based on the higher truck traffic expected within the subject site. Based on the proposed site plan provided by the client, the proposed site improvements within the subject site will include, but not be limited to, screen walls and new PCC pavements. Based on the existing topography, cuts and fills of less than 1± foot are expected to be necessary to achieve the proposed site grades.

## Previous Studies

### Geotechnical Investigation

SCG previously conducted a geotechnical investigation for the existing development. The results of this investigation were presented in the above-referenced report. The subsurface exploration conducted for this project consisted of twelve (12) borings advanced to depths of 5 to 50± feet below the existing site grades.

Native alluvial soils were encountered at the ground surface at all twelve (12) of the boring locations. The alluvium varies widely in composition and strength, generally consisting of stiff to very stiff silty clays and clayey silts as well as loose to dense silty sands and fine sandy silts. These interbedded layers of sands, silts and clays, generally extended to at least the maximum depth explored of 50± feet.

Based on the subsurface conditions encountered at the boring locations, SCG recommended that a portion of the near-surface native alluvial soils be overexcavated to a depth of at least 4 feet below existing grade and to a depth of at least 4 feet below proposed building pad subgrade elevation. Where not encompassed within the general building pad overexcavation, additional overexcavation was recommended within the influence zones of the new foundations, to provide for a new layer of compacted structural fill extending to a depth of 3 feet below proposed bearing grade. Foundations were recommended to be designed for a maximum allowable soil bearing pressure of 2,500 lbs/ft$^2$.

### Earthwork Observation and Testing

As the geotechnical of record for the overall site, SCG previously observed the grading activities performed for this project. Minor grading was required for the existing pavements and flatwork areas within the subject site. The precise grading activities within the existing pavements and flatwork areas generally consisted of fills and cuts of less than 1± foot in order to establish the proposed elevations. The subgrade soils within the these areas were scarified, moisture treated, and then recompacted in general accordance with the recommendations provided in the project geotechnical report.

## Field Exploration

The subsurface exploration for the current project consisted of three (3) borings (identified as Boring Nos. B-13, B-14 and B-15) advanced to a depth of 15± feet below the existing site grades.

The boring locations were cleared by a private geophysical testing company prior to drilling. The borings were logged during drilling by a member of our staff.

The borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Representative bulk and undisturbed soil samples were taken during drilling. Relatively undisturbed samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. The sampler is driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

The approximate locations of the borings from the previous and current investigations are indicated on the Boring Location Plan, included as Plate 1 of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in this report.

## Geotechnical Conditions

### Artificial Fill

Artificial fill soils were encountered at the ground surface at Boring Nos. B-13, B-14 and B-15, extending to a depth of 3½± feet below the existing site grades. The fill soils generally consist of medium stiff to very stiff sandy clays with varying silt content. The fill soils possess a disturbed and mottled appearance resulting in their classification as artificial fill.

### Alluvium

Native alluvial soils were encountered beneath the artificial fill soils at Boring Nos. B-13, B-14 and B-15, extending to at least the maximum depth explored of 15± feet below the existing site grades. The alluvium generally consists of stiff to very stiff clayey silts, sandy clays and silty clays, with occasional medium dense silty sands to sandy silts. Boring No. B-13 encountered a stratum consisting of loose sandy silts at a depth of 8½ to 12± feet.

### Groundwater

Free water was not encountered during the drilling of Boring Nos. B-13, B-14 and B-15. Based on the lack of any water within the borings and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 15± feet at the time of the subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. Several monitoring wells are located within a mile radius of the subject site with high groundwater level readings ranging from 26 to 108± feet from the ground surface. Therefore, the high groundwater depth of 26± feet (February 2012) reported in a monitoring well located 0.75 miles east of the subject site is considered to be conservative with respect to the recent site conditions.

## Laboratory Testing

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

### Classification

Recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. Field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

### Dry Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

### Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 and C-2 of this report.

### Soluble Sulfates

A representative sample of the near-surface soil was submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

| Sample Identification | Soluble Sulfates (%) | Severity | Class |
|---|---|---|---|
| B-2 @ 2 to 6 feet | 0.006 | Not Applicable | S0 |

Corrosivity Testing

A representative bulk sample of the near-surface soils was submitted to a subcontracted corrosion engineering laboratory for determination of electrical resistivity, pH, and chloride concentrations. The resistivity of the soils is a measure of their potential to attack buried metal improvements such as utility lines. The results of some of these tests are presented below.

| **Sample Identification** | **Saturated Resistivity (ohm-cm)** | **pH** | **Chlorides (mg/kg)** | **Nitrates (mg/kg)** | **Sulfides (mg/kg)** | **Redox Potential (mV)** |
|---|---|---|---|---|---|---|
| B-2 @ 2 to 6 feet | 1,340 | 9.1 | 23.0 | 14.4 | 1.95 | 164 |

R-value

R (resistance)-value testing was conducted on a representative sample of the on-site soils within the subject site. The R-value was determined in accordance with CA Test Method 301. This test provides a measure of the pavement support characteristics of the soils, and is used in the pavement thickness design procedure. The result of the R-value testing is as follows:

| **Sample ID** | **R-Value** |
|---|---|
| B-15 @ 2-6 feet | 15 |

Based on the results of R-value testing and the soil conditions encountered at the remaining boring locations, the pavement sections for the proposed pavements are recommended to be designed for an R-value of 15.

