PRELIMINARY INVESTIGATION REPORT
RELATED BRISTOL PROJECT
SANTA ANA, ORANGE COUNTY, CA

Prepared for

RELATED CALIFORNIA RESIDENTIAL, LLC
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Prepared by

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Group Delta Project No. IR737
August 15, 2022

GROUP DELTA

Related California Residential, LLC
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August 15, 2022
Group Delta Project No. IR737

Attention: Mr. Steven Oh
Senior Vice President

Subject: Preliminary Geotechnical Investigation Report
Related Bristol Project
Santa Ana, Orange County, CA

Dear Mr. Oh,

Group Delta Consultants (Group Delta) is pleased to submit this geotechnical feasibility report for the Related Bristol Project in the City of Santa Ana of Orange County, California. Our scope of work was to perform a geotechnical assessment of the site to support your decisions and pricing of the proposed development. A limited field investigation and laboratory work were performed for the subject site and this report will not be suitable for final design. A comprehensive investigation involving additional field and laboratory work will be needed as the project proceeds to the design stage.

We appreciate the opportunity to provide geotechnical services for this project. Should you have any questions regarding this report, or if we can be of further service, please contact the undersigned.

Sincerely,
GROUP DELTA CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
MICHAEL JAMES GIVENS
NO. 3145
Exp. 9/30/2023
GEOTECHNICAL
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1.0 INTRODUCTION

1.1 General

This report presents the results of our geotechnical assessment for the proposed Related Bristol Development that is located in Santa Ana, California and is bounded by Sunflower Ave to the south, South Bristol St to the east, West MacArthur Blvd to the north and generally by South Plaza Dr. to the west as shown in Figure 1. The conceptual concept master plan and phasing are depicted in Figure 2. An aerial photograph of the site is shown in Figure 3. Detailed plans for the proposed development are not available at this time.

The purpose of our scope of work was to perform a geotechnical feasibility assessment for the proposed development, involving data review, field exploration work, limited laboratory testing, and limited engineering analysis. The aim of this study is to aid in your decisions and pricing of the proposed development. This report does not contain sufficient data for design nor for submission to the City of Santa Ana for permit approval.

1.2 Project Description

Related Bristol has been designed to be a new vibrant, walkable, people-first destination that makes a gateway into Santa Ana. The proposed mixed-use development program, which will be contained in a Specific Plan, includes up to 3,750 residential units, 250 hotel rooms, 200 units of senior continuum care, 350,000 square feet (sf) of retail and restaurant uses, and extensive open space connected by a network of landscaped paseos and pedestrian-friendly pathways.

Currently, the site is developed with approximately 465,000 (sf) of retail and respective paved parking lots. An Orange County Flood Control (OCFD) culvert and easement diagonally crosses the northeastern corner of the property near the existing Chase Bank building.

1.3 Objectives and Scope of Work

The objective of this report is to assess the feasibility of the proposed project from a geotechnical standpoint, including identifying the primary geotechnical factors that impact development at the site and preliminary geotechnical recommendations for the project. Our authorized scope of work includes:

- Review of available conceptual plans, geotechnical and geologic data, maps, and reports;
- Perform 7 Cone Penetration Tests (CPT) and 5 hollow stem auger borings to evaluate subsurface soil conditions;
- Install 1 temporary groundwater monitoring well;
- Perform 4 percolation tests to obtain unfactored infiltration rates;
- Perform limited laboratory testing to characterize the subsurface profile and to evaluate the engineering properties of the soils encountered;

- Perform limited engineering analyses to develop conceptual geotechnical recommendations for the site development, including recommendations for grading, foundations, active and passive earth pressures, and other construction-related issues such as shoring and foundation construction;
- Summarize our findings and preparing a preliminary geotechnical investigation report;
- Our geotechnical investigation excludes all issues related to environmental engineering, hazardous materials, and related matters.

2.0 GEOTECHNICAL INVESTIGATIONS

2.1 Field Explorations

A site investigation program for preliminary design for the project was undertaken on February 14, 2020 and January 4 and 5, 2021 that included the following:

- Seven (7) CPTs extending to depths ranging from 60 to 115 feet bgs;
- Five (5) hollow stem auger borings advanced to depths of 30 to 70 feet bgs; and
- Four (4) percolation tests at depth interval of 0 to 5 feet bgs.

One seismic cone penetration test (SCPT) was completed at the site. SCPT soundings recorded shear waves at intervals of 5 feet as well as the aforementioned standard CPT measurements.

The locations of our CPTs and exploratory borings, and percolation tests are shown in Figure 3. Prior to drilling, the locations were cleared through DigAlert, and the top 5 feet of drilling was performed with a hand auger to visually clear the hole of utilities. Additionally the locations were cleared of utilities by geophysical surveying. Details of the current Group Delta field exploration, including borings and CPT logs and interpretations are presented in Appendix A.

2.2 Laboratory Testing Program

The following limited laboratory testing was performed for this investigation to evaluate the physical properties and engineering characteristics of the subsurface materials encountered at the site.

- Moisture content and dry density (ASTM D2937, D2216);
- Atterberg limits (ASTM D4318);
- Percent passing No. 200 sieve (ASTM D1140);
- Sieve Analysis (ASTM C136);
- Soil Corrosivity (pH, Sulfate, Chloride, and Minimum Resistivity - CTM 417, 422 643);
- Expansion Index (ASTM D4829);
- Consolidation (ASTM D2435); and
- Unconsolidated-Undrained Triaxial Compression (ASTM D2850).

A detailed description of the laboratory testing program and test results are presented in Appendix B.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Regional Geology

The site is located within the Los Angeles Basin which is part of the Peninsular Range Geomorphic Province of California. The Peninsular Ranges are characterized by a series of northwest trending mountain ranges separated by valleys. Range geology consists of granitic rock intruding the older metamorphic rocks. Valley geology is typified by shallow to deep alluvial basins consisting of gravel, sand, silt and clay.

Specifically, the site is located at the southern margin of the Los Angeles Basin, which ends abruptly with the Newport-Inglewood uplift. The uplift is characterized by coastal mesas of late Miocene to early Pleistocene marine sediments and late Pleistocene marine terrace deposits.

Based on the geologic maps, the site is situated on Holocene alluvial soils. The near surface soils are characterized by young axial channel deposits. Figure 4 shows the regional geologic map of this section of Orange County.

3.2 Surface Conditions

The existing site is developed with approximately 475,000 square feet (sf) of retail and respective paved parking lots. An Orange County Flood Control (OCFD) culvert and easement diagonally crosses the northeastern corner of the property near the existing Chase Bank building. The current building configurations and pavement areas at the site are shown in the aerial image in Figure 3.

3.3 Subsurface Conditions

The subsurface soils at the site generally consist of three distinct soil zones to the maximum depth explored to 115 feet bgs, with the exception of CPT-1 where soil zone 3 described below was not well identified. The three soil zones are discussed below and have been schematically represented as cross-sections in Figure 5A and Figure 5B:

- Soil Zone 1 – The upper approximately 25 to 30 feet consists predominantly of medium stiff to stiff lean clay (CL) and fat clay (CH) that has a medium to high plasticity;
- Soil Zone 2 – Underlying soil zone 1 soils to a depth ranging between approximately 70 to 85 feet consists of a mixed soil condition with interbedded silty sand (SM), poorly-graded sands (SP) and lean clays (CL). CPT-1 located near the southwest property line exhibited this interbedded layer to depth explored.
- Soil Zone 3 – Underlying soil zone 2 is a very dense layer of poorly graded sands that ranges in thickness generally between 20 to 30 feet thick, with the exception of exploration CPT-1.

Preliminary analyses have been based on site-specific subsurface data. The subsurface stratigraphy has been interpreted based on the preliminary site investigation performed specifically for the Related Bristol project. For planning purposes and to highlight slight variations in subsurface profile across the site, the subsurface stratigraphy in Figures 5A through 5D has been grouped north and south of Callen's Common. The generalized soil profile and preliminary engineering properties are summarized in Table 1 and presented on Figures 5C and 5D for the northern and southern portion of the property, respectively. These are preliminary design values for planning purposes and do not represent the actual thickness encountered at all exploration locations.

Table 1: Generalized Soil Profile

Generalized Soil Zone	Depth[1] (feet bgs)	Predominant Soil Type	Internal Friction Angle, ϕ (deg)	Undrained Shear Strength, Su (psf)
1	0 to 30	Lean Clay (CL) and Fat Clay (CH)	-	750
2	10 to 80	Silty Sand (SM) and Poorly-Graded Sands (SP) with Interbeds of clays (CL/CH)	35	-
3	80 to 100	Poorly-Graded Sands (SP)	39	-

Note:
(1) Soil zones south of Callen's Common were encountered at a shallower depth compared to generalized soil profile. Soil zone 1 was encountered as shallow as 25 ft bgs and soil zone 3 as shallow as 70 bgs.

The subsurface investigation included a site-specific assessment of the static (small-strain) $V_{s,30}$, the time-weighted average shear wave velocity in the top 100 feet (30 meters). The $V_{s,30}$ was evaluated as a direct measurement of shear wave velocity from a seismic CPT and represents soils with non-liquefaction (static) strengths. The results of the V_s readings for each of the 5-foot intervals are provided in Figures 5C and 5D. The $V_{s,30}$ was taken down to a depth of 100 feet bgs. The $V_{s,30}$ measurements indicate that soil is Site Class D.

3.3 Groundwater

Historic highest groundwater at the site has been mapped at a depth of about 5 feet bgs (CGS, 1997). Groundwater was encountered during the current preliminary site investigation between a depth of 12 feet and 16 feet bgs (El. 23 to 17 feet NAVD88). Groundwater levels measured during the geotechnical investigations are a "snapshot" of the groundwater level and do not account for potential fluctuations in groundwater level due to seasonal and tidal variations. No nearby existing groundwater monitoring wells were available for review of long-term groundwater trends. A temporary groundwater monitoring well was installed at boring B-1 and can be utilized for investigating seasonal variation.

3.4 Infiltration Rates

Our investigation included percolation testing at four locations shown in Figure 3. Percolation locations were drilled using a truck mounted rig to a maximum depth of 5 feet bgs. Groundwater was not encountered at the explored depths of the percolation test locations. Our field procedures were conducted in accordance with the Orange County Technical Guidance Document (OCTGD) for the Water Quality Management Plan (WQMP).

Percolation testing was performed in accordance with the OCTGD Section VII, Infiltration Rate Protocol and Factor of Safety Recommendations. The wells were installed using 3-inch-diameter schedule 40 PVC solid and screen-wall casing. Logs of the percolation borings are shown in Appendix A. After the completion of the percolation tests, the wells were abandoned, PVC pipes were removed, and the boreholes backfilled with clean sand and cold patch asphalt for finishing.

The results of the percolation field tests are summarized in Table 2. The onsite soils above the groundwater typically consist of lean clay materials and based on the percolation test results are not suitable for infiltration.

Table 2. Field Unfactored Infiltration Rates

Test ID (Boring)	Approximate Ground Elevation (feet)	Location	Field Infiltration Rate (in/hr)	Predominant Soil Type	Bottom of test hole Elevation (feet)	Depth of Test Interval (feet)
P-1	34	Boring B-1	<0.1	Lean Clay (CL)	29	0 to 5
P-2	33	CPT C-2	<0.1	Lean Clay (CL)	28	0 to 5
P-5	34	Boring B-5	<0.1	Lean Clay (CL)	29	0 to 5
P-6	34	CPT C-6	<0.1	Lean Clay (CL)	29	0 to 5

4.0 Discussion and Recommendations

4.1 Potential Seismic Hazards

The site is located in a seismically active region of Southern California. The site is subjected to seismic hazards during its design life. Potential seismic hazards include strong ground shaking, ground surface rupture due to faulting, liquefaction and seismic settlement, and slope instability. The following sections discuss these potential seismic hazards with respect to the proposed development.

4.1.1 Ground Surface Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Zone and Figure 6 shows the site regional fault activity map of southern California. The closest two active faults are the San Joaquin Hills fault and Newport-Inglewood fault zones that are located at about 1.3 and 4.1 miles from the site, respectively. The San Joaquin Hills fault located closest to the site is a blind thrust fault that does not rupture at the ground surface. Due to the distance from the major faults, fault rupture is not a significant hazard for the site.

4.1.2 Liquefaction and Seismic Settlement

Liquefaction involves the sudden loss in strength of a saturated, cohesionless soil (sand and non-plastic silts) caused by the build-up of pore water pressure during cyclic loading, such as produced by an earthquake. This increase in pore water pressure can temporarily transform the soil into a fluid mass, resulting in vertical settlement and can also cause lateral ground deformations. Typically, liquefaction occurs in areas where there are loose to medium dense granular soils and the depth to groundwater is less than 50 feet from the surface.

Based on our site-specific field investigation, subsurface material at the site are predominantly clayey soils to a depth of approximately 30 feet below the existing ground surface and underlying soils are mixed soil condition with interbedded dense to very dense silty sand (SM), poorly-graded sands (SP) and lean clays (CL). Considering the cohesive and dense nature of the soils in the upper 50 feet, liquefaction is considered low.

4.1.3 Seismic Slope Stability

The site is generally level and no post-construction slopes are planned. Therefore, slope stability in not considered a hazard at the site. The site is not within a seismic-induced landslide hazard zone area.

4.1.4 Flood Hazard Zone

The project site is in an area with reduced flood risk due to levee and is determined to be outside the 0.2% annual chance floodplain as defined by the United States Federal Emergency Management Agency.

4.1.5 Other Seismic Hazards

All low-lying areas along California's coast are subject to potentially dangerous tsunamis. Due to the site being about 6 miles away from the ocean and site elevation (about El. 34 feet), tsunamis are not a hazard at the site.

4.2 Preliminary Seismic Design Parameters

Mapped seismic design acceleration parameters were developed in accordance with 2019 California Building Code (CBC) and ASCE 7-16 (ASCE/SEI 7-16). Based on the subsurface exploration and underlying geology, the site classification for seismic design is Site Class D, in accordance with Chapter 20 of ASCE 7-16. The preliminary seismic design parameters for the site were calculated using the SEAOC/OSHPD Seismic Design Mapping Tool (Version 5.1.0) and are presented in Table 3.

Table 3. Preliminary Seismic Design Parameters

Parameter (Latitude: 33.6970, Longitude: -117.8871)	Value
Site Class	D
Mapped MCE Spectral Response Acceleration at Short Period (S_S)	1.287
Mapped MCE Spectral Response Acceleration at Period of 1 Second (S_1)	0.462
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.838
Adjusted MCE Spectral Response Acceleration at Short Period (S_{MS})	1.287
Adjusted MCE Spectral Response Acceleration at Period of 1 Second (S_{M1})	0.849
Design Earthquake Spectral Response Acceleration at Short Period (S_{DS})	0.858
Design Earthquake Spectral Response Acceleration at Period of 1 Second (S_{D1})	0.566
Peak Ground Acceleration Adjusted for Site Class (PGA_M)	0.550

Mapped design acceleration parameters are required to meet Exception 2 of Section 11.4.8 of ASCE 7-16. for Site Class D. Therefore the mapped design values may only be used if Exception 2 below is met:

- **If $T \leq 1.5\ T_S$**: The value of the seismic response coefficient C_S is determined by Eq. (12.8-2), i.e., S_{DS} is used to obtain C_S
- **If $T_L \geq T > 1.5\ T_S$**: The value of seismic response coefficient C_S is taken as ***1.5 times*** the value computed in Eq. (12.8-3), i.e., ***1.5****S_{D1} is used to obtain C_S, or
- **If $T > T_L$**: The value of seismic response coefficient C_S is taken as ***1.5 times*** the value computed in Eq. (12.8-4), i.e., ***1.5****S_{D1} is used to obtain C_S.

Based on this exception, if the fundamental period is less than or equal to $1.5T_S$, S_{DS} must be used to determine the seismic response coefficient, C_S, with equation 12.8-2. If the fundamental period is higher than 1.5 T_S (longer period structures), then the determination of C_S is increased by a factor of 1.5.

Depending upon the structure type, fundamental period of the structure, and structural analysis method, either site-specific values or mapped values (meeting Exception 2 of ASCE 7-16, Section 11.4.8) may be used. However, a site-specific acceleration response spectrum is recommended for final design if tall buildings are progressed into the final concept and can be provided in accordance with Chapter 21 of ASCE 7-16.

4.3 Expansive Soils

The upper 25 to 30 feet bgs of the site is generally composed of clayey material that are medium to highly expansive. Expansion and contraction can occur when expansive soils undergo alternating cycles of wetting (swelling) and drying (shrinking). During these cycles, the volume of the soil changes markedly, and can cause structural damage to buildings and infrastructure. Expansive soils are generally high plasticity clays.

Expansion index testing was performed on two soil samples collected in the recent investigation. The tests were performed on bulk sample of the upper 5 feet from borings B-1 and B-5 that, respectively, had an expansion index of 85 and 120, which indicates a medium to high expansion potential. Based on the Atterberg limit testing performed for the proposed project, the soils tested had liquid limits greater than 46 and plasticity index greater than 31. Moderately to highly expansive soils are present at the site and the foundation should be designed to resist these expansion pressures or these soils should be removed to sufficient depth.

4.4 Soil Corrosion Potential

The subsurface soils in the upper 25 to 30 feet generally consist of lean and fat clay alluvial deposits. One representative sample of the near surface soils from Borings B-4 was tested to evaluate corrosion characteristics. The test included pH, electrical resistivity, soluble chloride, and soluble sulfate concentrations. Test results are summarized in Table 4 below and are provided in Appendix B.