## Seismic Design Considerations

Seismic Design Parameters

The 2022 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2022 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2022 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake ($MCE_R$) site accelerations at 0.01-degree intervals for each of the code documents. The table below was created using data obtained from the application. The output generated from this program is included as Plate E-1 with this report.

The 2022 CBC states that for Site Class D sites with a mapped $S_1$ value greater than 0.2, a site-specific ground motion analysis may be required in accordance with Section 11.4.8 of ASCE 7-16. Supplement 3 to ASCE 7-16 modifies Section 11.4.8 of ASCE 7-16 and states that "a ground motion hazard analysis is not required where the value of the parameter $S_{M1}$ determined by Eq. (11.4-2) is increased by 50% for all applications of $S_{M1}$ in this Standard. The resulting value of the parameter $S_{D1}$ determined by Eq. (11.4-4) shall be used for all applications of $S_{D1}$ in this Standard." The seismic design parameters presented in the table below were calculated using the site coefficients (Fa and Fv) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2022 CBC. It should be noted that the site coefficient Fv and the parameters $S_{M1}$ and $S_{D1}$ were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the ASCE 7-16 standard. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2022 CBC using the value of $S_1$ obtained from the Seismic Design Maps Tool. **The values of $S_{M1}$ and $S_{D1}$ tabulated below** were evaluated using equations 11.4-2 and 11.4-4 of ASCE 7-16 (Equations 16-20 and 16-23, respectively, of the 2022 CBC) and **do not include a 50 percent increase.** As discussed above, if a ground motion hazard analysis has not been performed, $S_{M1}$ and $S_{D1}$ must be increased by 50 percent for all applications with respect to ASCE 7-16.

**2022 CBC SEISMIC DESIGN PARAMETERS**

| Parameter | | Value |
|---|---|---|
| Mapped Spectral Acceleration at 0.2 sec Period | $S_S$ | 1.500 |
| Mapped Spectral Acceleration at 1.0 sec Period | $S_1$ | 0.578 |
| Site Class | --- | D |
| Site Modified Spectral Acceleration at 0.2 sec Period | $S_{MS}$ | 1.500 |
| Site Modified Spectral Acceleration at 1.0 sec Period | $S_{M1}$ | 0.995* |
| Design Spectral Acceleration at 0.2 sec Period | $S_{DS}$ | 1.000 |
| Design Spectral Acceleration at 1.0 sec Period | $S_{D1}$ | 0.664* |

*Note: These values must be increased by 50 percent if a site-specific ground motion hazard analysis has not been performed. However, this increase is not expected to affect the design of the structure type proposed for this site. This assumption should be confirmed by the project structural engineer. The values tabulated above do not include a 50-percent increase.

## Geotechnical Design Considerations

### General

Based on our subsurface exploration, the near-surface soils encountered within the proposed site improvement areas consist of artificial fill soils, extending to a depth of 3½± feet below the existing site grades. The existing fill soils are underlain by native alluvium which possesses favorable consolidation/collapse characteristics. Therefore, remedial grading is considered warranted within the proposed screen wall areas in order to remove the fill soils in their entirety and the upper portion of the near-surface alluvium, and replace these materials as compacted structural fill soils.

### Soluble Sulfates

The results of the soluble sulfate testing, discussed in a previous section of this report, indicate soluble sulfate concentrations less than 0.006 percent. These concentrations are considered to be negligible or "not applicable" with respect to the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3.

Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes.

Corrosion Potential

The results of laboratory testing indicate that the tested sample of the near-surface soils possesses a saturated resistivity of 1,340 ohm-cm, and a pH value of 9.1. The soils possess a redox potential of 164 mV and a sulfide concentration of 1.95 mg/kg. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity, pH, sulfide concentration, redox potential, and moisture content are the five factors that enter into the evaluation procedure. Based on these factors, the on-site soils are considered to be moderately corrosive to ferrous pipes. **Therefore, corrosion protection will be required for cast iron or ductile iron pipes.**

Based on American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary, reinforced concrete that is exposed to external sources of chlorides requires corrosion protection for the steel reinforcement contained within the concrete. ACI 318 defines concrete exposed to moisture and an external source of chlorides as "severe" or exposure category C2. ACI 318 does not clearly define a specific chloride concentration at which contact with the adjacent soil will constitute a "C2" or severe exposure. However, the Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 mg/kg are considered to be corrosive to reinforced concrete. The results of the laboratory testing indicate a chloride concentration of 23.0 mg/kg. Although the soils contain some chlorides, we do not expect that the chloride concentrations of the tested soils are high enough to constitute a "severe" or C2 chloride exposure. Therefore, a chloride exposure category of C1 is considered appropriate for this site.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested sample possesses a nitrate concentration of 14.4 mg/kg. Based on the test result, the on-site soils are not considered to be corrosive to copper pipe.

Since SCG does not practice in the area of corrosion engineering, we recommend that the client contact a corrosion engineer to provide a more thorough evaluation of these test results.

## Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications enclosed with the referenced geotechnical report, unless superseded by site-specific recommendations presented below.