Table 4. Corrosion Potential Test Results

Sample/Depth	pH	Resistivity [Ohm-cm]	Sulfate Content [ppm]	Chloride Content [ppm]
B-4 @ 0-5’	7.7	371	10,274	377

Based on large sulfate content of the test sample, the near surface soils are considered corrosive to concrete. The correlation below can generally be used between electrical resistivity and corrosion potential.

Electrical Resistivity (Ohm-Cm)	Corrosion Potential
Less than 1,000	Severe
1,000 to 2,000	Corrosive
2,000 to 10,000	Moderate
Greater than 10,000	Mild

Based on the soluble chloride concentration and electrical resistivity results, the test sample is classified as severely corrosive to buried metals. Further evaluation/testing and recommendations for corrosion protection should be provided by a corrosion consultant.

5.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

5.1 General

Based on our understanding of the conceptual plan for the proposed development, several building typologies and associated loading demands have been considered for planning purposes to identify feasibility of foundation types. At-grade and one subterranean level are being considered at this stage for most structures with the exception of wrap-around Type III wood residential structures. For the purpose of preliminary foundation design the following structures have been evaluated:

- Five-story Type III wood frame residential structure at-grade or one-level below grade (e.g. wrap-around residential);
- Podium Structure - three-story concrete podium with five-story Type III wood construction above the podium;
- Six-story or shorter concrete structure (e.g. business and residential)
- Eight-story or taller concrete structure (e.g. hotel and residential)
- Five-story concrete short-span parking structure (e.g. residential wrap-around parking);
- Six-story concrete long-span parking structure (e.g. centralized mixed-use parking);

Preliminary structural loads have been provided by DCI Engineers for the aforementioned structure types and are presented in Figure 7.

Geotechnical design considerations at the site include:

- Shallow groundwater (measured at approximately 12 ft bgs);
- Shallow expansive clayey soils (from ground surface to approximately 25 to 30 ft bgs);
- Moderately compressible soils and settlement potential; and
- Low infiltration rates.

Expansive soils at the site will require mitigation measures and/or incorporation of expansive forces into structural design to protect the proposed development from cyclic expansion and contraction from wetting and drying. The mitigation measures could include special drainage provisions to minimize water infiltration into soils below structures and/or overexcavation and replacement of expansive soils below foundations, slabs and flat work (see Section 5.8). Foundations, slabs and flatwork can be structurally designed to resist bending forces in-lieu of removal and replacement of existing soils. Removal and replacement will require import of very low expansive soils as discussed in Section 6.4.

One subterranean level is being considered for podium structures to facilitate additional parking below ground and it has been assumed for preliminary design that the foundation would be situated approximately 14 ft bgs. Due to the shallow design groundwater level consistent with the mapped historic high, the subterranean level walls will require waterproofing and the foundation will require design for buoyant forces.

Group Delta believes there are several types of foundations that may be utilized at the site and choice is dependent on the building typology and whether the building is built at grade or with one subterranean level. The preliminary recommendations for foundation design are provided in the sections below and has been summarized in Figure 7.

5.2 Type III Wood Frame Residential Structures

A five-story Type III wood construction residential building at-grade or one-level below grade can be founded on conventional shallow foundations, mat slab, post-tension slab, or deep foundations. Based on the presence of the expansive material, a normal slab on grade is not feasible without removal and replacement of 4 feet of expansive material with low expansive material and recommendations in Section 5.8 should be followed.

The following preliminary design criteria for shallow foundations are recommended:

- Shallow spread footings should have a minimum dimension of 2 feet;
- Shallow continuous footings should have a minimum dimension of 1.5 feet;
- Individual spread footings should bear on a minimum of 4 feet of low expansive fill;
- Preliminary allowable bearing pressure are provided in Table 5 and these recommended bearing values may be increased by one-third for wind, seismic or other transient loading conditions;
- Short term static settlements for the footing pressures in Table 5 are expected to be 1 inch or less; and
- A differential settlement equal to one-half of the total settlement over a distance of 30 feet can be used for planning purposes.

Table 5. Allowable Bearing Pressure for Type III At-Grade Wood Structures

Footing Width (Feet)	Allowable Bearing Pressure (1) (psf)
2	1,800
5	1,100

Note:
(1) Values can be linearly interpreted for intermediate footing widths.
(2) Values determined based on 1 inch of settlement or less.

As an alternative to shallow foundations a mat slab, post-tensioned slab or deep foundation could be utilized. Mat or post-tensioned slabs should be a minimum embedment of 24-inches below the lowest adjacent soil grade and designed by the structural engineer for high expansion potential, if removal and replacement with low expansive material is not chosen. Deep foundations discussed in Section 5.4 are also a viable foundation option.

5.3 Podium Structures

Podium structures vary from one- to three-story of concrete podium above ground with up to five-story of wood construction above the podium. The podium structures are also planned to have one-level below grade. The maximum loads of the three-story concrete podium and basement level with five-story of Type III wood construction above the podium is considered for evaluation of the foundation options.

Mat slabs are capable of providing satisfactory support to podium structures one-level below grade, if designed to reduce concentrated bearing loads from column loads and design to resist expansive soil. The amount of settlement will be dependent on the rigidity of the mat slab and transmission of loading to the ground. Mat slab foundations have a variable capacity to spread loading from column and perimeter wall loads. The two extremes can be thought of as a concentrated larger direct column point load when there is a very thin slab to a fairly uniform loading across the foundation when there is a very thick and heavily reinforce slab. The mat slab should be designed by the structural engineer and the preliminary column spacing has been assumed to be at 30 feet center-to-center.

Preliminary settlement analyses have been performed and are provided in Table 6 to provide an anticipated performance criterion for preliminary planning of the structural thickness and reinforcement of the mat slab. The preliminary settlement analyses consider the following two scenarios:

1. Uniform loading over a large mat slab footprint that evaluates impacts of settlement to greater depths; and
2. An equivalent footing loading with variable concentrated bearing load over smaller areas to represent mat slabs that are not perfectly rigid.

Table 6. Mat Slab (One-Level Below Grade) Settlement Estimates

Foundation Element	Footing Width (feet)	Footing Length (feet)	Bearing Load (psf)	Estimated Settlement (inches)
Uniform Mat	300	300	900	2.0
Column Loading on Equivalent Footing	15	15	3,500	4.6
	20	20	2,000	2.9
	25	25	1,250	1.7
	30	30	900	1.1

Ground improvement can be utilized to control total and differential settlements as discussed in Section 5.6. As an alternative, deep foundations as discussed in Section 5.4 may be utilized. Post-tension slabs may be utilized as a structural slab to resist expansive soils if a mat is not preferred and either deep foundations or ground improvement will be required.

5.4 Concrete Structures

5.4.1 Concrete Structures 6-Story or Less

Six-story or shorter concreate structures are being considered for the project with either one or two basement levels. These structures have similar loads as the podium structures with preliminary column loads provided in Figure 7. The column loads of the four-story to six-story concrete structures range from 490 kips to 750 kips. Therefore, six-story concrete structures with basement levels can follow the foundation recommendations in Sections 5.3.

5.4.2 Concrete Structures 8-Story or Taller

Eight-story or taller concrete structures are being considered with one level of basement, such as a hotel, residential, and assisted living facilities. Deep foundations are necessary to support taller than eight-story concrete structures at the site. Shallow and mat slab foundations are not considered feasible for these structures given the large column loads and potential settlement. Preliminary column loads provided in Figure 7 indicate that from eight-story to 24-story the column loads range from 1,020 kips to 3,280 kips. In addition, a structural slab on grade will be required to address expansive forces of the soil and inclusion of a subterranean level would need to design for hydrostatic buoyant forces.

The following deep foundations have been considered for the project:

- Driven piles;
- Drilled shafts (also referenced as Cast-In-Drilled-Hole, CIDH piles); and
- Auger cast piles.

It is Group Delta's opinion that the ACD piles provide the most benefit for the project considering the planned staging of construction and subsurface conditions. ACD piles are installed by rotating a continuous flight hollow shaft auger into the soil to a specified depth. High strength sand cement grout is pumped through the hollow shaft as the auger is slowly withdrawn while slowly turning in a clockwise direction. While the cement grout is still fluid, reinforcing steel is then inserted into the pile. The resulting grout column hardens and forms an ACD pile. Advantages of the ACD piles compared to the other foundation recommendations are listed below:

- **Less noise** – ACD piles and CIDH piles are drilled and pumped and not driven. This eliminates the hammer impact noise created by driven piles;
- **Minimizes vibrations** – Minimal vibrations are generated during construction that limits vibrations at adjacent structures, walls, and other structural components compared to larger vibrations that may occur from other methods such as pile driving;
- **Protects against caving during construction** – Due to the presence of shallow groundwater and collapsible sands, CIDH piles would require casing or slurry (referred to as 'wet' method) for construction, not required for driven and ACD piles; and
- **Minimizes soil cuttings** – CIDH piles generate large amounts of soil cuttings that require more export transportation off-site compared to driven and ACD piles.

The following section present preliminary deep foundation recommendations for ACD piles.

5.4.2.1 Auger-Cast-Displacement (ACD) Pile

ACD piles are recommended to support buildings with large column loads to control total and differential settlements. ACD pile diameters typically range from 12-inches to 24-inches. For planning purposes we have provided preliminary ultimate axial capacities for a 16-inch and 24-inch diameter ACD pile in Figure 8A and Figure 8B, respectively. Figures 8A and 8B present the preliminary ultimate tension (upward) capacity and two compression (downward) axial capacities. The compression axial capacities are presented for purely frictional piles and piles that gain capacities from friction along the pile and from the tip of the pile (end bearing). Generally end bearing is mobilized when ACD and driven piles are tipped in a dense sand. This will be achieved in soil zone 3 that typically has a very dense sand layer at least 20 feet thick and may be partially achieved in soil zone 2 that is interbedded. The depth and thickness of these layers should be investigated during final design at the proposed building footprints as there is some variability in the depth of these layers. Therefore, for planning purposes the skin friction piles can be utilized for preliminary pile lengths.

Allowable axial capacities should include a factor of safety for determination of the pile lengths. The ultimate capacities include in Figures 8A and 8B include no factor of safety. An allowable downward axial capacity should consider a factor of safety of 2. The allowable would be for dead-plus-live load capacity, where a one-third increase may be used for wind or seismic loads. The allowable upward axial capacity should consider a factor of safety of 3. Uplift due to wind or seismic loading may use a reduced factor of safety of 2. These capacities are based on the strength of the soils; the compressive and tensile strengths of the pile sections will need to be checked to verify the structural capacity of the piles.

For preliminary structural analyses, 16-inch-diameter ACD piles extending to 40 feet to 60 feet below ground should achieve an ultimate axial downward capacity on the order of 200 kip and 400 kips respectively (i.e., allowable of 100 to 200 kips). For planning purposes the downward capacity has been determined from skin friction. During final design piles sufficiently embedded (at least 1.5 diameter) in sand layers may have larger capacities due to well mobilized end bearing resistance as shown in Figures 8A and 8B. The sand layers in soil zone 2 were of variable thickness and not continuous across the project site. During final design, the sand layers in soil zone 2 will be further evaluated for continuity across a building's footprint for potential use of end bearing in the final foundations to decrease the pile lengths.

5.4.2.2 Driven Steel Pipe Pile

Driven steel pipe piles are feasible as a secondary option. Driven pile feasibility is highly dependent on acceptability of noise and vibration generation. Pipe piles could be driven with closed-end or open-ended. Open-ended pipe piles are better suited to penetrate the interbedded dense to very dense sands in soil zone 2 and very dense sands in soil zone 3 compared to closed-ended piles. For planning purposes 16-inch-diameter pipe piles can be assumed to be the same capacity as the ACD piles in Section 5.4.2.1.

Pile driving equipment will need to produce a sufficient amount of energy to install the piles to the required depths. A pile drivability analysis should be performed by a piling contractor that considered the proposed pile/hammer configuration and driving equipment.

5.5 Parking Structures

Parking structures considered for this study include a five-story short-span concrete parking structure considered for residential wrap-around parking and a long-span parking structure considered for a higher capacity centralized mixed-use parking that maybe constructed in the initial phase of development to support subsequent phase development.

5.5.1 Short-Span Parking

A short-span parking structure column loading is similar to that of a podium structure, as shown in Figure 7. Therefore, at-grade short-span parking structures can follow the foundation recommendations in Sections 5.3. Ground improvement as discussed in Section 5.6 should be considered for planning purposes to control settlements that are anticipated to be at least 2-inches. Parking structures typically can accommodate more settlement and there may be opportunity to decrease ground improvement quantities if more settlement is allowed by the structural engineer.

5.5.2 Long-Span Parking

Long-span parking structures have column loads that are larger than 1,000 kips and a deep foundation is recommended to control total and differential settlements. Therefore, long-span parking structures can follow the foundation recommendations in Sections 5.4.2.

5.6 Ground Improvement – Aggregate (Stone) Columns

Ground improvement has been recommended for several building types in other sections of this report in conjunction with a mat or post-tensioned slab foundation to control long-term total and differential settlements. Based on the preliminary subsurface profile the upper 25 to 30 feet of soil is predominantly lean clay and fat clay that is prone to long-term settlements and poor bearing capacity without proper mitigation. Ground improvement is recommended to extend from the bottom of footing through soil zone 1 (discussed in Section 3.3. and shown in Figures 5A and 5B).

Several methods can be considered for ground improvement such as deep soil mixing or grouting techniques; however, these may not be economically feasible at the project site. Aggregate (stone) columns are considered economically feasible for ground improvement of the project site and recommendations for other options can be provided upon request.

Aggregate (stone) columns construction involves the introduction of rock material into the native material by downhole vibratory or ramming methods. Stone column construction is often referenced as vibro-replacement or vibro-displacement that can be a top or bottom feed process to install stone columns to the targeted depths. Alternative to vibration methods include rammed aggregate piers (RAP) that are installed by drilling and ramming lifts of well-graded aggregate to form the high-density columns.

A qualified soil improvement contractor should be selected and provide design of the depth, spacing, and size of the zone of treatment based on the target foundation design parameters and their design requirements. Preliminary cost estimates have been provided by a specialty contractor to provide a rough order of magnitude for planning purposes. The aggregate columns are estimated to cost $12 per square foot of improvement for an at-grade structure and $8 per square foot for a building with one-subterranean level. The total depth of improvement is anticipated to be on the order of 25 to 30 feet deep for the at-grade structure with a reduction of the excavation for a below grade level. Mobilization cost for this technique are modest and division into individual phases will not result in a large cost difference as opposed to one individual phase.

Quality control procedures for installation and verification of material strengths will need to be developed and implemented in final design.

5.7 Basement Walls

Basement walls should be designed to resist at-rest earth pressures. Accordingly, for the case where the grade is level behind the walls, a triangular distribution of lateral earth pressure equivalent to that developed by a fluid with a density of 60 pounds per cubic foot. This earth pressure assumes that all walls are constructed with a properly designed drainage system to prevent buildup of hydrostatic pressures behind the wall. The walls should be designed to accommodate hydrostatic pressure based on the assumed historical high groundwater table (5 feet below the existing ground surface). Any surcharge loadings, such as stockpiled materials or traffic, should be added to the lateral pressure. The recommended pressure should also be confirmed during the design-level geotechnical investigation.

Basement walls should also be designed for seismic earth pressure. Assuming the basement wall is backfilled with compacted sands, the basement walls should be designed to resist, an active pressure combined with a seismic increment of lateral earth pressure. Seismic loading is based a horizontal coefficient (k_{eq}) of 0.23g, which is corresponding to one-half of the design peak ground acceleration (PGA_M) that is 0.55g. The active pressure combined with seismic increment of 60 pcf may be used for design of basement wall. If cohesive soils are not removed from behind the wall (about 1H:1V up from footing), higher earth pressure than the above will be exerted on the wall. The recommended value of earth pressure should be confirmed in the design geotechnical report.

5.8 Slabs-on-Grade

Concrete floor slabs and hardscape should be installed on a properly prepared subgrade and should be designed for the expansion potential of the supporting subgrade, as discussed in the following sections. To reduce the potential for moisture transmission through the floor slab, we recommend that a minimum 6-mm thick Visqueen moisture barrier be placed under the slab prior to the placement of concrete. The moisture barrier should be sandwiched between two layers of select sand, each with a minimum thickness of 2 inches. Care shall be taken not to puncture the moisture barrier during construction. Any utility stub-outs should be properly wrapped and sealed.

The local standard of practice for the design and construction of foundations, slabs and hardscape supported with a medium to high expansion potential is provided below. Structural design requirements may require greater thickness and/or more reinforcing than indicated, and should be evaluated by the structural engineer.

- Footings should be founded at least 24 inches below lowest adjacent grade.
- Footings should be reinforced with two #4 bars top and bottom.
- Floor slab should be at least 4 inches thick and should be reinforced with #3 bars at 18 inches on center, each way.

- Prior to placing concrete, the subgrade should be pre-saturated to 120 percent of the optimum to a depth of at least 18 inches below the bottom of the footing or slab.
- Concrete slabs and hardscape should have a maximum joint spacing of 10 feet; #3 bars dowels at construction joints; and the outside edge should be deepened to a thickness of 12 inches. One #3 bar should be used to reinforce the flared edge area.
- The adjacent area should be sloped at 2 percent or greater, to drain away from slabs and pavement.
- For additional protection, consideration should also be given to removing the upper 12 inches of expansive soil below the slab and replacing it with very low expansive sandy material having an EI of less than 20.
- Bushes, trees, and irrigation pipes and valves should be kept sufficiently away from the edges of foundations and walkways to prevent root damage and/or moisture changes in the supporting subgrade.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Groundwater Issues

Groundwater levels measured at the site were as high as about El. 23 to 17 feet (12 to 16 feet bgs) as measured during the recent field investigation and as presented in Appendix A. Excavations within a few feet of the measured groundwater elevations are anticipated to need stabilization. If wet or unstable subgrade is encountered, stabilization may consist of the placement of a granular working mat consisting of geogrid and coarse gravel or subexcavation and replacement with dried soil.

Due to clayey nature (low permeability) of the onsite soils, dewatering through dewatering well to lower groundwater table during construction may not be feasible. Sump area may be needed at the bottom of excavation to collect groundwater inflow and pumped to a storm drain. Groundwater should be evaluated to determine if treatment is required before transported to storm drain.

Groundwater levels can fluctuate due to seasonal rainfall amount, local irrigation and groundwater recharge programs and other man-made conditions. A temporary groundwater monitoring well has been installed at Boring B-1 and should be periodically monitored to evaluate seasonal variability.

6.2 Construction Phasing

Construction is proposed in phases allowing construction to move forward while keeping some existing businesses in operation. The conceptual construction phasing is shown in Figure 2B and several phases may be progressed simultaneously. The construction phasing should consider utility needs servicing the site and potential conflicts from subsequent excavations. A temporary excavation plan should be developed considering the staged construction and potential impact from or to already constructed buildings.

6.3 Adjacent Structure

The project is considering several building typologies including both at-grade and inclusion of subterranean levels. Permanent loads and construction loading (or unloading) on adjacent structures should be considered and evaluated as part of the final design. Project phasing will need to consider both existing structures and phased construction in temporary shoring and cutback approaches to excavations.

An existing Orange County Flood Control (OCFD) culvert and easement diagonally crosses the northeastern corner of the property near the existing Chase Bank Building. Final building layouts should avoid vertical and lateral loads on the existing culvert.

6.4 On-Site and Imported Fills

On-site soils in the upper 25 to 30 feet bgs are predominantly lean clays and fat clays. If the foundations are designed for expansive soils, on-site clayey soils, after clearing and grubbing and removal of deleterious materials, may be used for compacted fills. On-site soils will not be suitable for specific purposes where very low expansive granular fill is required.

Very low expansive imported borrow will most likely be used as replacement material below slabs and shallow foundations at the site. Very low expansive material should have an EI of less than 20. Additionally import borrow should have a maximum particle size of 6 inches, have less than 35% passing no. 200 sieve, and have a Plasticity Index (PI) of 12 or less. Prospective imported borrow materials should be tested at the borrow site to verify they are acceptable for the intended use prior to purchase and import. Any imported soil should also be evaluated for corrosion characteristics if they will be with buried or at grade structures and appropriate mitigative measure should be included.

6.5 Temporary Excavation and Shoring

Excavations for construction of subterranean levels are anticipated to be as deep as 14 feet below existing grade. Excavations can be readily accomplished with light to moderate effort using conventional heavy-duty grading equipment such as scrapers, loaders, dozers, and excavators. The contractor will be responsible for excavation safety, and all excavations should comply with the current California and Federal Occupational Safety and Health Administration (CALOSHA) requirements (29 CFR-Part 1926, Subpart P), as applicable. Temporary slopes, up to 20 feet high, may be cut at a gradient of 1.5H:1V (horizontal:vertical) with the bottom 4 feet is permitted to be cut vertically. Unshored excavations should not extend below a 1H:1V plane extending down from any improvements or foundations to be protected in place.

If sloping or benching is not practical due to space constraints, temporary shoring may be used. Vertical temporary excavations deeper than 5 feet should be shored. 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 excavation, unless the shoring is designed for surcharge loading. All shoring should comply with OSHA regulations and 29 CFR Part 1926 guidelines and be observed and deemed safe by the designated competent person on site. The designated competent person should observe all excavations to determine the safety prior to excavation.

6.6 Pile and Ground Improvement Load Testing

Auger cast piles and the aggregate piers will require load testing during construction. Pile lengths can be optimized by advancing a pilot test program before final design to compare the design axial capacities to measured values. If sufficient time is allowed between construction phases shown in Figure 2B, then there may be opportunity to incorporate load testing from a previous stage into future design at the site.

The static axial pile load testing program for ACD piles will generally consist of the following:

- Number of static load tests:

Total Production Piles	No. of Static Load Tests Required
<100	1
101-300	2
301-1000	3
1001-2000	4
2001-4000	5

- Minimum one (1) pile load test shall be performed per 30,000 square feet of building footprint;
- Gamma-Gamma Test and Low Strain Integrity Test shall be conducted on all test piles and reaction piles;
- Low Strain Integrity Test shall be performed on 10% of the production piles.

In addition to testing each pile to the ASTM 1143 standards, a creep test is recommended at the allowable load. The creep test holds the allowable load for at least two hours to demonstrate displacement of the test pile slows to less than 0.005 inch per hour, which is half the rate recommended in ASTM 1143.

6.0 Additional Investigations for Final Design and Construction

The current scope of work identified the general characteristics of the subsurface soils and identified shallow expansive soils, static and seismic settlement, and relatively shallow groundwater as potential issues for the proposed development. Design level geotechnical investigations should be planned when building types and configurations are determined. The design level investigation should include installation of monitoring wells, borings and CPTs to further characterize the subsurface.

During construction phase, the scope of geotechnical testing and inspections will depend on foundation type. For planning purposes, for shallow foundations, geotechnical observation and testing of grading operations will be required. For deep foundations, geotechnical observation of pile installation, installation of test piles and furnishing of pile load test results will be required.

7.0 LIMITATIONS

This investigation was performed in accordance with generally accepted Geotechnical Engineering principles and practice. The professional engineering work and judgments presented in this report meet the standard of care of our profession at this time. No other warranty, expressed or implied, is made. This report has been prepared for Related California Residential, LLC and their design consultants. It may not contain sufficient information for other parties or other purposes, and should not be used for other projects or other purposes without review and approval by Group Delta.

The recommendations for this project, to a high degree, are dependent upon proper quality control of site grading, fill and backfill placement, and pile foundation installation. The recommendations are made contingent on the opportunity for Group Delta to provide final geotechnical recommendations and observe the earthwork operations. This firm should be notified of any pertinent changes in the project, or if conditions are encountered in the field, which differ from those described herein. If parties other than Group Delta are engaged to provide such services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project, and must either concur with the recommendations in this report or provide alternate recommendations.

8.0 REFERENCES

California Department of Conservation, Division of Mines and Geology, 1990, "Fault-Rupture Hazard Zones in California, Alquist-Priolo Special Studies Zones Act of 1972," Special Publication 42, Department of Conservation, California Division of Mines and Geology.

California Building Code (CBC), 2019, published by International Conference of Building Officials, Whittier, California.

California Geological Survey, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," Special Publication 117A, dated 2008.

California Geological Survey (CGS, 1997). Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-minute Quadrangle, Orange County, California. Seismic Hazard Zone Report 03.

SEAOC/OSHPD Seismic Design Maps, https://seismicmaps.org/, accessed February 28, 2019.

Youd, T. L., et al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, October 2001.

FIGURES

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NOT TO SCALE

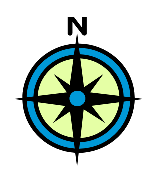
GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	FIGURE NO.: 1
PROJECT NAME: Related Bristol, Santa Ana, CA	PROJECT NO.: IR737
SITE LOCATION MAP	

- Residential: 3,750 units
- Senior Care: 200 units
- Retail: 350k SF
- Hotel: 250 rooms

NORTH PHASE

SOUTH PHASE

Note: The initial concept is provided for visualization and discussion. The proposed building geometries and types are subject to change.



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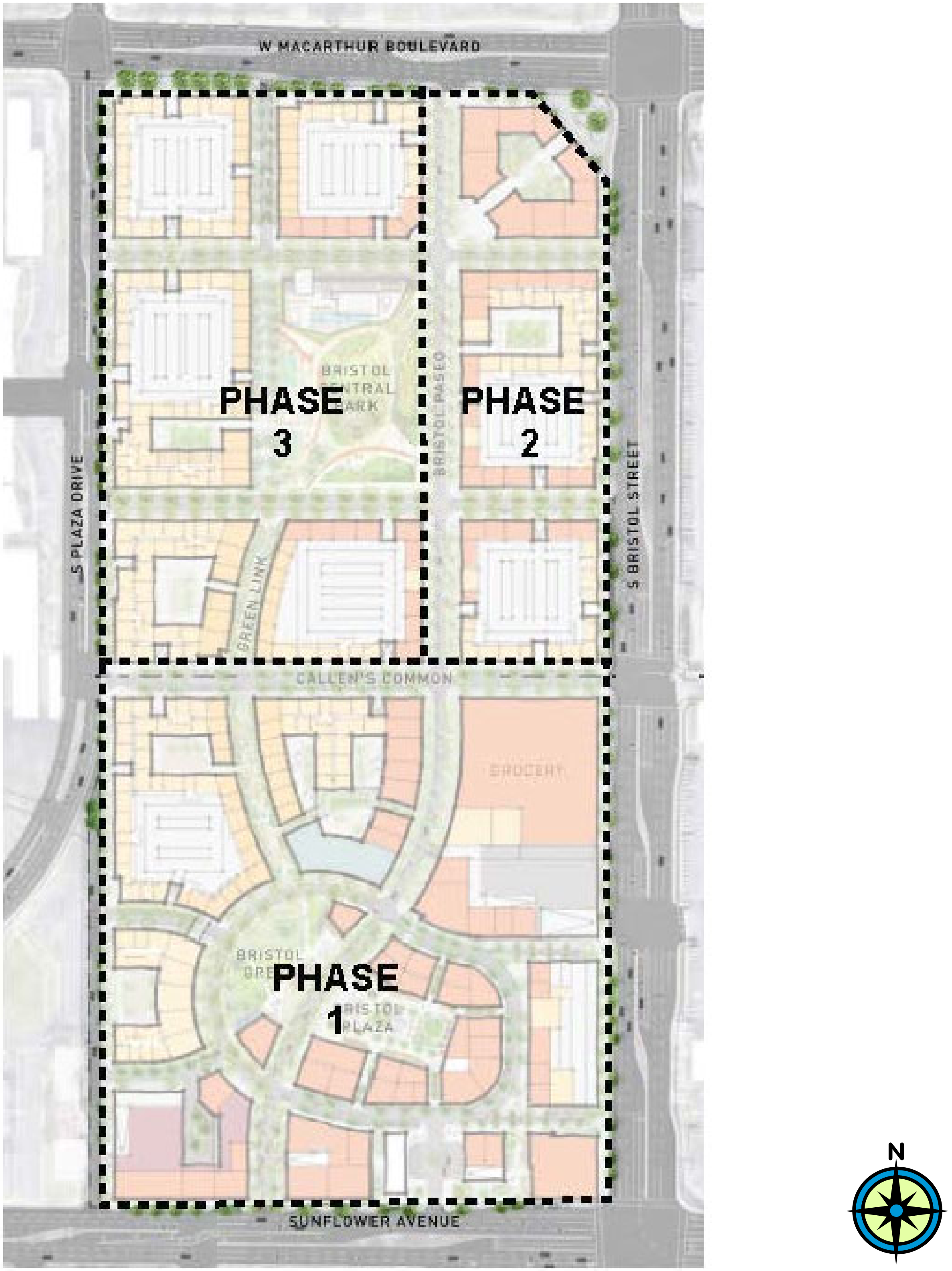
LEGEND
RESIDENTIAL
SENIOR CONTINUUM CARE
HOTEL
RETAIL

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	FIGURE NO.: **2A**
	PROJECT NAME: Related Bristol, Santa Ana, CA	PROJECT NO.: IR737
	CONCEPTUAL MASTER PLAN	



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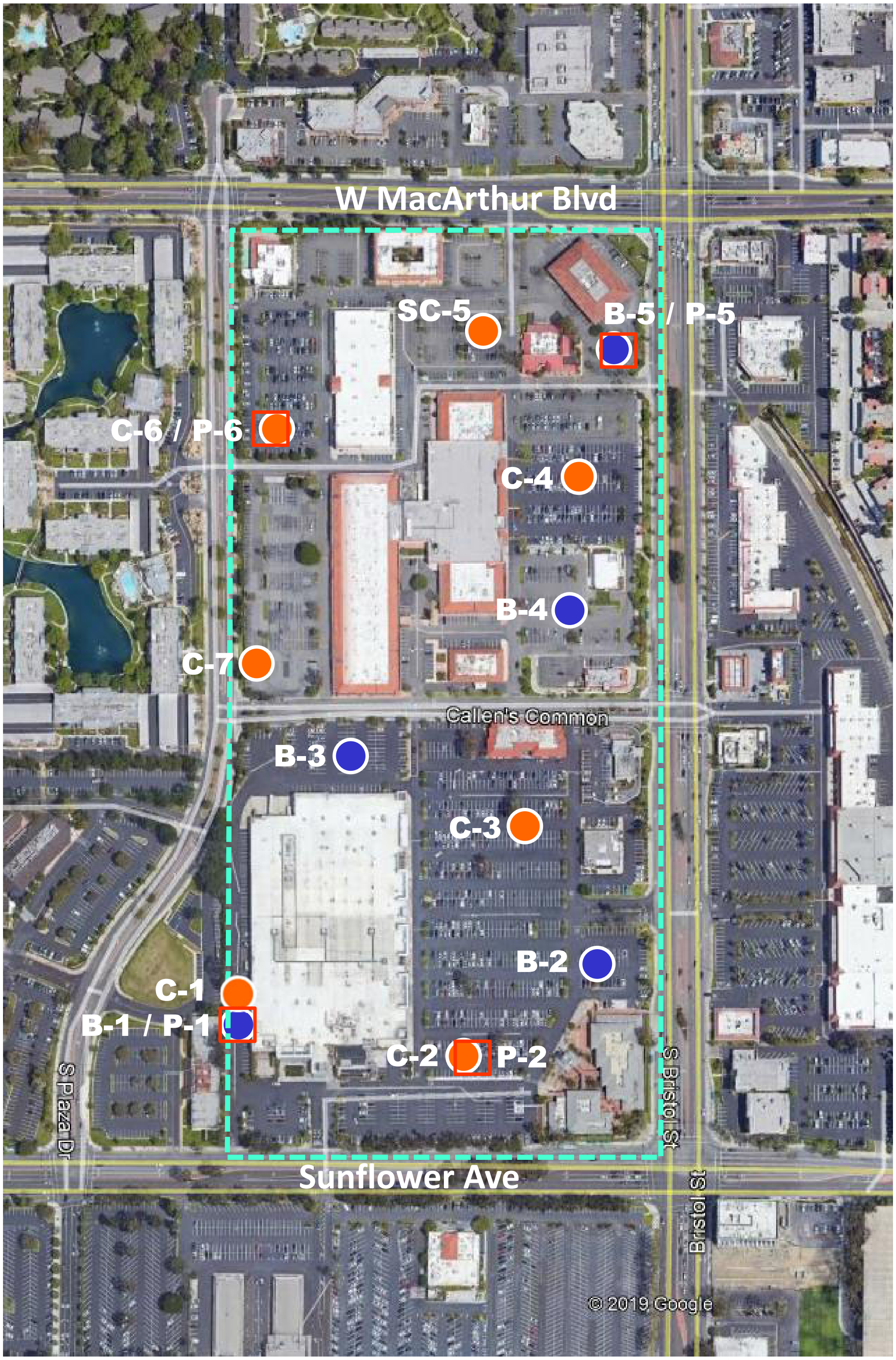
PROJECT NAME: Related Bristol, Santa Ana, CA	FIGURE NO.: **2B**
CONCEPTUAL PHASING PLAN	PROJECT NO.: IR737



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Approximate Site Boundary

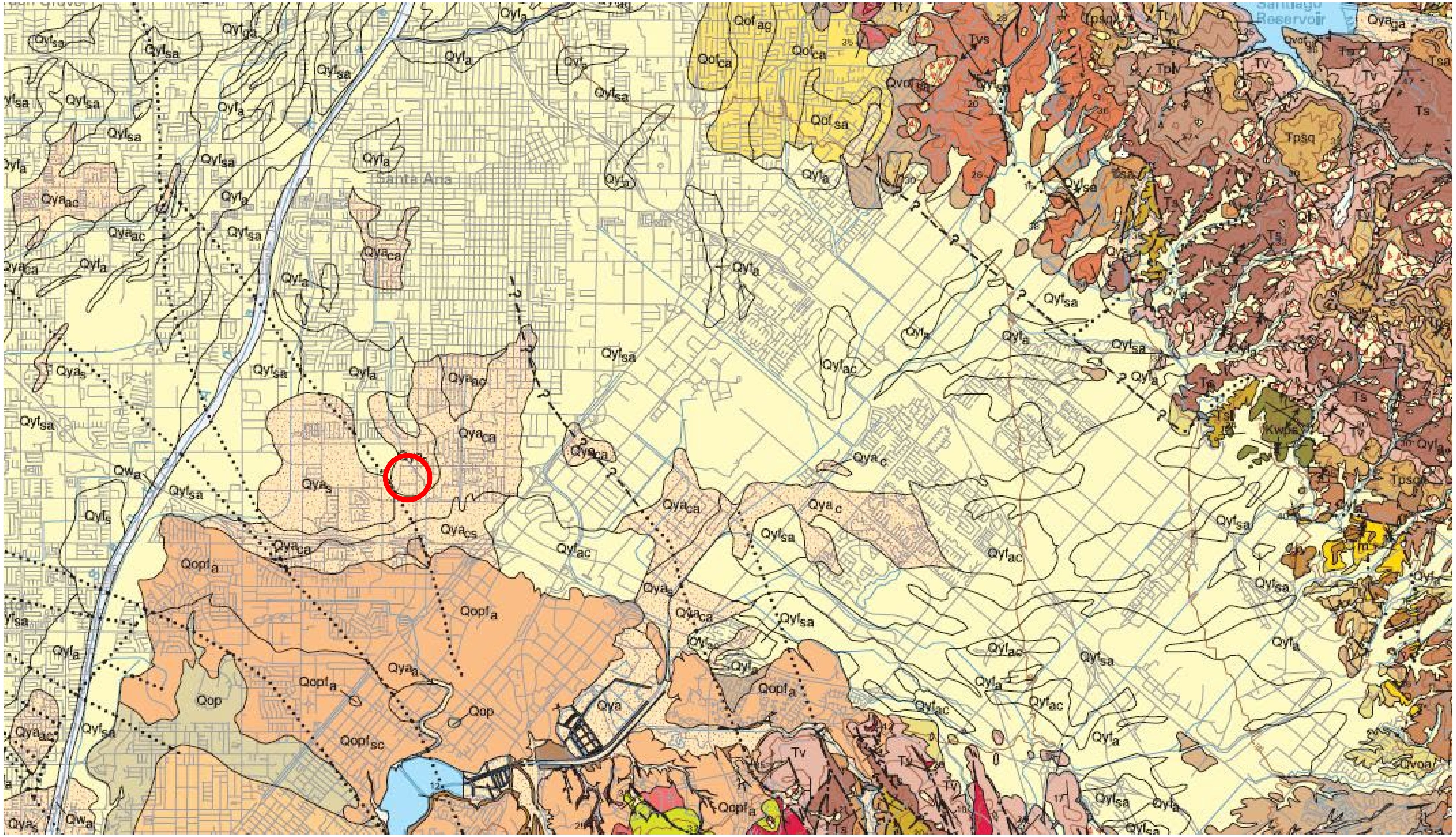
Boring Locations

Cone Penetration Test Locations

Percolation Test Locations

Note: Explorations south of Callen's Common St and percolations are planned and have not been advanced.