Site Preparation

Site preparation should be performed in accordance with the recommendations provided in the referenced report.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of the proposed site walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill. Any existing undocumented fill soils should also be removed in their entirety. The overexcavation areas should extend at least 3 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to 2 to 4 percent above optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral extent of overexcavation is not achievable for the proposed walls, foundation elements must be redesigned using a lower bearing pressure. Further details regarding foundation design and construction for the retaining walls are presented in the subsequent section of this report.

Treatment of Existing Soils: Parking and Drive Areas

Based on economic considerations, overexcavation of the existing fill soils in the new parking and drive areas is not considered warranted, with the exception of areas where lower strength, or unstable, soils are identified by the geotechnical engineer during grading.

Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

The grading recommendations presented above for the proposed parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within these areas. The grading recommendations presented above do not completely mitigate the extent of existing undocumented fill soils in the parking and drive areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2022 CBC and the grading code of the City of Perris.

- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of low expansive (EI < 50), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included in the referenced geotechnical report.

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the City of Perris. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v (horizontal to vertical) plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

## Construction Considerations

Excavation Considerations

The near-surface soils within the subject site generally consist of sandy clays and clayey silts, with occasional sandy silts. Some of these materials may be subject to caving within shallow excavations. Where caving occurs, flattened excavation slopes may be sufficient to provide excavation stability. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. Temporary excavation slopes should be no steeper than 1.5h:1v. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

The near-surface soils possess appreciable silt and clay content and will become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a drier, less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pavement and screen wall areas as well as the need for a stabilization layer.

Groundwater

As previously discussed, the groundwater table is considered to be present at a depth in excess of 15± feet below the existing site grades. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

## Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new non-retaining site wall foundations will be underlain by structural fill soils extending to a depth of at least 3 feet below foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed site walls may be supported on conventional shallow foundations.

Foundation Design Parameters

The proposed non-retaining wall may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft$^2$.
- Maximum, net allowable soil bearing pressure: 1,500 lbs/ft$^2$ if the full recommended lateral extent of remedial grading cannot be achieved, typically for new footings along the property lines.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom).

The allowable bearing pressures presented above may be increased by one-third when considering short duration wind loads. However, the allowable bearing pressure should not be increased for seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the subgrade soils throughout the construction process.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 275 lbs/ft$^3$
- Friction Coefficient: 0.28

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2,500 lbs/ft$^2$.

## Portland Cement Concrete Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of sandy clays and clayey silts, with occasional sandy silts. These soils are generally considered to possess varying pavement support characteristics. Based on the results of R-value testing, the subsequent pavement design is therefore based upon an R-value of 15. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after the completion of rough grading to verify the R-value of the as-graded parking subgrade.

The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20-year design life, assuming six operational traffic days per week.

| Traffic Index | No. of Heavy Trucks per Day |
|---|---|
| 4.0 | 0 |
| 5.0 | 1 |
| 6.0 | 3 |
| 7.0 | 11 |
| 8.0 | 35 |
| 9.0 | 93 |

The minimum recommended thicknesses for the Portland cement concrete pavement sections are as follows:

| PORTLAND CEMENT CONCRETE PAVEMENTS (R = 15) | | | | |
|---|---|---|---|---|
| **Materials** | **Thickness (inches)** | | | |
| | Autos and Light Truck Traffic (TI = 6.0) | Truck Traffic | | |
| | | TI = 7.0 | TI = 8.0 | TI = 9.0 |
| PCC | 5 | 5½ | 7 | 8½ |
| Compacted Subgrade (95% minimum compaction) | 12 | 12 | 12 | 12 |

The concrete should have a 28-day compressive strength of at least 3,000 psi. Reinforcing within all pavements should be designed by the structural engineer. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness. The actual joint spacing and reinforcing of the Portland cement concrete pavements should be determined by the structural engineer.

**<u>Further Plan Reviews</u>**

It is recommended that copies of the final grading plans, when they become available, be provided to our office for review. We also recommend that our office review the foundation plans for the proposed site improvements, as they become available.

**Closure**

We sincerely appreciate the opportunity to be of continued service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