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	FIGURE NO.: 3
	PROJECT NAME: Related Bristol, Santa Ana, CA	PROJECT NO.: IR737
	EXPLORATION LOCATION PLAN	



Reference: USGS 30'x60' Santa Ana Quadrangle, see for additional details on units

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Symbol	Unit
Qw	**Wash Deposits (late Holocene)**
Qyf	**Young alluvial fan deposits (Holocene to latest Pleistocene)**
Qya	**Young axial channel deposits (Holocene to latest Pleistocene)**
Qop	**Old paralic deposits (late to middle Pleistocene)**
Qopf	**Old paralic overlay by alluvial fan deposits (late to middle Pleistocene)**
Ttlt	**Los Trancos Member (Siltstone/Sandstone)**
Tv	**Vaqueros Formation (Sandstone Conglomerate)**
Ts	**Sespe Formation (Siltstone/Sandstone)**

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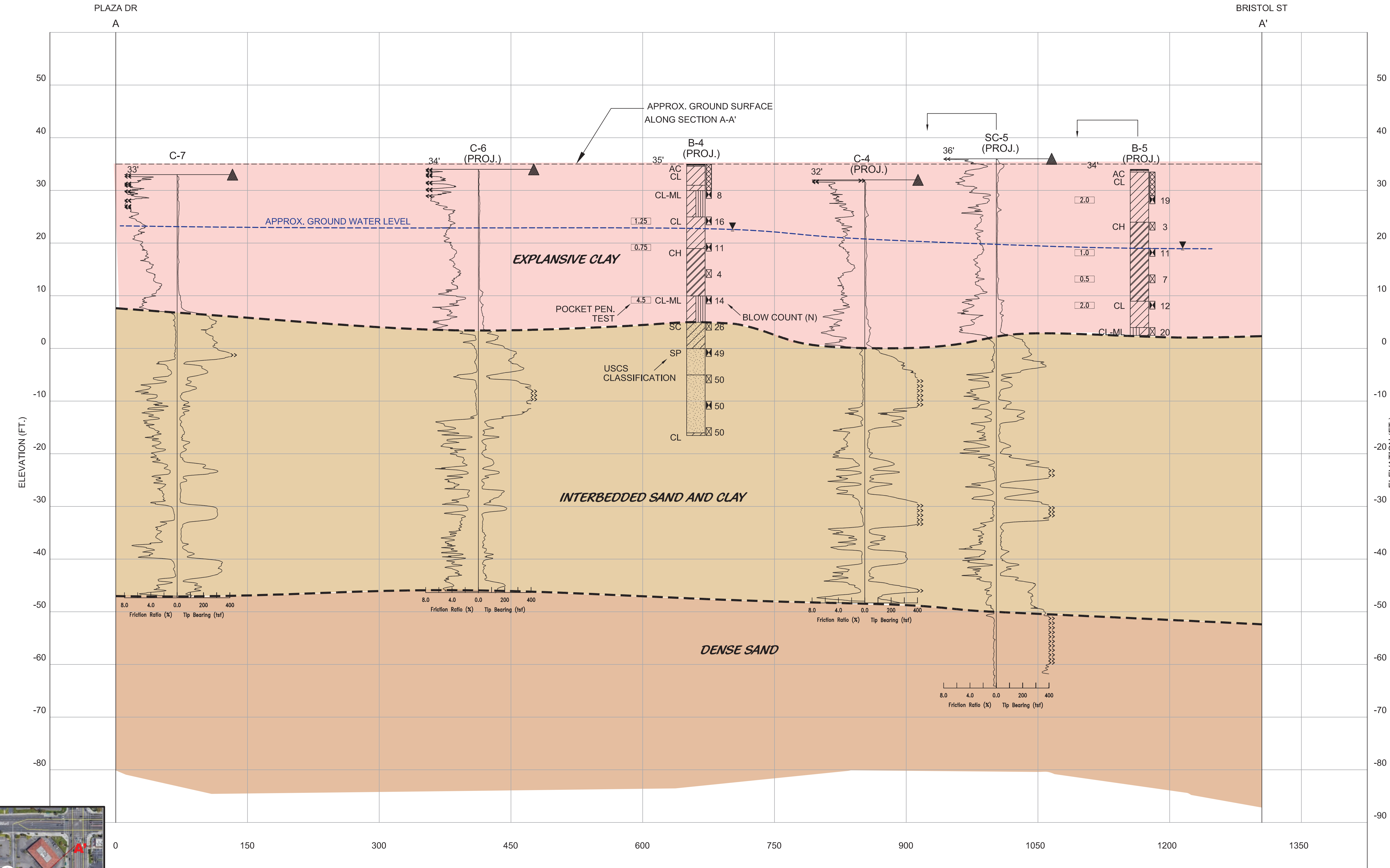
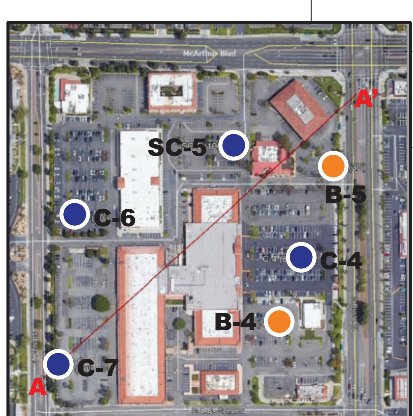
FIGURE NO.:
4

PROJECT NAME:
Related Bristol,
Santa Ana, CA

PROJECT NO.:
IR737

GEOLOGIC MAP





Notes:

Stratigraphic boundaries are shown here to illustrate generic soil types for use in preliminary design. Conditions encountered during construction will vary from those represented on the figure. Additional geotechnical data collection is recommended at final design.

GROUP DELTA CONSULTANTS, INC.
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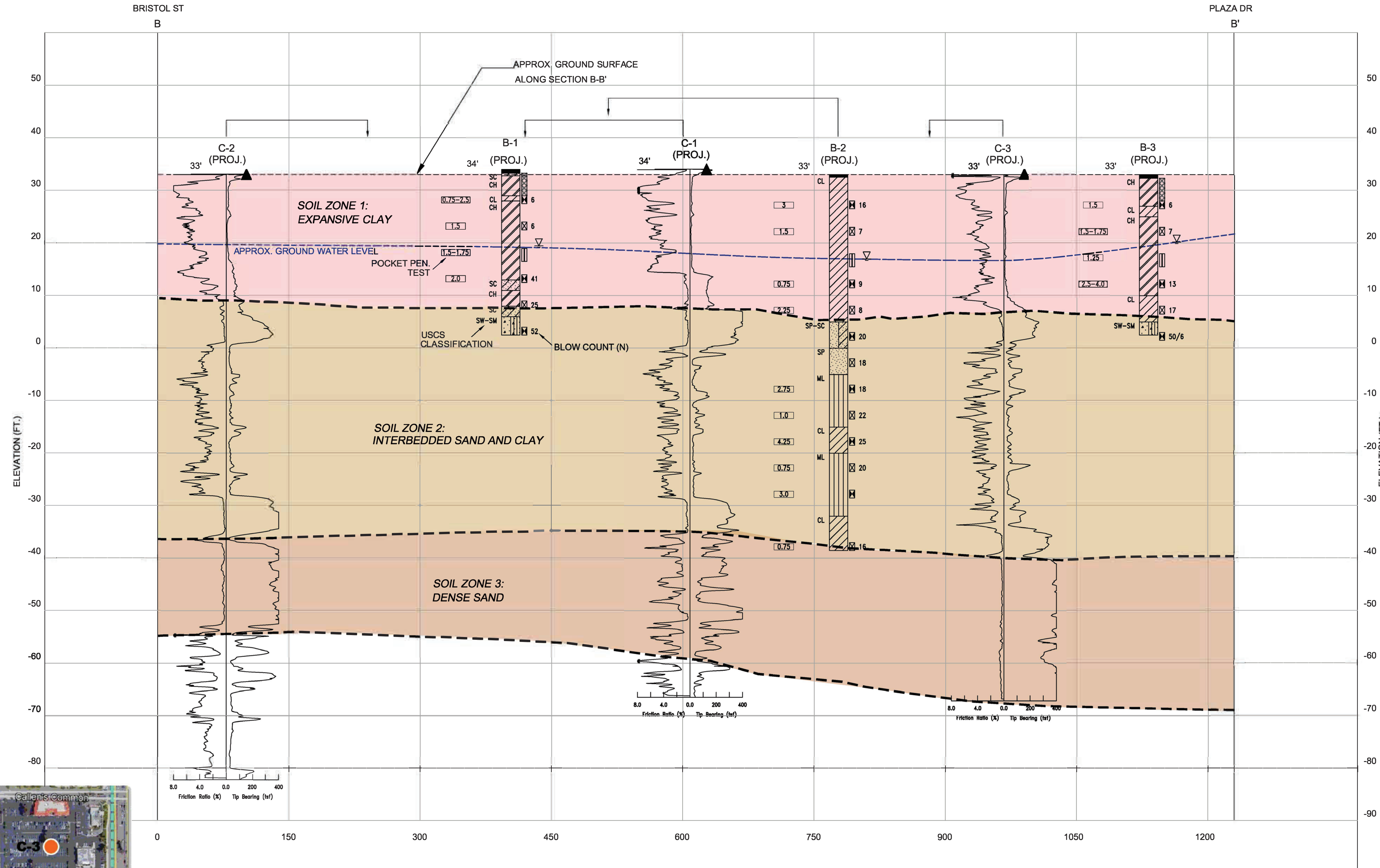
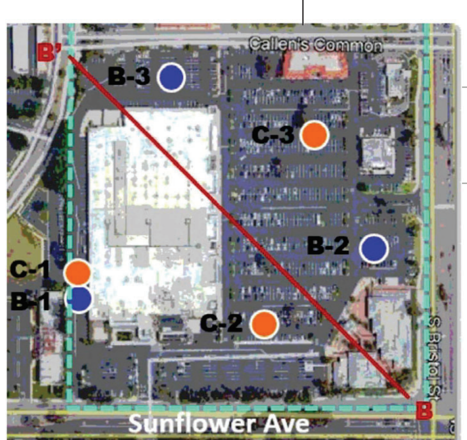
FIGURE NUMBER: 5A

PROJECT NAME: Related Bristol, SANTA ANA, CALIFORNIA

PROJECT NUMBER: IR-737

CROSS SECTION A-A'





Notes:

Stratigraphic boundaries are shown here to illustrate generic soil types for use in preliminary design. Conditions encountered during construction will vary from those represented on the figure. Note CPT-1 soil zone 3 indicates interbedded very dense sands with clay layers. Additional geotechnical data collection is recommended at final design.

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	FIGURE NUMBER 5B
	PROJECT NAME Related Bristol, SANTA ANA, CALIFORNIA	PROJECT NUMBER IR-737
	CROSS SECTION B-B'	

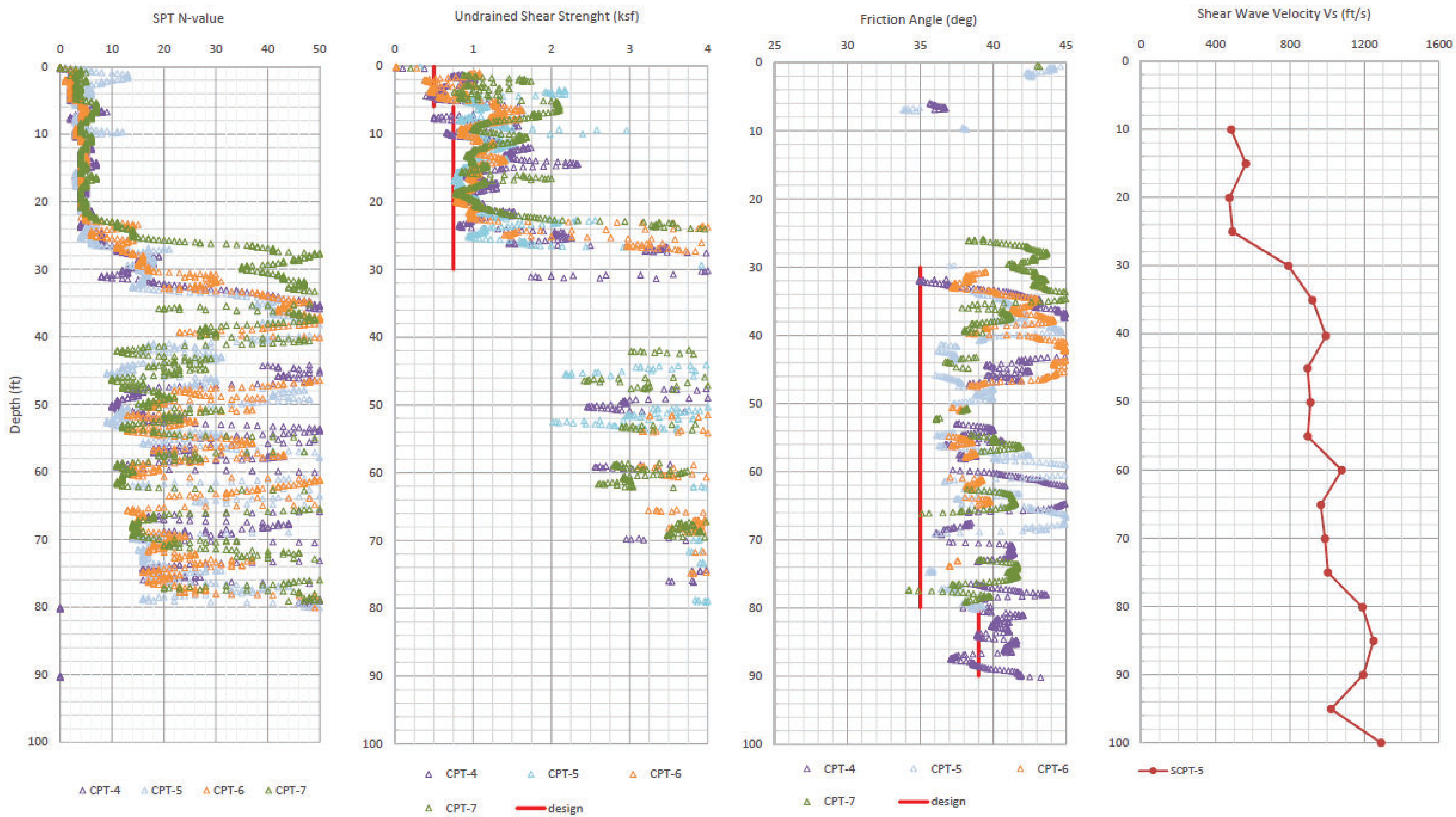


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FIGURE NUMBER	5C
PROJECT NAME	Related Bristol, SANTA ANA, CALIFORNIA
PROJECT NUMBER	IR 737

Subsurface Characterization A-A'