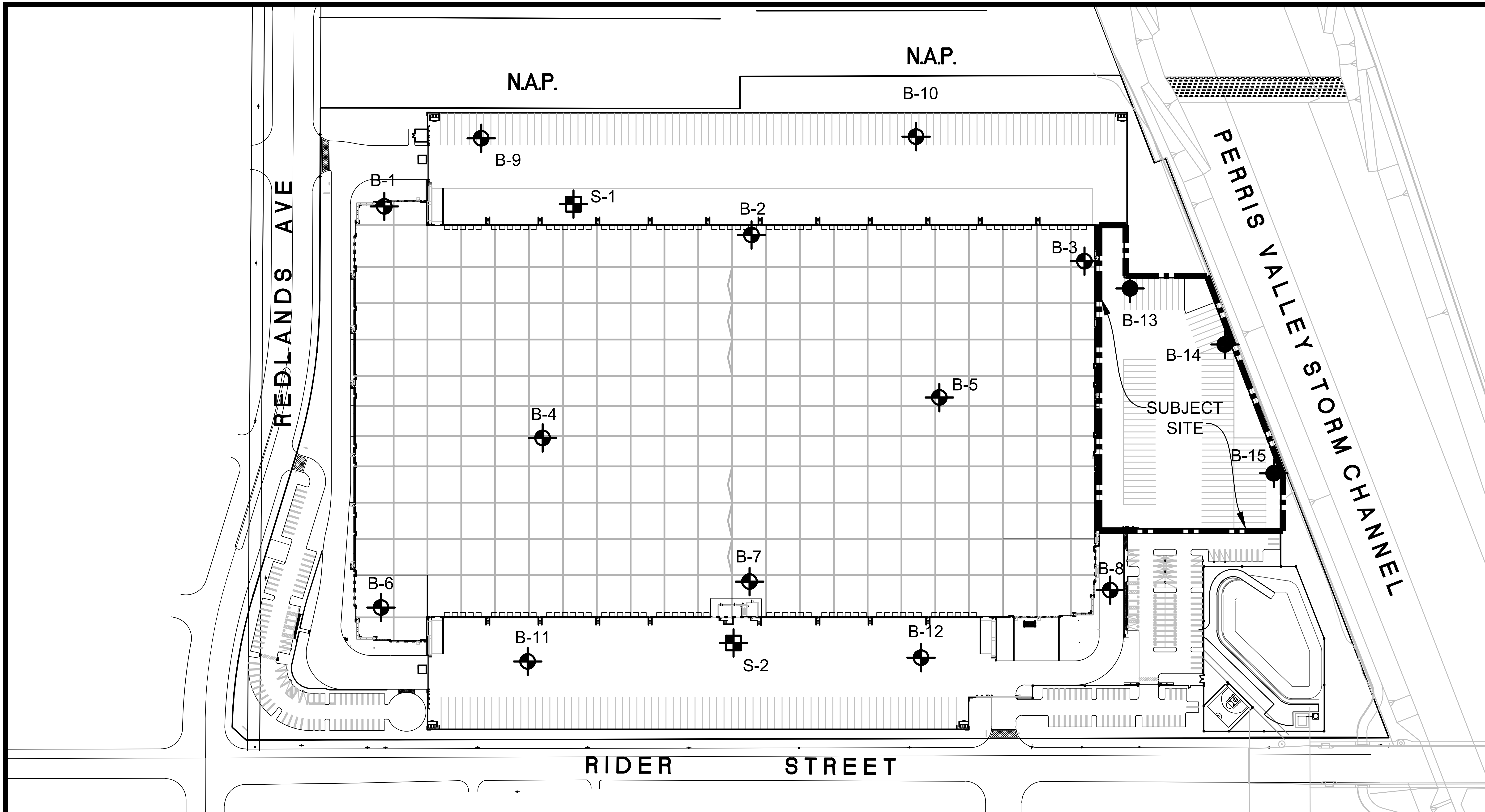
**SOUTHERN CALIFORNIA GEOTECHNICAL, INC.**

Joseph Lozano Leon
Staff Engineer

Gregory K. Mitchell, GE 2364
Principal Engineer

Enclosures: Plate 1: Boring Location Plan
Boring Log Legend and Logs
Plates C-1 and C-2: Consolidation/Collapse Test Results
Plate E-1: Seismic Design Parameters – 2022 CBC

Distribution: (1) Addressee




GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- PREVIOUS BORING LOCATION (SCG PROJECT NO 17G199-1R2)
- PREVIOUS R-VALUE SAMPLE LOCATION (SCG PROJECT NO 17G199-1R2)

NORTH
SoCalGeo

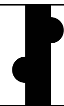
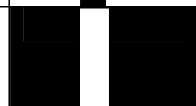
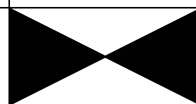
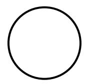
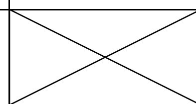
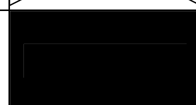
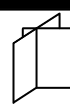
NOTE: SITE PLAN PREPARED BY HPA, INC.

**BORING LOCATION PLAN**
RIDER 2 - PROPOSED SITE IMPROVEMENTS
PERRIS, CALIFORNIA

SCALE: 1" =150'
DRAWN: JLL
CHKD: GKM
SCG PROJECT
17G199-6
PLATE 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

# BORING LOG LEGEND

| SAMPLE TYPE | GRAPHICAL SYMBOL | SAMPLE DESCRIPTION |
|---|---|---|
| **AUGER** |  | SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED) |
| **CORE** |  | ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK. |
| **GRAB** |  | SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED) |
| **CS** |  | CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED) |
| **NSR** |  | NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL. |
| **SPT** |  | STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED) |
| **SH** |  | SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED) |
| **VANE** |  | VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED. |

**COLUMN DESCRIPTIONS**

**DEPTH:** Distance in feet below the ground surface.

**SAMPLE:** Sample Type as depicted above.

**BLOW COUNT:** Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3” indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

**POCKET PEN.:** Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

**GRAPHIC LOG:** Graphic Soil Symbol as depicted on the following page.

**DRY DENSITY:** Dry density of an undisturbed or relatively undisturbed sample in lbs/ft$^3$.

**MOISTURE CONTENT:** Moisture content of a soil sample, expressed as a percentage of the dry weight.

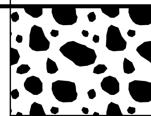
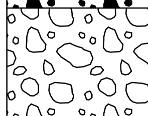
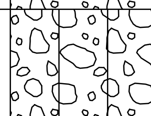
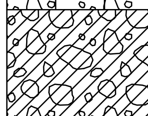
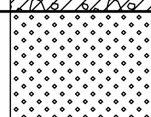
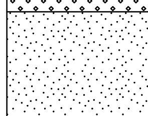
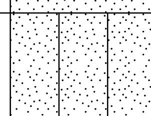
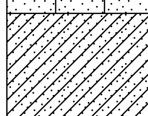
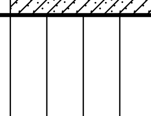
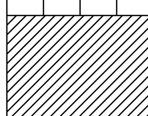
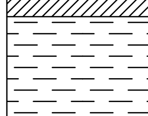
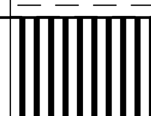
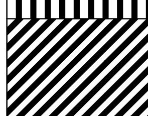
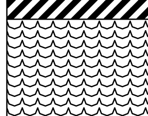
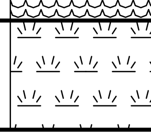
**LIQUID LIMIT:** The moisture content above which a soil behaves as a liquid.

**PLASTIC LIMIT:** The moisture content above which a soil behaves as a plastic.

**PASSING #200 SIEVE:** The percentage of the sample finer than the #200 standard sieve.

**UNCONFINED SHEAR:** The shear strength of a cohesive soil sample, as measured in the unconfined state.

# SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS | | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| | | | GRAPH | LETTER | |
| COARSE GRAINED SOILS<br><br>MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE | GRAVEL AND GRAVELLY SOILS<br><br>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | CLEAN GRAVELS<br><br>(LITTLE OR NO FINES) |  | GW | WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | |  | GP | POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | GRAVELS WITH FINES<br><br>(APPRECIABLE AMOUNT OF FINES) |  | GM | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | |  | GC | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| | SAND AND SANDY SOILS<br><br>MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE | CLEAN SANDS<br><br>(LITTLE OR NO FINES) |  | SW | WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES |
| | | |  | SP | POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES |
| | | SANDS WITH FINES<br><br>(APPRECIABLE AMOUNT OF FINES) |  | SM | SILTY SANDS, SAND - SILT MIXTURES |
| | | |  | SC | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS<br><br>MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE | SILTS AND CLAYS | LIQUID LIMIT LESS THAN 50 |  | ML | INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY |
| | | |  | CL | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS |
| | | |  | OL | ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY |
| | SILTS AND CLAYS | LIQUID LIMIT GREATER THAN 50 |  | MH | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS |
| | | |  | CH | INORGANIC CLAYS OF HIGH PLASTICITY |
| | | |  | OH | ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS |
| HIGHLY ORGANIC SOILS | | |  | PT | PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-13

JOB NO.: 17G199-6
PROJECT: Rider 2 - Proposed Site Improvements
LOCATION: Perris, California
DRILLING DATE: 2/11/25
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Caleb Brackett
WATER DEPTH: Dry
CAVE DEPTH: 10 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: ---- MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | FILL: Dark Brown fine Sandy Clay, little Silt, trace fine root fibers, very stiff-moist | | | | | | | |
| | | | | | | | 16 | | | | | |
| | | 37 | 4.0 | | | 111 | 13 | | | | | |
| | | | | | ALLUVIUM: Light Brown Clayey Silt, little fine Sand, abundant Calcareous nodulies/veining, very stiff-moist | | | | | | | |
| 5 | | 38 | 3.5 | | | 101 | 15 | | | | | |
| | | | | | Brown fine Sandy Clay, little Silt, little Calcareous nodulies/veining, stiff to very stiff-damp | | | | | | | |
| | | 19 | 3.5 | | | 120 | 9 | | | | | |
| | | | | | Brown fine Sandy Silt, trace to little Clay, loose to medium dense-moist | | | | | | | |
| | | 14 | | | | 118 | 11 | | | | | |
| 10 | | | | | | | | | | | | |
| | | 20 | | | | 111 | 14 | | | | | |
| 15 | | | | | | | | | | | | |
| | | | | | Boring Terminated at @ 15 feet | | | | | | | |

TBL 17G199-6.GPJ SOCALGEO.GDT 2/26/25

**TEST BORING LOG**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-14

JOB NO.: 17G199-6
PROJECT: Rider 2 - Proposed Site Improvements
LOCATION: Perris, California

DRILLING DATE: 2/11/25
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Caleb Brackett

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: ---- MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | FILL: Dark Brown fine Sandy Clay, trace fine root fibers, very stiff-moist | | | | | | | |
| | | | | | | | 18 | | | | | |
| | | 35 | 3.5 | | | 96 | 21 | | | | | |
| | | | | | ALLUVIUM: Light Brown Clayey Silt, abundant Calcareous nodulies/veining, very stiff-moist | | | | | | | |
| 5 | | 41 | | | Brown fine Sandy Silt, little Clay, little Calcareous nodulies/veining, medium dense-moist | 113 | 12 | | | | | |
| | | 20 | 3.5 | | Brown Silty Clay, little fine Sand, little Calcareous nodulies/veining, stiff to very stiff-moist | 110 | 15 | | | | | |
| | | 29 | 3.5 | | | 113 | 13 | | | | | |
| 10 | | | | | | | | | | | | |
| | | | | | Light Brown fine Sandy Silt, some Clay, trace medium Sand, medium dense-damp to moist | | | | | | | |
| | | 44 | | | | 117 | 9 | | | | | |
| 15 | | | | | | | | | | | | |
| | | | | | Boring Terminated at @ 15 feet | | | | | | | |