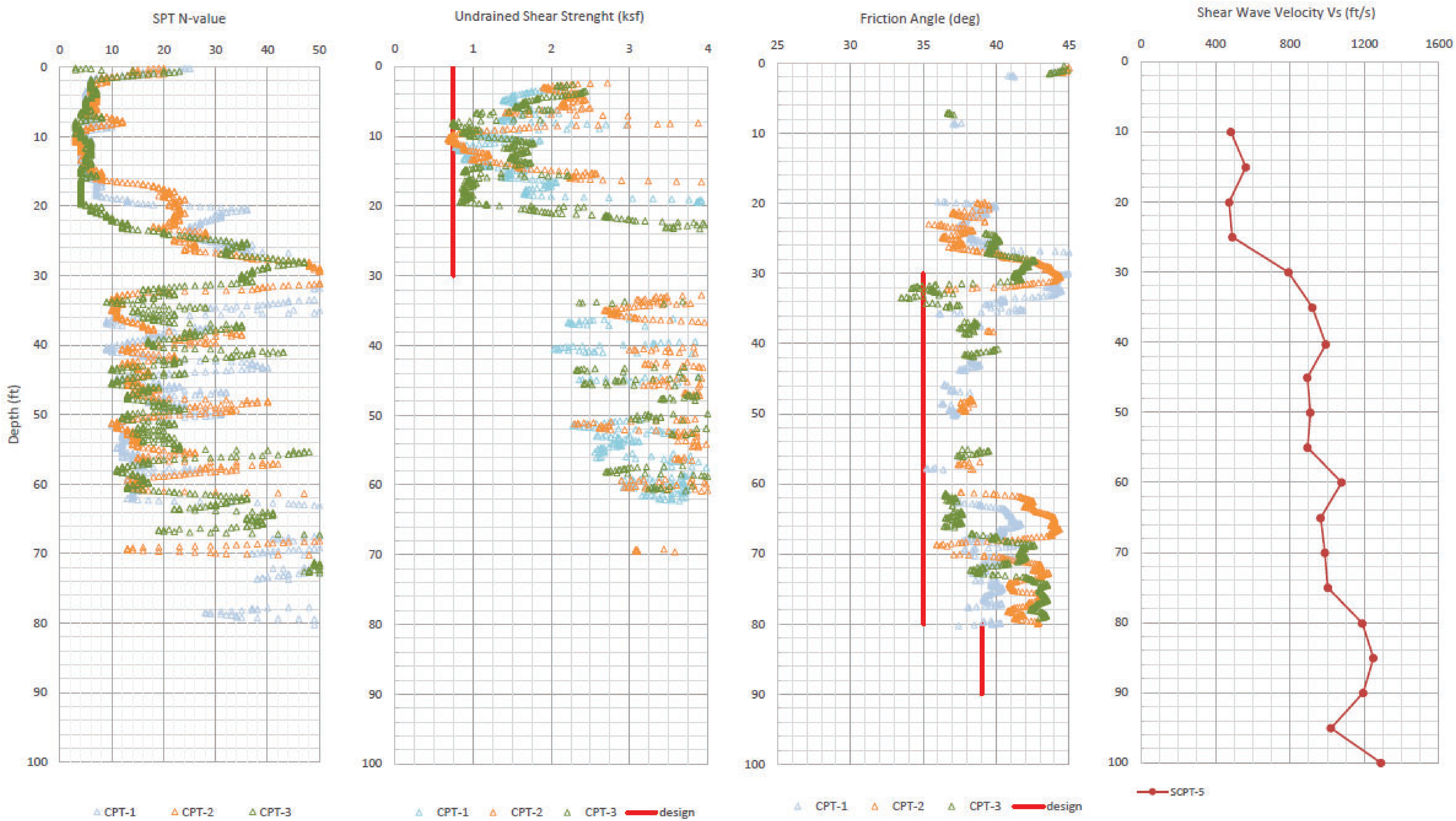

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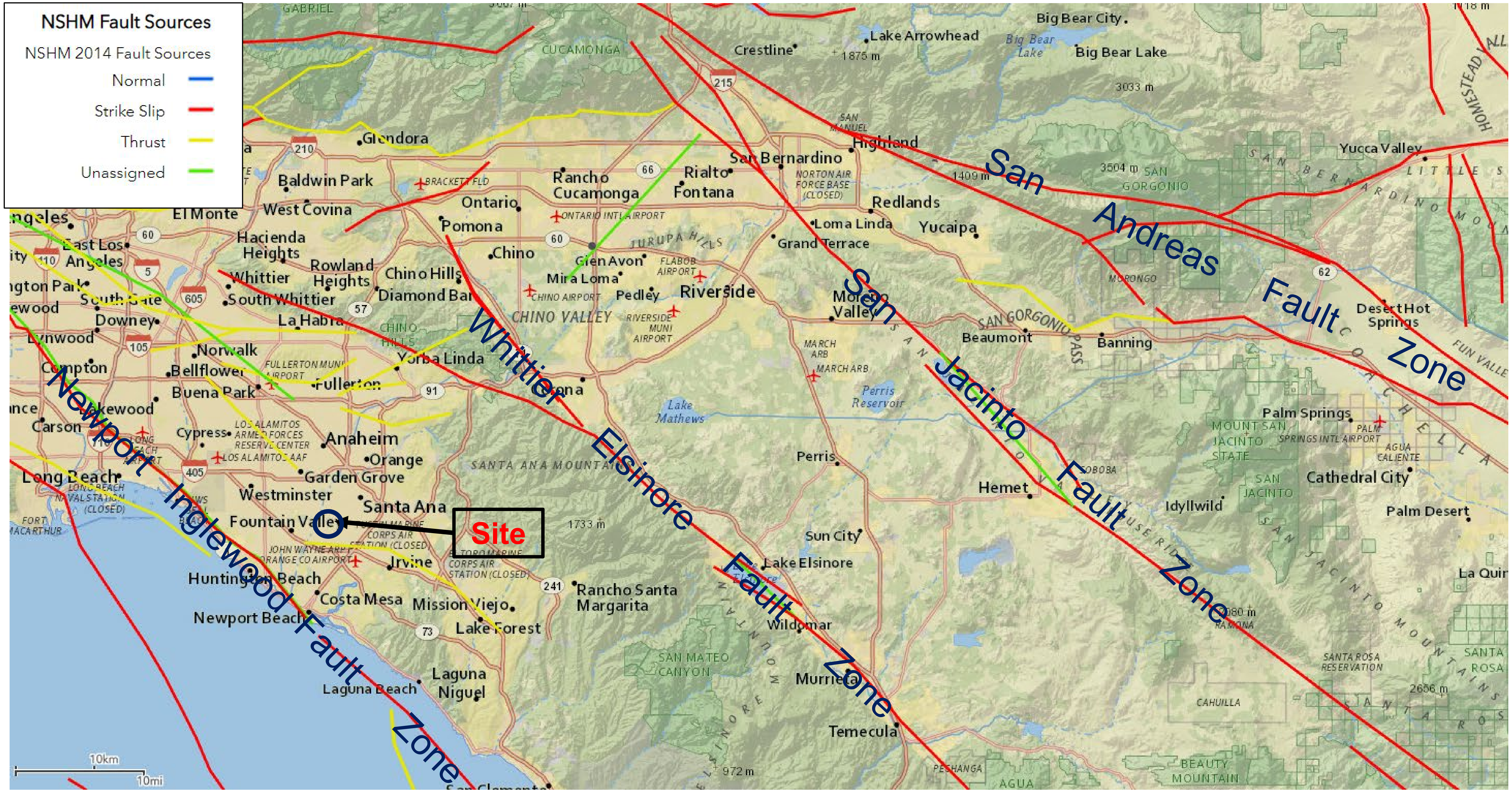
FIGURE NUMBER	PROJECT NAME	PROJECT NUMBER
5D	Related Bristol, SANTA ANA, CALIFORNIA	IR737

Subsurface Characterization B-B'



Reference: https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf

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FIGURE NO.: 6	PROJECT NO.: IR737
PROJECT NAME: Related Bristol, Santa Ana, CA	

FAULT AND SEISMICITY MAP

Applicable Report Section [1]	Details Provided by DCI Engineers on 07/14/2022								Details Provivded by Group Delta
	# Stories	Building Type	Typical Uniform Load			Typical Column Load			Foundation Types
			D (psf)	L (psf)	D+L (psf)	D (kips)	L (kips)	D+L (kips)	
5.2	5	Wood Framed On-Grade	190	160	350	N/A	N/A	N/A	· Shallow foundation on 4 ft of imported non-expansive material; or · Mat or post-tensioned slab; or · Deep foundations (i.e., auger cast piles);
	6	5-Story Wood Framed + Basement (Residential + Retail at Base)	230	200	430	210	180	390	
5.3	7	5-Story Wood Framed over 1-Level PT Concrete Podium + Basement (Residential)	320	200	520	290	180	470	· Mat slab (if settlements are acceptable); or · Post-tensioned slab with ground improvement; or · Deep foundations (i..e, auger cast piles);
	8	4-Story Wood Framed over 3-Level PT Concrete Podium + Basement (Residential)	550	240	790	500	220	720	
	9	5-Story Wood Framed over 3-Level PT Concrete Podium + Basement (Residential)	590	280	870	530	250	780	
5.4.1	4	PT Concrete Residential + 2-Level Basement (Retail at Base)	420	120	540	380	110	490	· Mat slab (if settlements are acceptable); or · Post-tensioned slab with ground improvement; or · Deep foundations (i..e, auger cast piles);
	4	PT Concrete Office Building + Basement	450	150	600	410	140	550	
	6	PT Concrete Residential + 2-Level Basement (Retail at Base)	630	200	830	570	180	750	
5.4.2	8	PT Concrete Residential + Basement	860	280	1,140	770	250	1,020	· Deep foundations (i..e, auger cast piles);
	9	PT Concrete Residential + Basement	970	320	1,290	870	290	1,160	
	9	PT Concrete Residential + Basement (Retail at Base)	970	320	1,290	870	290	1,160	
	17	PT Concrete Hotel Tower + Basement	1,930	640	2,570	1,740	580	2,320	
	24	PT Concrete Residential Tower + Basement (Atria)	2,720	920	3,640	2,450	830	3,280	
5.5.1	5	PT Concrete Short-Span Parking Structure On-Grade	510	200	710	460	180	640	· Mat or post-tensioned slab with ground improvement; or · Deep foundations (i..e, auger cast piles);
5.5.2	6	PT Concrete Free-Standing Long-Span Parking Structure On-Grade	670	240	910	800	290	1,090	· Deep foundations (i..e, auger cast piles);

Note:
1. Refer to corresponding report section for details
2. Foundation mass not included in loads.
3. Slabs on grade, mat slabs and/or post-tensioned slabs should be designed for expansive forces or at least 4 feet of removal and recompaction with non-expansive (import) material will be required.

DCI ENGINEERS

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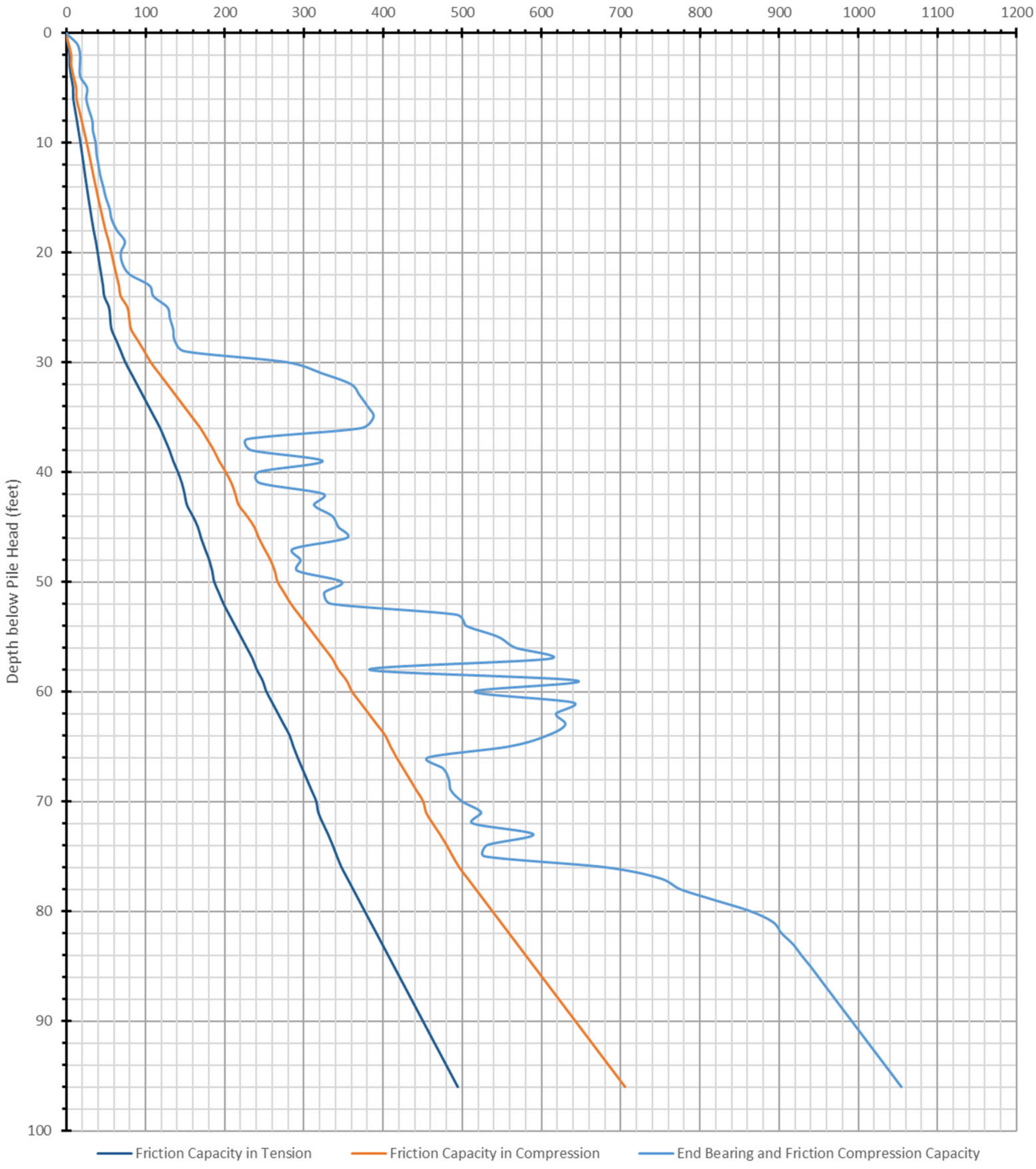
FIGURE NO.: 7

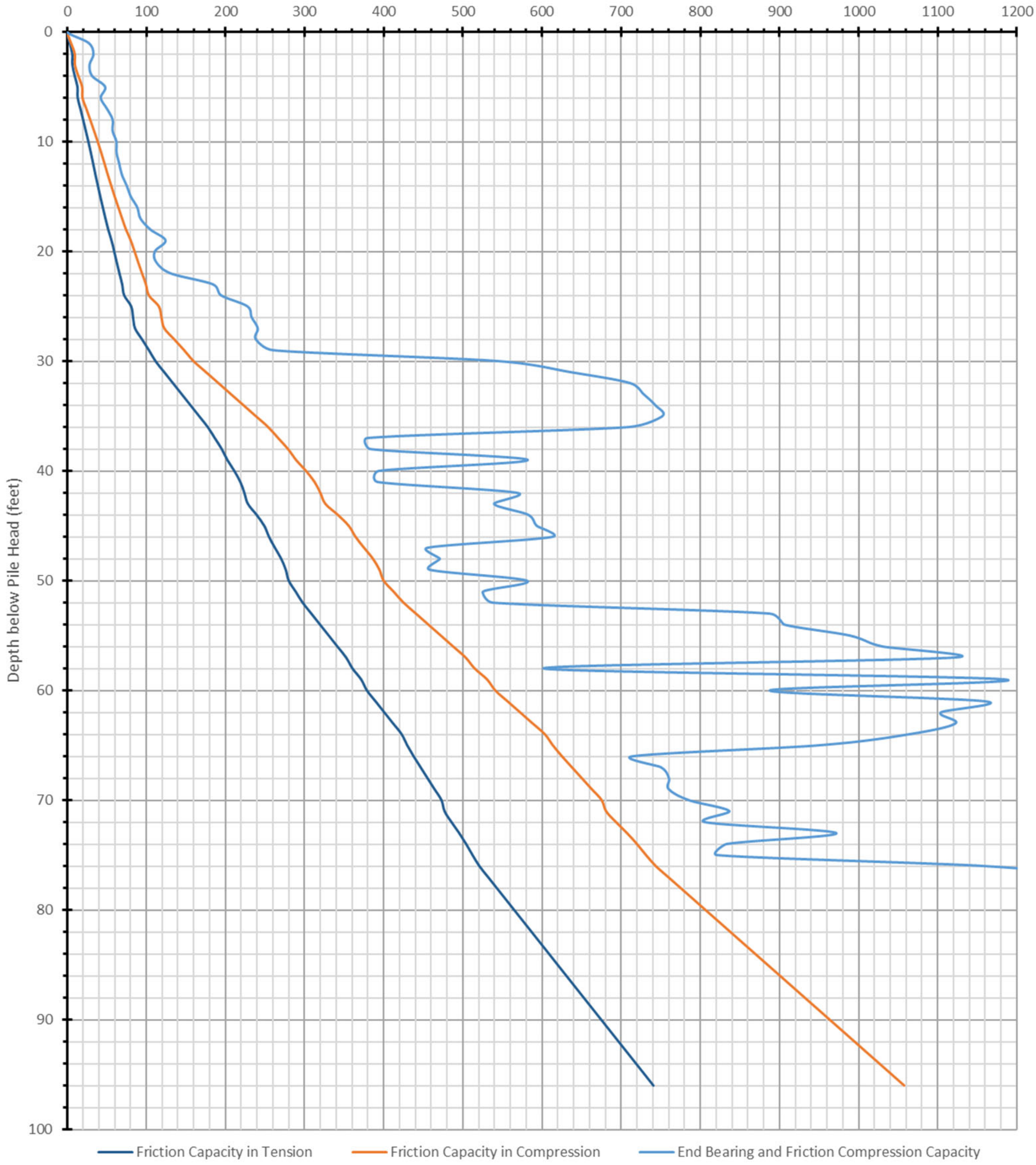
PROJECT NAME: Related Bristol, Santa Ana, CA

PROJECT NO.: IR737

Preliminary Structural Loading for Building Typology



GROUP DELTA	**GROUP DELTA CONSULTANTS, INC.** ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	FIGURE NO.: **8A**
	PROPOSAL NAME: Related Bristol , Santa Ana, CA	PROPOSAL NO.: IR737
	Preliminary Ultimate Axial Capacity 16-Inch Diameter ACD Piles	



GROUP DELTA	**GROUP DELTA CONSULTANTS, INC.** ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	FIGURE NO.: **8B**
	PROPOSAL NAME: Related Bristol , Santa Ana, CA	PROPOSAL NO.: IR737
	Preliminary Ultimate Axial Capacity **24-Inch Diameter ACD Piles**	

APPENDIX A
FIELD INVESTIGATION

APPENDIX A
FIELD INVESTIGATION

A.1 Introduction

The subsurface conditions at the Related Bristol project site were investigated by performing five hollow stem borings, and seven Cone Penetration Tests (CPTs) in the periods on February 14, 2020, and January 4 and 5, 2021. The locations of the explorations are presented in Figure 3 of the main report.

Prior to beginning the exploration program, access permission and drilling permits were obtained as necessary from Orange County Environmental Health Agency, and the property tenants and owners. Subsurface utility maps were reviewed prior to selecting locations for subsurface investigations. Underground Service Alert (USA) was notified and each exploration location was cleared for underground utilities. Approved traffic control plans were implemented where necessary during field activities. The exploration methods are described in the following sections.

A.2 Soil Drilling and Sampling

Drilling, Logging, and Soil Classification

Borings were performed by GDC's drilling subcontractors ABC Liovin Drilling, Inc. and Martini Drilling Corporation under the continuous technical supervision of a GDC field engineer, who visually inspected the soil samples, measured groundwater levels, maintained detailed records of the borings, and visually / manually classified the soils in accordance with the ASTM D 2488 and the Unified Soil Classification System (USCS). Logging and classification were performed in general accordance with Caltrans "Soil and Rock Logging, Classification, and Presentation Manual (2010 Edition)". A Boring Record Legend and Key for Soil Classification are presented in Figures A-1A through A-1E. The boring records are presented in Figures A-2a through Figure A-10.

Sampling

Bulk samples of soil cuttings were collected at selected depths and drive samples were collected at a typical interval of 5 feet from the borings. The sampling was performed using Standard Penetration Test (SPT) samplers in accordance with ASTM D 1586, Ring-Lined "California" Split Barrel samplers in accordance with ASTM D 3550 and Thin-walled (Shelby) Tube in accordance with ASTM D 1587.

Bulk samples were collected from auger cuttings and placed in plastic bags.

SPT drive samples were obtained using a 2-inch outside diameter and 1.375-inch inside diameter split-spoon sampler without lining. The soil recovered from the SPT sampling was sealed in plastic bags to preserve the natural moisture content.

California drive samples were collected with a 3-inch outside diameter 2.5-inch inside diameter split barrel sampler with a 2.42-inch inside diameter cutting shoe. The sampler barrel is lined with 18-inches of metal rings for sample collection and has an additional length of waste barrel. Stainless steel or brass liner rings for sample collection are 1-inch high, 2.42-inch inside diameter, and 2.5-inch outside diameter. California samples were removed from the sampler, retained in the metal rings and placed in sealed plastic canisters to prevent loss of moisture.

Shelby tube samples were obtained by pressing a 3-inch outside diameter 2.87-inch inside diameter thin-walled metal tube 30 inches into the in-situ soil at the bottom of a boring. The soil-filled tube was removed and applied seals to the soil surfaces to prevent soil movement and moisture gain or loss.

At each sampling interval, the drive samplers were fitted onto sampling rod, lowered to the bottom of the boring, and driven 18 inches or to refusal (50 blows per 6 inches) with a 140-lb hammer free-falling a height of 30-inches using an automatic hammer for SPT and California drive samples, and pushed 30 inches or to refusal with the drilling rig for Shelby tube samples.

A relatively intact sample is obtained by Shelby tube. Compared to the SPT, the California sampler provides less disturbed samples.

Penetration Resistance

SPT blow counts adjusted to 60% hammer efficiency (N_{60}) are routinely used as an index of the relative density of coarse grained soils, and are sometimes used (but less reliable) to estimate consistency of cohesive soils. For samples collected using non-SPT samplers, different hammer weight and drop height, and/or efficiency different than 60%, correction factors can be applied to estimate the equivalent SPT N_{60} value following the approach of Burmister (1948) as follows:

$$N^*_{60} = N_R * C_E * C_H * C_S$$

where

N^*_{60} = equivalent SPT N_{60}

N_R = Raw Field Blowcount (blows per foot)

C_E = Hammer Efficiency Correction = Er_i / 60%

C_H = Hammer Energy Correction = (W * H) / (140 lb * 30 in)

C_S = Sampler Size Correction = $[(2.0\text{ in})^2-(1.375\text{ in})^2]/[D_o^2-D_i^2]$

Er_i = hammer efficiency, %

W= actual drive hammer weight, lbs

H = actual drive hammer drop, inch

D_o, D_i = actual sampler outside and inside diameter, respectively, inches

Burmister's correction assumes that penetration resistance (blowcount) is inversely proportional to the hammer energy. For a hammer other than a 140# hammer with 30" drop the hammer energy correction is equal to the ratio of the theoretical hammer energy (weight times drop) to the theoretical SPT hammer energy, or $C_H = (W * H) / (140 \text{ lb} * 30 \text{ in})$.

Burmister's correction assumes that penetration resistance (blowcount) is proportional to the annular end area of the drive sampler. For California drive samplers with D_o=3 inch and D_i=2.42 inch the sampler size correction factor is the ratio of the annular area of an SPT split spoon to that of the California Sampler, or $C_S = [2.0^2 - 1.375^2]/[3^2 - 2.42^2] = 0.67$.

To normalize the field SPT and California blowcounts to a hammer with 60% efficiency, an energy correction factor equal to Hammer Efficiency (%) / 60% was applied to the field blowcounts. Hammer efficiency was determined by Pile Driving Analyzer (PDA) measurement. Hammer efficiency measurements are presented in this Appendix.

The correction factors applied to obtain N^*_{60} are summarized in the following table:

Hammer Type	**Hammer Weight and Drop**	**C_H**	**Hammer Efficiency (%)**	**C_E**	**Cal Sampler Dimensions**	**C_S**	**Combined Correction Factor SPT Samples**	**Combined Correction Factor CAL Samples**
CME 85 ABC Drilling	140# 30"	1	62.6	1.04	D_o=3.0" D_i=2.42"	0.67	1.04	0.70
CME 75 Martini Drilling	140# 30"	1	79.3	1.32	D_o=3.0" D_i=2.42"	0.67	1.32	0.89

Corrected N^*_{60} are generally used, with due engineering judgment, only for qualitative assessment of in place density or consistency, and are not used for other more critical analyses such as liquefaction.

Relative Density and Consistency

Equivalent SPT N_{60} values were used as the basis for classifying relative density of granular/cohesionless soils. Wherever possible consistency classification of cohesive soils was based on undrained shear strength estimated in the field with a pocket penetrometer or by testing in the laboratory. Where pocket penetrometer or other tests could not be performed, consistency of cohesive soils was estimated by correlations to Equivalent SPT N_{60}. The correlations for consistency and relative density are shown in the Boring Record Legend, Figures A-1A through A-1C. Drive sample field blow counts, SPT N^*_{60} values, pocket penetrometer readings, and corresponding density/consistency classifications are presented on the boring records.

Borehole Abandonment

At the completion of the drilling groundwater was measured (where possible) and the borings were abandoned by backfilling the borehole with Bentonite grout or by transferring the borehole into a temporary well, as indicated on the records. Excess cuttings and drilling fluids were placed in 55 gallon drums, sampled and tested for contaminants, temporarily stored at an approved location, and legally disposed of off-site. The surface was patched with cold mix asphalt concrete or quickset concrete, as necessary. Notes describing the borehole abandonment are presented at the bottom of each boring record.

Sample Handling and Transport

Geotechnical samples were sealed to prevent moisture loss, packed in appropriate protective containers, and transported to the geotechnical laboratory for further examination and geotechnical testing.

Laboratory Testing

The soils were further examined and tested in the laboratory and classified in accordance with the Unified Soil Classification System following ASTM D 2487 and D 2488 (see Figures A-1D and A-1E). Field classifications presented on the records were modified where necessary on the basis of the laboratory test results. Descriptions of the laboratory tests performed and a summary of the results are presented in Appendix B.

A.3 Cone Penetration Tests

CPT Soundings

Kehoe Engineering & Testing performed the CPT soundings as a subcontractor to GDC. The CPTs were conducted in accordance with ASTM D 5778 using an electronic piezocone penetrometer. The test consists of hydraulically pushing a conical pointed penetrometer with a cylindrical friction sleeve and a piezo-element located behind the conical point into subsurface soils at a slow, steady rate. Parameters electronically measured and recorded nearly continuously during the CPT are soil bearing resistance at the cone tip (q_c), soil frictional resistance along the cylindrical friction sleeve (fs), and pore water pressure directly behind the cone tip (U). These measured values are then used to estimate the type and engineering properties of soils being penetrated using published correlations between q_c, fs, and U.

The CPT data in graphical form and accompanying data interpretation by GDC are presented in this Appendix. At the completion of the sounding the apparent groundwater depth and cave-in depth was measured with weighted tape and the CPT hole was abandoned by backfilling bentonite into the hole. Paved surfaces were patched with cold mix asphalt or quickset concrete, as necessary.

Seismic CPT Shear Wave Velocity Measurement

Shear wave velocity measurements versus depth were made in selected CPTs. After each 5 ft of penetration the probe was stopped, a shear wave was generated at the ground surface, and the arrival of the shear wave was detected by the CPT probe. The arrival times of the shear waves were used to calculate the shear wave velocity versus depth. The shear wave velocity data are presented in this Appendix.

SOIL IDENTIFICATION AND DESCRIPTION SEQUENCE

Sequence		Refer to Section: Field	Refer to Section: Lab	Required	Optional
1	Group Name	2.5.2	3.2.2	●	
2	Group Symbol	2.5.2	3.2.2	●	
	Description Components				
3	Consistency of Cohesive Soil	2.5.3	3.2.3	●	
4	Apparent Density of Cohesionless Soil	2.5.4		●	
5	Color	2.5.5		●	
6	Moisture	2.5.6		●	
7	Percent or Proportion of Soil	2.5.7	3.2.4	●	◍
	Particle Size	2.5.8	2.5.8	●	◍
	Particle Angularity	2.5.9			○
	Particle Shape	2.5.10			○
8	Plasticity (for fine-grained soil)	2.5.11	3.2.5		○
9	Dry Strength (for fine-grained soil)	2.5.12			○
10	Dilatency (for fine-grained soil)	2.5.13			○
11	Toughness (for fine-grained soil)	2.5.14			○
12	Structure	2.5.15			○
13	Cementation	2.5.16		●	
14	Percent of Cobbles and Boulders	2.5.17		●	
	Description of Cobbles and Boulders	2.5.18		●	
15	Consistency Field Test Result	2.5.3		●	
16	Additional Comments	2.5.19			○

Describe the soil using descriptive terms in the order shown

Minimum Required Sequence:

USCS Group Name (Group Symbol); Consistency or Density; Color; Moisture; Percent or Proportion of Soil; Particle Size; Plasticity (optional).

◍ = optional for non-Caltrans projects

Where applicable:

Cementation; % cobbles & boulders;
Description of cobbles & boulders;
Consistency field test result

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010)

HOLE IDENTIFICATION

Holes are identified using the following convention:

H-YY-NNN

Where:
H: Hole Type Code
YY: 2-digit year
NNN: 3-digit number (001-999)

Hole Type Code	Description
A	Auger boring (hollow or solid stem, bucket)
R	Rotary drilled boring (conventional)
RC	Rotary core (self-cased wire-line, continuously-sampled)
RW	Rotary core (self-cased wire-line, not continuously sampled)
P	Rotary percussion boring (Air)
HD	Hand driven (1-inch soil tube)
HA	Hand auger
D	Driven (dynamic cone penetrometer)
CPT	Cone Penetration Test
O	Other (note on LOTB)

Description Sequence Examples:

SANDY lean CLAY (CL); very stiff; yellowish brown; moist; mostly fines; some SAND, from fine to medium; few gravels; medium plasticity; PP=2.75.

Well-graded SAND with SILT and GRAVEL and COBBLES (SW-SM); dense; brown; moist; mostly SAND, from fine to coarse; some fine GRAVEL; few fines; weak cementation; 10% GRANITE COBBLES; 3 to 6 inches; hard; subrounded.

Clayey SAND (SC); medium dense, light brown; wet; mostly fine sand,; little fines; low plasticity.

GROUP DELTA

GROUP DELTA CONSULTANTS, INC. GEOTECHNICAL ENGINEERS AND GEOLOGISTS	FIGURE NUMBER **A-1A**
PROJECT NAME BRISTOL COMMONS PROJECT SANTA ANA, CA	PROJECT NUMBER IR-737

BORING RECORD LEGEND #1

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

- **C** Consolidation (ASTM D 2435-04)
- **CL** Collapse Potential (ASTM D 5333-03)
- **CP** Compaction Curve (CTM 216 - 06)
- **CR** Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
- **CU** Consolidated Undrained Triaxial (ASTM D 4767-02)
- **DS** Direct Shear (ASTM D 3080-04)
- **EI** Expansion Index (ASTM D 4829-03)
- **M** Moisture Content (ASTM D 2216-05)
- **OC** Organic Content (ASTM D 2974-07)
- **P** Permeability (CTM 220 - 05)
- **PA** Particle Size Analysis (ASTM D 422-63 [2002])
- **PI** Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
- **PL** Point Load Index (ASTM D 5731-05)
- **PM** Pressure Meter
- **PP** Pocket Penetrometer
- **R** R-Value (CTM 301 - 00)
- **SE** Sand Equivalent (CTM 217 - 99)
- **SG** Specific Gravity (AASHTO T 100-06)
- **SL** Shrinkage Limit (ASTM D 427-04)
- **SW** Swell Potential (ASTM D 4546-03)
- **TV** Pocket Torvane
- **UC** Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
- **UU** Unconsolidated Undrained Triaxial (ASTM D 2850-03)
- **UW** Unit Weight (ASTM D 4767-04)
- **VS** Vane Shear (AASHTO T 223-96 [2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (after drilling, date)

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010)

DEFINITIONS FOR CHANGE IN MATERIAL

Term	Definition	Symbol
Material Change	Change in material is observed in the sample or core, and the location of change can be accurately measured.	————
Estimated Material Change	Change in material cannot be accurately located because either the change is gradational or because of limitations in the drilling/sampling methods used.	- - - - - - - - -
Soil/Rock Boundary	Material changes from soil characteristics to rock characteristics.	

GROUP DELTA

GROUP DELTA CONSULTANTS, INC. GEOTECHNICAL ENGINEERS AND GEOLOGISTS	FIGURE NUMBER A-1B
PROJECT NAME BRISTOL COMMONS PROJECT SANTA ANA, CA	PROJECT NUMBER IR-737

BORING RECORD LEGEND #2

CONSISTENCY OF COHESIVE SOILS

Descriptor	Shear Strength (tsf)	Pocket Penetrometer, PP Measurement (tsf)	Torvane, TV. Measurement (tsf)	Vane Shear, VS. Measurement (tsf)
Very Soft	< 0.12	< 0.25	< 0.12	< 0.12
Soft	0.12 - 0.25	0.25 - 0.50	0.12 - 0.25	0.12 - 0.25
Medium Stiff	0.25 - 0.50	0.50 - 1.0	0.25 - 0.50	0.25 - 0.50
Stiff	0.50 - 1.0	1.0 - 2.0	0.50 - 1.0	0.50 - 1.0
Very Stiff	1.0 - 2.0	2.0 - 4.0	1.0 - 2.0	1.0 - 2.0
Hard	> 2.0	> 4.0	> 2.0	> 2.0

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE

Descriptor		Size (in)
Boulder		> 12
Cobble		3 - 12
Gravel	Coarse	3/4 - 3
	Fine	1/5 - 3/4
Sand	Coarse	1/16 - 1/5
	Medium	1/64 - 1/16
	Fine	1/300 - 1/64
Silt and Clay		< 1/300

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CONSISTENCY OF COHESIVE SOILS VS. N_{60}

Description	SPT N_{60} (blows / foot)
Very Soft	0 - 2
Soft	2 - 4
Medium Stiff	4 - 8
Stiff	8 - 15
Very Stiff	15 - 30
Hard	> 30

Ref: Peck, Hansen, and Thornburn, 1974, "Foundation Engineering", Second Edition

Note: Only to be used (with caution) when pocket penetrometer or other data on undrained shear strength are unavailable. Not allowed by Caltrans Soil and Rock Logging and Classificaton Manual, 2010

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010), with the exception of consistency of cohesive soils vs. N_{60}.