TBL 17G199-6.GPJ SOCALGEO.GDT 2/26/25

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-15

JOB NO.: 17G199-6
PROJECT: Rider 2 - Proposed Site Improvements
LOCATION: Perris, California

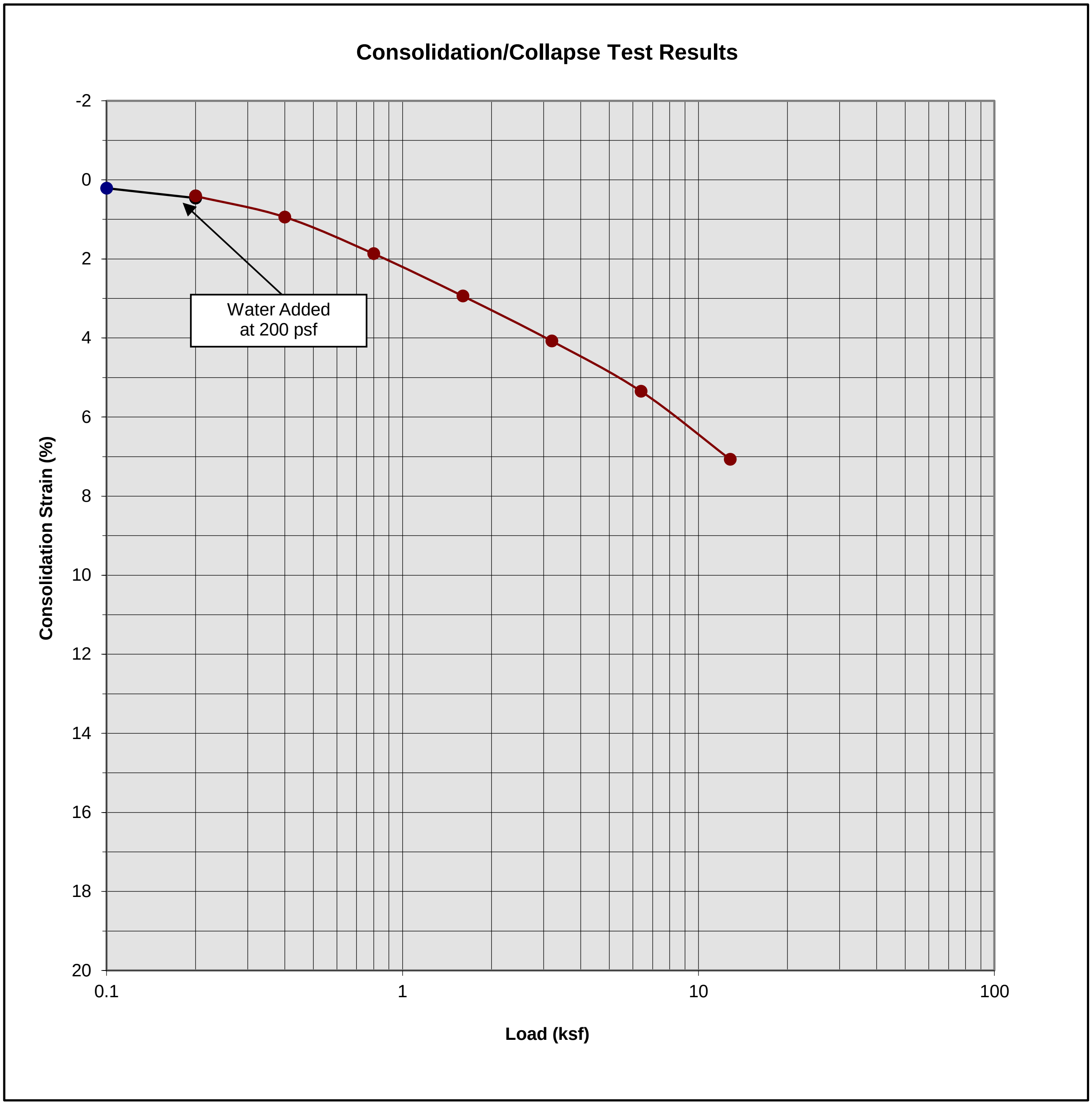
DRILLING DATE: 2/11/25
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Caleb Brackett