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.
GEOTECHNICAL ENGINEERS AND GEOLOGISTS

FIGURE NUMBER
A-1C

PROJECT NAME
BRISTOL COMMONS PROJECT
SANTA ANA, CA

PROJECT NUMBER
IR-737

BORING RECORD LEGEND #3

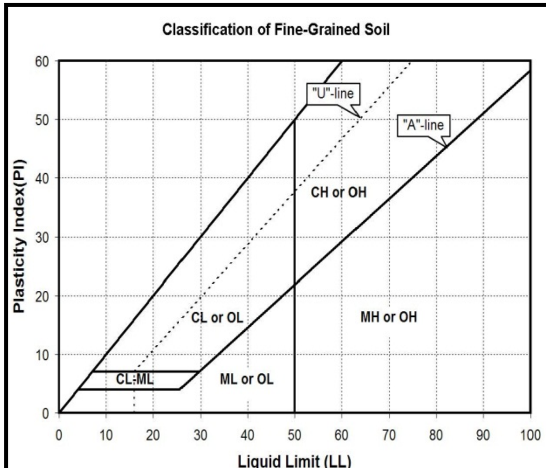
CLASSIFICATION OF INORGANIC FINE GRAINED SOILS (Soils with ≥50% finer than No. 200 Sieve)

GROUP SYMBOL				GROUP NAME
CL	<30% plus No. 200	<15% plus No. 200		Lean clay
		15-25% plus No. 200	%sand ≥ %gravel	Lean clay with sand
			%sand < %gravel	Lean clay with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy lean clay
			≥15% gravel	Sandy lean clay with gravel
		% sand < % gravel	<15% sand	Gravelly lean clay
			≥15% sand	Gravelly lean clay with sand
ML	<30% plus No. 200	<15% plus No. 200		Silt
		15-25% plus No. 200	%sand ≥ %gravel	Silt with sand
			%sand < %gravel	Silt with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy silt
			≥15% gravel	Sandy silt with gravel
		% sand < % gravel	<15% sand	Gravelly silt
			≥15% sand	Gravelly silt with sand
CH	<30% plus No. 200	<15% plus No. 200		Fat clay
		15-25% plus No. 200	%sand ≥ %gravel	Fat clay with sand
			%sand < %gravel	Fat clay with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy fat clay
			≥15% gravel	Sandy fat clay with gravel
		% sand < % gravel	<15% sand	Gravelly fat clay
			≥15% sand	Gravelly fat clay with sand
MH	<30% plus No. 200	<15% plus No. 200		Elastic silt
		15-25% plus No. 200	%sand ≥ %gravel	Elastic silt with sand
			%sand < %gravel	Elastic silt with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy elastic silt
			≥15% gravel	Sandy elastic silt with gravel
		% sand < % gravel	<15% sand	Gravelly elastic silt
			≥15% sand	Gravelly elastic silt with sand

Reference:
ASTM D 2487 and 2488

Laboratory Classification of Clay and Silt

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).



CL: LL<50; above A-Line.

CH: LL≥50; above A-Line.

ML: LL<50; below A-Line, or PI<4, or Non-Plastic

MH: LL≥50; below A-Line.

CL-ML: above A-Line and PI=4 to 7

CL/CH, ML/MH: at or near LL=50

ML/CL, MH/CH: at or near the A-Line

Field Identification of Clays and Silts

Group Symbol	Dry Strength	Dilatancy	Toughness	Plasticity
ML	None to low	Slow to rapid	Low or thread cannot be formed	Low to nonplastic
CL	Medium to high	None to slow	Medium	Medium
MH	Low to medium	None to slow	Low to medium	Low to medium
CH	High to very high	None	High	High

Group Delta Project No. IR-737

BRISTOL COMMONS PROJECT
SANTA ANA, CA

KEY FOR SOIL CLASSIFICATION #1

Figure A-1D

CLASSIFICATION OF COARSE-GRAINED SOILS (Soils with <50% "fines" passing No. 200 Sieve)

				GROUP SYMBOL		GROUP NAME
GRAVEL % GRAVEL> % SAND	≤5%fines (<5% fines)	Well-graded		GW	<15% sand	Well-graded gravel
					≥15% sand	Well-graded gravel with sand
		Poorly graded		GP	<15% sand	Poorly graded gravel
					≥15% sand	Poorly graded gravel with sand
	10%fines (5-12% fines)	Well-graded	Fines=ML or MH	GW-GM	<15% sand	Well-graded gravel with silt
					≥15% sand	Well-graded gravel with silt and sand
			Fines=CL or CH	GW-GC	<15% sand	Well-graded gravel with clay
					≥15% sand	Well-graded gravel with clay and sand
		Poorly graded	Fines=ML or MH	GP-GM	<15% sand	Poorly graded gravel with silt
					≥15% sand	Poorly graded gravel with silt and sand
			Fines=CL or CH	GP-GC	<15% sand	Poorly graded gravel with clay
					≥15% sand	Poorly graded gravel with clay and sand
	≥15%fines (>12% fines)		Fines=ML or MH	GM	<15% sand	Silty gravel
					≥15% sand	Silty gravel with sand
			Fines=CL or CH	GC	<15% sand	Clayey gravel
					≥15% sand	Clayey gravel with sand
SAND % SAND≥ % GRAVEL	≤5%fines (<5% fines)	Well-graded		SW	<15% gravel	Well-graded sand
					≥15% gravel	Well-graded sand with gravel
		Poorly graded		SP	<15% gravel	Poorly-graded sand
					≥15% gravel	Poorly-graded sand with gravel
	10%fines (5-12% fines)	Well-graded	Fines=ML or MH	SW-SM	<15% gravel	Well-graded sand with silt
					≥15% gravel	Well-graded sand with silt and gravel
			Fines=CL or CH	SW-SC	<15% gravel	Well-graded sand with clay
					≥15% gravel	Well-graded sand with clay and gravel
		Poorly graded	Fines=ML or MH	SP-SM	<15% gravel	Poorly graded sand with silt
					≥15% gravel	Poorly graded sand with silt and gravel
			Fines=CL or CH	SP-SC	<15% gravel	Poorly graded sand with clay
					≥15% gravel	Poorly graded sand with clay and gravel
	≥15%fines (>12% fines)		Fines=ML or MH	SM	<15% gravel	Silty sand
					≥15% gravel	Silty sand with gravel
			Fines=CL or CH	SC	<15% gravel	Clayey sand
					≥15% gravel	Clayey sand with gravel

Reference:

ASTM D 2487 and 2488

Note: Values estimated to nearest 5% to be used for visual identification, values in parentheses to be used for classification when based on laboratory grain size data.

Granular Soil Gradation Parameters

Coefficient of Uniformity: $C_u = D_{60}/D_{10}$

Coefficient of Curvature: $Cc = D_{30}^2 / (D_{60} \times D_{10})$

D_{10} = 10% of soil is finer than this diameter

D_{30} = 30% of soil is finer than this diameter

D_{60} = 60% of soil is finer than this diameter

Group Symbol	Gradation or Plasticity Requirement
SW	$C_u > 6$ and $1 \leq C_c \leq 3$
GW	$C_u > 4$ and $1 \leq C_c \leq 3$
GP or SP	Clean gravel or sand not meeting requirement for SW or GW
SM or GM	Non-plastic fines or below A-Line or PI<4
SC or GC	Plastic fines or above A-Line and PI>7

GROUP DELTA

Group Delta Project No. IR-737

BRISTOL COMMONS PROJECT
SANTA ANA, CA

KEY FOR SOIL CLASSIFICATION #2

Figure A-1E

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/5/2021	1/5/2021	1 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 85	Hollow Stem Auger	Y. Wang	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	62.6%	8	31.5	34	⊻ 14.5 / *19.5* DURING DRILLING; ▼ NM / *NE* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
SPT (1.4"), CAL (2.4")	$N_{60} = 1.04N_{SPT} = 0.70N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															PAVEMENT: ASPHALT CONCRETE (2") over AGGREGATE BASE (6").
			B-1												CLAYEY SAND (SC); dark olive brown; moist; mostly fine to medium SAND; little fines; trace fine subangular GRAVEL; nonplastic.
			B-2									EI			Fat CLAY (CH); dark grey; moist; trace fine SAND; high plasticity.
	30														
5			R-3	3 4 4	8	6			8	103					SANDY Lean CLAY (CL); medium stiff; dark olive brown; moist; some fine to medium SAND; medium plasticity. PP=0.75 tsf
															Fat CLAY (CH); very stiff; dark grey; moist; trace fine to medium SAND; high plasticity. PP=2.5 tsf
	25														
10			S-4	3 3 3	6	6									Stiff; olive brown; trace fine SAND. PP=1.5 tsf
	20														
15			SH-5								46:31	CON			Sandy CLAY (CL): Stiff, brown, moist, mostly fines, little fine grained SAND, medium plasticity. PP=1.5 to 1.75 tsf
	15														

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE

A-2 a

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/5/2021	1/5/2021	2 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 85	Hollow Stem Auger	Y. Wang	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	62.6%	8	31.5	34	∇ 14.5 / *19.5* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
SPT (1.4"), CAL (2.4")	$N_{60} = 1.04N_{SPT} = 0.70N_{MC}$	▼ NM / *NE*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			R-6	4 24 34	58	41			13	119					Fat CLAY (CH); very stiff; dark grey; moist; trace fine to medium SAND; high plasticity. PP=2.0 tsf
															CLAYEY SAND with GRAVEL (SC); very dense; olive brown; wet; mostly fine to medium SAND; little fines; little coarse rounded GRAVEL; nonplastic.
	10														
25			S-7 S-8	9 12 12	24	25									Fat CLAY with SAND (CH); olive grey to brown; wet; little fine to coarse SAND; high plasticity.
															CLAYEY SAND (SC); medium dense; olive brown; wet; mostly fine to coarse SAND; some fines; low to nonplastic.
	5														
30			R-9	26 36 38	74	52									Well-graded SAND with SILT (SW-SM), very dense; light olive brown; wet; mostly fine to coarse SAND; few fines; nonplastic.
	0														Total depth = 31.5 feet (Target depth reached). Groundwater encountered at 14.5 feet during drilling. Boring converted into a monitoring well on 1/5/2021 shortly after drilling. This Boring Record was prepared in general accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
35															
	-5														

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	B-2

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/4/2021	1/4/2021	1 of 5

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Martini Drilling	CME 75	Hollow Stem Auger	G. Valdivia	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	79.3%	8	71.5	33	▽ 16.0 / *17.0* DURING DRILLING; ▼ NM / *NE* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
SPT (1.4"), CAL (2.4")	$N_{60} = 1.32N_{SPT} = 0.89N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	33														**PAVEMENT:** ASPHALT CONCRETE (4.5") over AGGREGATE BASE (2").
															Fat CLAY with SAND (CH): dark grey, moist, mostly fines, few fine grained SAND, high plasticity.
															Gravel =0.2% Sand = 17.8% Fines = 82%
	30														
5															Very stiff, PP=3.0 tsf
			R-1	4 7 11	18	16			32	87	54:33				
	25														
10															Stiff, PP=1.5 tsf
			S-2	1 2 3	5	7									
	20														
15															Tan brown
			SH-3												
	15														

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FIGURE A-3 a

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	B-2

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/4/2021	1/4/2021	2 of 5

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Martini Drilling	CME 75	Hollow Stem Auger	G. Valdivia	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	79.3%	8	71.5	33	▽ 16.0 / *17.0* DURING DRILLING; ▼ NM / *NE* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
SPT (1.4"), CAL (2.4")	$N_{60} = 1.32N_{SPT} = 0.89N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			R-4	2 4 6	10	9			34	87					Lean CLAY (CL); medium stiff; light brown; moist; medium plasticity. PP=0.75 tsf
	10														
25			S-5	2 2 4	6	8									SANDY lean CLAY (CL); very stiff; reddish brown; moist; some fine SAND; medium plasticity. PP=2.25 tsf
	5														
30			R-6	3 6 16	22	20			11	121					Well-graded SAND with CLAY (SW-SC); medium dense; light brown; wet; mostly coarse SAND; few fines; nonplastic.
	0														
35			S-7	8 6 8	14	18									Well-graded SAND (SW); medium dense; light brown; wet; mostly coarse SAND; trace fines; nonplastic.
	-5														

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BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	B-2

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/4/2021	1/4/2021	3 of 5

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Martini Drilling	CME 75	Hollow Stem Auger	G. Valdivia	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	79.3%	8	71.5	33	▽ 16.0 / *17.0* DURING DRILLING; ▼ NM / *NE* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
SPT (1.4"), CAL (2.4")	$N_{60} = 1.32N_{SPT} = 0.89N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		CAL	R-8	4 7 13	20	18			25	101					SANDY SILT (ML); very stiff; tan brown; moist; some fine SAND; low plasticity. PP=2.75 tsf
	-10														
45		SPT	S-9	6 7 10	17	22									Stiff. PP=1.0 tsf
	-15														
50		CAL	R-10	6 12 16	28	25			31	93					Hard; yellow to greyish brown medium plasticity. PP=4.25 tsf
	-20														
55		SPT	S-11	3 5 10	15	20									SILT (ML); medium stiff; tan brown; moist; low plasticity. PP=0.75 tsf
	-25														

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	B-2

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/4/2021	1/4/2021	4 of 5

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Martini Drilling	CME 75	Hollow Stem Auger	G. Valdivia	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	79.3%	8	71.5	33	▽ 16.0 / *17.0* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
SPT (1.4"), CAL (2.4")	$N_{60} = 1.32N_{SPT} = 0.89N_{MC}$	▼ NM / *NE*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			R-12	4 7 12	19	17			29	95					Very stiff, PP=3.0 tsf
	-30														
65															
	-35														
70			S-13	2 4 8	12	16									SANDY lean CLAY (CL); medium stiff; dark grey; moist; some fine grained SAND; medium plasticity. PP=0.75 tsf
	-40														
75															
	-45														

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	B-2

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/4/2021	1/4/2021	5 of 5

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Martini Drilling	CME 75	Hollow Stem Auger	G. Valdivia	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	79.3%	8	71.5	33	⊻ 16.0 / *17.0* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
SPT (1.4"), CAL (2.4")	$N_{60} = 1.32N_{SPT} = 0.89N_{MC}$	▼ NM / *NE*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			R-14	2 6 15	21	19									Well Graded SAND (SW): medium dense, brown, wet, mostly medium grained SAND.
	-50														Total depth = 81.5 feet (Target depth reached). Groundwater encountered at 16 feet during drilling. Boring backfilled on 1/4/2020 shortly after drilling with bentonite cement grout, and capped with cold patch asphalt. This Boring Record was prepared in general accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
85															
	-55														
90															
	-60														
95															
	-65														

GROUP DELTA

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FIGURE

A-3 e

BORING RECORD

PROJECT NAME Related Bristol Project			**PROJECT NUMBER** IR737		**HOLE ID** B-3
SITE LOCATION Santa Ana, CA			**START** 1/5/2021	**FINISH** 1/5/2021	**SHEET NO.** 1 of 2
DRILLING COMPANY ABC Liovin	**DRILL RIG** CME 85	**DRILLING METHOD** Hollow Stem Auger		**LOGGED BY** Y. Wang	**CHECKED BY**
HAMMER TYPE (WEIGHT/DROP) Hammer: 140 lbs., Drop: 30 in.	**HAMMER EFFICIENCY (ERi)** 62.6%	**BORING DIA. (in)** 8	**TOTAL DEPTH (ft)** 30.5	**GROUND ELEV (ft)** 33	**DEPTH/*ELEV.* GW (ft)** ∇ 12.8 / *20.2* DURING DRILLING; ▼ NM / *NE* AFTER DRILLING
DRIVE SAMPLER TYPE(S) & SIZE (ID) SPT (1.4"), CAL (2.4")	**NOTES** $N_{60} = 1.04N_{SPT} = 0.70N_{MC}$				

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	33														**PAVEMENT:** ASPHALT CONCRETE (2") over AGGREGATE BASE (6").
															Fat CLAY (CH); dark grey; moist; trace fine SAND; high plasticity.
	30		B-1												
5			R-2	4 4 5	9	6			27	86					
															Sandy CLAY (CL); stiff; olive brown with rusted color; moist; few fine SAND; medium plasticity. PP=1.5 tsf
	25														
10			S-3	3 3 4	7	7									Fat CLAY (CH); stiff; olive brown; moist; trace fine SAND; high plasticity. PP=1.5 ~ 1.75 tsf
	20													∇	
15			SH-4												Stiff; dark grey. PP=1.25 tsf
	15														

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BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	**B-3**

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/5/2021	1/5/2021	2 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 85	Hollow Stem Auger	Y. Wang	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	62.6%	8	30.5	33	⊻ 12.8 / *20.2* DURING DRILLING; ▼ NM / *NE* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
SPT (1.4"), CAL (2.4")	$N_{60} = 1.04N_{SPT} = 0.70N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		CAL	R-5	5 7 12	19	13			21	106					Fat CLAY (CH); very stiff to hard; olive brown; moist; trace fine SAND; high plasticity. PP=2.5 ~ 4.0 tsf
	10														
25		SPT	S-6	6 6 10	16	17									SANDY lean CLAY (CL); olive brown; wet; some fine to medium SAND; medium plasticity.
	5														
30		CAL	R-7	50/6	50/6	50/6									Well-graded SAND with SILT (SW-SM), very dense; light olive brown; wet; mostly fine to coarse SAND; few fines; nonplastic.
	0														Total depth = 30.5 feet (Target depth reached). Groundwater encountered at 12.8 feet during drilling. Boring backfilled on 1/5/2020 shortly after drilling with bentonite cement grout, and capped with cold patch asphalt. This Boring Record was prepared in general accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
35															
	-5														

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE

A-4 b

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	B-4

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	2/14/2020	2/14/2020	1 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 85	Hollow Stem Auger	Y. Gao	A. Bieda

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	62.6%	8	51.5	34	⊻ 12.4 / *13.2* DURING DRILLING; ▼ NM / *NE* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
SPT (1.4"), CAL (2.4")	$N_{60} = 1.04N_{SPT} = 0.70N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															PAVEMENT: ASPHALT CONCRETE (2.4") over AGGREGATE BASE (2.4").
	25														Lean CLAY (CL); yellowish to reddish brown; moist; mostly fines; little fine SAND; little fine to coarse GRAVEL; medium plasticity.
			B-1									CR			
															Yellowish brown.
5	20		R-2	3 4 4	8	6			25.6	91					
															Medium stiff; reddish brown; little fine to coarse SAND.
10	15		R-3	5 7 9	16	11									Stiff; dark reddish brown; trace fine SAND.
15	10		R-4	4 5 6	11	8			29.4	94		PA PI C			
															Fat CLAY (CH); medium stiff; dark reddish brown; moist; mostly fines; trace fine SAND; trace fine GRAVEL; high plasticity. (LL=56; PL=22; PI=34)

BORING RECORD

PROJECT NAME: Related Bristol Project	PROJECT NUMBER: IR737	HOLE ID: **B-4**	
SITE LOCATION: Santa Ana, CA	START: 2/14/2020	FINISH: 2/14/2020	SHEET NO.: 2 of 3
DRILLING COMPANY: ABC Liovin	DRILL RIG: CME 85	DRILLING METHOD: Hollow Stem Auger	LOGGED BY: Y. Gao; CHECKED BY: A. Bieda
HAMMER TYPE (WEIGHT/DROP): Hammer: 140 lbs., Drop: 30 in.	HAMMER EFFICIENCY (ERi): 62.6%	BORING DIA. (in): 8; TOTAL DEPTH (ft): 51.5; GROUND ELEV (ft): 34	DEPTH/*ELEV.* GW (ft): ▽ 12.4 / *13.2* DURING DRILLING; ▼ NM / *NE* AFTER DRILLING
DRIVE SAMPLER TYPE(S) & SIZE (ID): SPT (1.4"), CAL (2.4")	NOTES: $N_{60} = 1.04N_{SPT} = 0.70N_{MC}$		

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	5		S-5	1 2 2	4	4									Fat CLAY (CH); soft; dark reddish brown; moist; mostly fines; trace fine SAND; trace fine GRAVEL; high plasticity. (95% SAND; 5% Fines)
25	0		R-6	7 7 7	14	10			15.2	116					Lean CLAY (CL); hard; olive brown; moist; mostly fines; little fine SAND; medium to low plasticity; oxidation staining present.
30	-5		S-7	5 12 14	26	27									CLAYEY SAND (SC); dense; reddish brown; moist; mostly fine to coarse SAND; little fines; low plasticity; oxidation staining present.
35	-10		R-8	4 16 33	49	34									SILTY SAND (SM); very dense; reddish brown; wet; mostly medium to coarse SAND; little fines; nonplastic.

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BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	**B-4**

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	2/14/2020	2/14/2020	3 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 85	Hollow Stem Auger	Y. Gao	A. Bieda

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	62.6%	8	51.5	34	⊻ 12.4 / *13.2* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
SPT (1.4"), CAL (2.4")	$N_{60} = 1.04N_{SPT} = 0.70N_{MC}$	▼ NM / *NE*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	-15		S-9	30 50/6"	REF	REF									Poorly-graded SAND (SP); very dense; reddish brown; wet; mostly medium to coarse SAND; trace fines; nonplastic.
45	-20		R-10	24 50/6"	REF	REF			11.3	129					Trace fine SAND.
50	-25		S-11	9 15 15	30	31									Medium dense.
															Lean CLAY (CL); very stiff; olive gray; wet; mostly fines; trace fine SAND; low plasticity; in shoe of samples.
55	-30														Total depth = 51.5 feet (Target depth reached). Groundwater encountered at 12.4 feet during drilling. Boring backfilled on 2/14/2020 shortly after drilling with bentonite cement grout, and capped with black-dyed rapid set concrete. This Boring Record was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	**B-5**

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	2/14/2020	2/14/2020	1 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 85	Hollow Stem Auger	Y. Gao	A. Bieda

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	62.6%	8	31.5	34	∇ 15.1 / *11.3* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
SPT (1.4"), CAL (2.4")	$N_{60} = 1.04N_{SPT} = 0.70N_{MC}$	▼ NM / *NE*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0															**PAVEMENT:** ASPHALT CONCRETE (2.4") over AGGREGATE BASE (3.6").
	25														Lean CLAY (CL); brownish gray; moist; mostly fines; few fine SAND; medium plasticity. (EI=85; Medium)
			B-1									EI			
5															Very stiff.
			R-2	6 7 12	19	13									
	20														
10															Fat CLAY (CH); soft; yellowish brown; moist; mostly fines; trace fine SAND; high plasticity. (4% SAND; 96% Fines) & (LL=66; PL=26; PI=40)
			S-3	1 1 2	3	3						PA PI			
	15														
15															∇ Stiff; yellowsh brown to greenish brown; wet; few trace fine SAND.
			R-4	2 4 7	11	8						UU			
	10														

GDC_LOG_BORING_2016 IR737- BRISTOL COMMONS LOGS.GPJ GDC2013.GDT 1/25/21

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Irvine, CA 92618

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE
A-6 a

BORING RECORD

PROJECT NAME: Related Bristol Project	PROJECT NUMBER: IR737	HOLE ID: **B-5**		
SITE LOCATION: Santa Ana, CA	START: 2/14/2020	FINISH: 2/14/2020	SHEET NO.: 2 of 2	
DRILLING COMPANY: ABC Liovin	DRILL RIG: CME 85	DRILLING METHOD: Hollow Stem Auger	LOGGED BY: Y. Gao	CHECKED BY: A. Bieda
HAMMER TYPE (WEIGHT/DROP): Hammer: 140 lbs., Drop: 30 in.	HAMMER EFFICIENCY (ERi): 62.6%	BORING DIA. (in): 8	TOTAL DEPTH (ft): 31.5	GROUND ELEV (ft): 34
DEPTH/*ELEV.* GW (ft): ⊻ 15.1 / *11.3* DURING DRILLING; ▼ NM / *NE* AFTER DRILLING				
DRIVE SAMPLER TYPE(S) & SIZE (ID): SPT (1.4"), CAL (2.4")	NOTES: $N_{60} = 1.04N_{SPT} = 0.70N_{MC}$			

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	5	SPT	S-5	3 3 4	7	7									Fat CLAY (CH); medium stiff; yellowish brown; moist; mostly fines; trace fine SAND; high plasticity.
25	0	CAL	R-6	4 5 7	12	8									SANDY lean CLAY (CL); very stiff; reddish brown; wet; mostly fines; little fine SAND; medium plasticity.
30	-5	SPT	S-7	5 7 13	20	21									Light reddish brown.
35	-10														Total depth = 31.5 feet (Target depth reached). Groundwater encountered at 15.1 feet during drilling. Boring backfilled on 2/14/2020 shortly after drilling with bentonite cement grout, and capped with black-dyed rapid set concrete. This Boring Record was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).

GDC_LOG_BORING_2016 IR737- BRISTOL COMMONS LOGS.GPJ GDC2013.GDT 1/25/21

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	P-1

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/5/2021	1/5/2021	1 of 1

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Martini Drilling	CME 75	Hollow Stem Auger	G. Valdivia	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
		8	5	34	⊻ NE / *NE* DURING DRILLING
					▼ / *NE* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^{*}_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															PAVEMENT: ASPHALT CONCRETE (2") over AGGREGATE BASE (6").
															CLAYEY SAND (SC); dark olive brown; moist; mostly fine to medium SAND; little fines; trace fine subangular GRAVEL; nonplastic.
															Fat CLAY (CH); dark grey; moist; trace fine SAND; high plasticity.
	30														
5															
															Total depth = 5.0 feet (Target depth reached). Groundwater was not encountered during drilling. 2-inch percolation pipe was installed shortly after drilling. This Boring Record was prepared in general accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
	25														
10															
	20														
15															
	15														

GROUP DELTA

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Irvine, CA 92618

FIGURE A-7

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	P-2

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/5/2021	1/5/2021	1 of 1

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Martini Drilling	CME 75	Hollow Stem Auger	G. Valdivia	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
		8	5	33	⊻ NE / *NE* DURING DRILLING ▼ / *NE* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															PAVEMENT: ASPHALT CONCRETE (4.5") over AGGREGATE BASE (2").
	25														Fat CLAY with SAND (CH): dark grey, moist, mostly fines, few fine grained SAND, high plasticity.
5															
	20														Total depth = 5.0 feet (Target depth reached). Groundwater was not encountered during drilling. 2-inch percolation pipe was installed shortly after drilling. This Boring Record was prepared in general accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
10															
	15														
15															
	10														

GDC_LOG_BORING_2016 IR737- BRISTOL COMMONS LOGS.GPJ GDC2013.GDT 1/25/21

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	P-5

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/5/2021	1/5/2021	1 of 1

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Martini Drilling	CME 75	Hollow Stem Auger	G. Valdivia	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
		8	5	34	∇ NE / *NE* DURING DRILLING; ▼ / *NE* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	34														**PAVEMENT:** ASPHALT CONCRETE (2.4") over AGGREGATE BASE (3.6").
	25														Lean CLAY (CL); brownish gray; moist; mostly fines; few fine SAND; medium plasticity.
5															Total depth = 5.0 feet (Target depth reached). Groundwater was not encountered during drilling. 2-inch percolation pipe was installed shortly after drilling. This Boring Record was prepared in general accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
	20														
10															
	15														
15															
	10														

GDC_LOG_BORING_2016 IR737- BRISTOL COMMONS LOGS.GPJ GDC2013.GDT 1/25/21

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Related Bristol Project	IR737	**P-6**

SITE LOCATION	START	FINISH	SHEET NO.
Santa Ana, CA	1/5/2021	1/5/2021	1 of 1

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Martini Drilling	CME 75	Hollow Stem Auger	G. Valdivia	

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
		8	5	34	⊻ NE / *NE* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	
		AFTER DRILLING ▼ / *NE*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0															**PAVEMENT:** ASPHALT CONCRETE (2.4") over AGGREGATE BASE (3.6").
															Fat CLAY with SAND (CH): dark grey, moist, mostly fines, few fine grained SAND, high plasticity.
	25														
5															Total depth = 5.0 feet (Target depth reached). Groundwater was not encountered during drilling. 2-inch percolation pipe was installed shortly after drilling. This Boring Record was prepared in general accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
	20														
10															
	15														
15															
	10														

GROUP DELTA

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Irvine, CA 92618

FIGURE

A-10

EARTHSPECTIVES
250 Goddard
Irvine, California 92618

Phone: (949) 777-1270
Fax: (949) 777-1283

October 29, 2019

ABC Liovin Drilling Inc.
1180 East Burnett Street
Signal Hill, California 90755

Attention: Mr. Ivan Liovin

Dear Mr. Liovin:

SPT Hammer Energy Measurement
Drill Rigs R-1 (CME-85) and R-5 (CME-85)
ES Project No. 190806-365

INTRODUCTION

This letter report summarizes the results of EarthSpectives' (ES) SPT hammer energy measurements performed on October 12, 2019. It provides a description of the test program and the results. Testing was performed on two CME 85 Drill Rigs equipped with Auto Trip hammers.

SPT energy measurements were accomplished using a Pile Driving Analyzer (PDA) system manufactured by Pile Dynamics, Inc. and was conducted in general accordance with ASTM 4945 and 6066 test standards. Results are summarized in Table 1, while more details regarding energy records are provided in Appendix A.

TESTING CONDITIONS

SPT hammer energy measurements were performed on two drill rig/hammer combination that were equipped with an automatic trip hammer. Drill rigs R-1 and R-5 were both CME-85 Rigs. Samplings were performed using NWJ drilling rod.

INSTRUMENTATION

SPT energy measurements were performed by placing a 2 ft instrumented section of drill rod at the top of the drill string between the hammer and the sampling rods. The instruments consist of two sets of accelerometers and strain transducers, mounted on opposite sides of the drill rod, with a view to evaluate normal and eccentric effects. The analyzer acquired and processed the signals during sampling, and provided real-time evaluations of the maximum SPT hammer transferred energy. The raw data were stored directly on a portable field computer for subsequent analysis in the office.

RESULTS

Results from SPT hammer energy measurements are summarized in Tables 1. It shows the Energy Transfer Ratio (ETR) for every sampling depth for the tested drill rig/hammer. ETR is the ratio of the measured maximum transferred energy to rated energy of the hammer which is the product of the weight of the hammer times the height of fall (140 lb x 30 inches = 4200 lb-in = 0.35 kip-ft).

Plots of the maximum transferred energy, energy transfer ratio, and blow rate is provided as function of depth in Appendix A. Table immediately following the plot also provides the minimum, maximum, and average values at every sampling depth. In general, average ETR value for the tested hammers were 83.5% and 62.6% for Drill Rigs R-1 and R-5, respectively, over all the sampling intervals as shown in Table 1.

TABLE 1 – SUMMARY OF SPT HAMMER ENERGY MEASUREMTS

Drill Rig Number Type and Model	**AVERAGE SPT HAMMER EFFICIENCY (ENERGY TRANSFER RATIO)**			
	Data Set # 1	Data Set # 2	Data Set # 3	Data Set # 4
Drill Rig R-1 CME 85	80.5%	87.5%	84%	82.1%
Drill Rig R-5 CME 85	63.7%	65.1%	61.4%	60.1%

LIMITATIONS

Professional judgments represented in this report are based on evaluations of the technical information gathered, our understanding of the proposed construction, and our general experience in the geotechnical field. We do not guarantee the performance of the project in any respect, only that our engineering work and judgments are rendered while striving to meet the standard of care of our profession at this time.

CLOSURE

We hope the above information satisfies the project needs at this time. Please call if you have any question or need more information.

Sincerely submitted for EarthSpectives,

Hossein Rashidi

Hossein K. Rashidi, PhD, PE
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER HOSSEIN K. RASHIDI No. C 055140 Exp. 6/30/18 CIVIL STATE OF CALIFORNIA

EARTHSPECTIVES
1920 E Warner Avenue, Suite 3-M
Santa Ana, California 92705
Phone: (949) 777-1270
Fax: (949) 777-1283

June 26, 2019

GeoDesign, Inc.
2121 S Towne Centre Place, Suite 104
Anaheim, California 92806
Attention: Mr. Andrew Atry

Dear Mr. Atry:

SPT Hammer Energy Measurement
Martini CME 75 Drill Rig # 1 Serial Number 208497 and CME 75 Drill Rig # 3 Serial Number 174752
ES Project No. 190604-254

INTRODUCTION

This letter report summarizes the results of EarthSpectives' (ES) SPT hammer energy measurements performed on June 14, 2019. It provides a description of the test program and the results. Testing was performed on two Drill Rigs equipped with Auto Trip hammers.

SPT energy measurements were accomplished using a Pile Driving Analyzer (PDA) system manufactured by Pile Dynamics, Inc. and was conducted in general accordance with ASTM 4945 and 6066 test standards. Results are summarized in Table 1, while more details regarding energy records are provided in Appendix A.

TESTING CONDITIONS

SPT hammer energy measurements were performed on two drill rig/hammer combination that were equipped with an automatic trip hammer. Both rigs were CME 75 Drill Rigs. They were Drill Rig # 1 with Serial Number 208497 and Drill Rig # 3 with Serial Number 174752. Samplings were performed using an AWJ drill rod.

INSTRUMENTATION

SPT energy measurements were performed by placing a 2 ft instrumented section of drill rod at the top of the drill string between the hammer and the sampling rods. The instruments consist of two sets of accelerometers and strain transducers, mounted on opposite sides of the drill rod, with a view to evaluate normal and eccentric effects. The analyzer acquired and processed the signals during sampling, and provided real-time evaluations of the maximum SPT hammer transferred energy. The raw data were stored directly on a portable field computer for subsequent analysis in the office.

RESULTS

Results from SPT hammer energy measurements are summarized in Tables 1. It shows the Energy Transfer Ratio (ETR) for every sampling depth for the tested drill rig/hammer. ETR is the ratio of the measured maximum transferred energy to rated energy of the hammer which is the product of the weight of the hammer times the height of fall (140 lb x 30 inches = 4200 lb-in = 0.35 kip-ft).

Plots of the maximum transferred energy, energy transfer ratio, and blow rate is provided as function of depth in Appendix A. Table immediately following the plot also provides the minimum, maximum, and average values at every sampling depth. In general, average ETR value for the tested hammers were 77.5% and 79.3% for Drill Rigs # 1 (Serial Number 208497) and # 3 (Serial Number 174752), respectively, over all the sampling intervals as shown in Table 1.

TABLE 1 – SUMMARY OF SPT HAMMER ENERGY MEASUREMTS

Drill Rig Model, and Rig No.	**AVERAGE SPT HAMMER EFFICIENCY (ENERGY TRANSFER RATIO)**			
	Data Set # 1	Data Set # 2	Data Set #3	Average
Drill Rig # 1 CME 75 Serial Number 208497	78%	77.5%	76.6%	77.5%
Drill Rig # 3 CME 75 Serial Number 174752	82.3%	76.2%	77.4%	79.3%

LIMITATIONS

Professional judgments represented in this report are based on evaluations of the technical information gathered, our understanding of the proposed construction, and our general experience in the geotechnical field. We do not guarantee the performance of the project in any respect, only