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | DESCRIPTION | LABORATORY RESULTS | | | | | | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | SURFACE ELEVATION: ---- MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | |
| | | | | | FILL: Dark Brown fine Sandy Clay, some Silt, trace fine root fibers, medium stiff to stiff-very moist | | | | | | | |
| | Bulk | | | | | | 27 | | | | | |
| | | 12 | 2 | | | 97 | 22 | | | | | |
| | | | | | ALLUVIUM: Light Brown Clayey Silt, trace to little fine Sand, abundant Calcareous nodules/veining, medium stiff to stiff-moist to very moist | | | | | | | |
| 5 | | 15 | 3.5 | | | 90 | 28 | | | | | |
| | | 29 | | | Brown fine Sandy Silt to Silty fine Sand, trace medium Sand, medium dense-damp to moist | 117 | 9 | | | | | |
| | | 30 | 3.5 | | Brown fine Sandy Clay, little Silt, medium very stiff-damp to moist | 124 | 9 | | | | | |
| 10 | | | | | | | | | | | | |
| | | | | | Brown Silty Clay to Clayey Silt, trace fine to medium Sand, very stiff-moist to very moist | | | | | | | |
| | | 43 | 3.5 | | | 114 | 18 | | | | | |
| 15 | | | | | | | | | | | | |

Boring Terminated at @ 15 feet

TBL 17G199-6.GPJ SOCALGEO.GDT 2/26/25

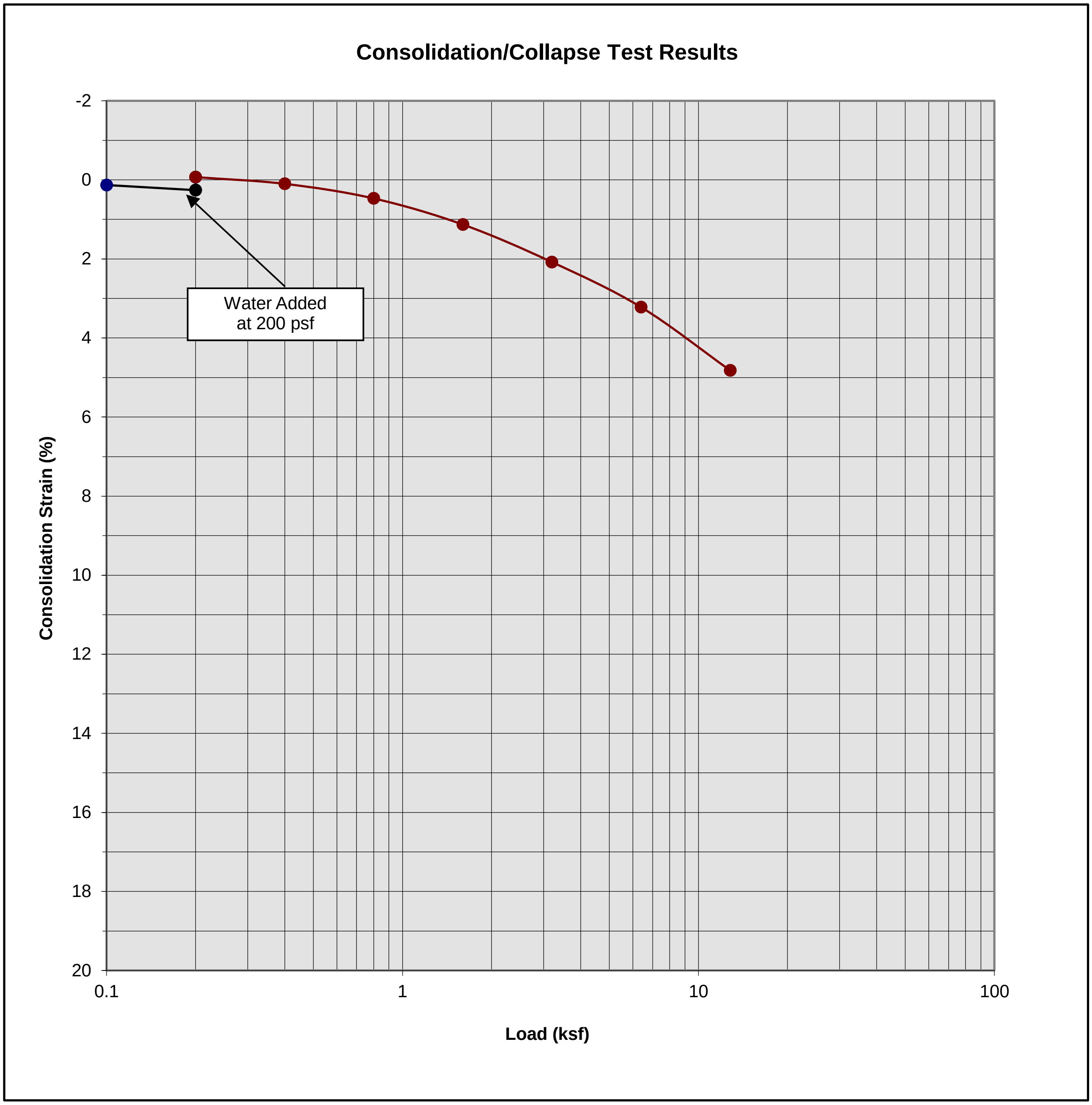



Classification: Light Brown Clayey Silt, trace to little fine Sand

| | | | |
|---|---|---|---|
| Boring Number: | B-15 | Initial Moisture Content (%) | 22 |
| Sample Number: | --- | Final Moisture Content (%) | 26 |
| Depth (ft) | 3½ to 4 | Initial Dry Density (pcf) | 97.1 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 103.5 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | -0.05 |

Rider 2 - Proposed Site Improvements
Perris, California
Project No. 17G199-6
**PLATE C- 1**

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
*A California Corporation*



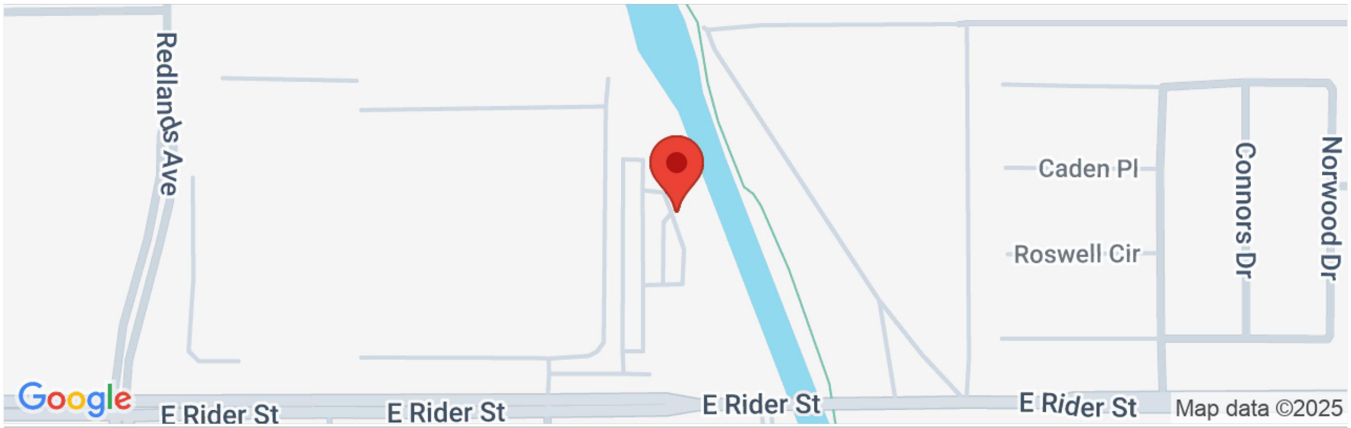

Classification: Light Brown Clayey Silt, trace to little fine Sand

| | | | |
|---|---|---|---|
| Boring Number: | B-15 | Initial Moisture Content (%) | 28 |
| Sample Number: | --- | Final Moisture Content (%) | 29 |
| Depth (ft) | 5 to 6 | Initial Dry Density (pcf) | 90.0 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 95.4 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | -0.33 |

Rider 2 - Proposed Site Improvements
Perris, California
Project No. 17G199-6
**PLATE C- 2**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Latitude, Longitude: 33.831807, -117.211708

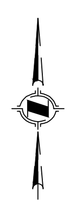



| | |
|---|---|
| **Date** | 2/19/2025, 3:55:57 PM |
| **Design Code Reference Document** | ASCE7-16 |
| **Risk Category** | II |
| **Site Class** | D - Stiff Soil |

| Type | Value | Description |
|---|---|---|
| $S_S$ | 1.5 | $MCE_R$ ground motion. (for 0.2 second period) |
| $S_1$ | 0.578 | $MCE_R$ ground motion. (for 1.0s period) |
| $S_{MS}$ | 1.5 | Site-modified spectral acceleration value |
| $S_{M1}$ | null -See Section 11.4.8 | Site-modified spectral acceleration value |
| $S_{DS}$ | 1 | Numeric seismic design value at 0.2 second SA |
| $S_{D1}$ | null -See Section 11.4.8 | Numeric seismic design value at 1.0 second SA |

| Type | Value | Description |
|---|---|---|
| SDC | null -See Section 11.4.8 | Seismic design category |
| $F_a$ | 1 | Site amplification factor at 0.2 second |
| $F_v$ | null -See Section 11.4.8 | Site amplification factor at 1.0 second |
| PGA | 0.501 | $MCE_G$ peak ground acceleration |
| $F_{PGA}$ | 1.1 | Site amplification factor at PGA |
| $PGA_M$ | 0.551 | Site modified peak ground acceleration |
| $T_L$ | 8 | Long-period transition period in seconds |
| SsRT | 1.541 | Probabilistic risk-targeted ground motion. (0.2 second) |
| SsUH | 1.653 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration |
| SsD | 1.5 | Factored deterministic acceleration value. (0.2 second) |
| S1RT | 0.578 | Probabilistic risk-targeted ground motion. (1.0 second) |
| S1UH | 0.635 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration. |
| S1D | 0.6 | Factored deterministic acceleration value. (1.0 second) |
| PGAd | 0.501 | Factored deterministic acceleration value. (Peak Ground Acceleration) |
| $PGA_{UH}$ | 0.657 | Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration |
| $C_{RS}$ | 0.932 | Mapped value of the risk coefficient at short periods |
| $C_{R1}$ | 0.911 | Mapped value of the risk coefficient at a period of 1 s |
| $C_V$ | 1.4 | Vertical coefficient |

SOURCE: SEAOC/OSHPD Seismic Design Maps Tool
<https://seismicmaps.org/>



**SEISMIC DESIGN PARAMETERS - 2022 CBC**

RIDER 2 - PROPOSED SITE IMPROVEMENTS

PERRIS, CALIFORNIA

DRAWN: JLL
CHKD: GKM

SCG PROJECT
17G199-6

**PLATE E-1**

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL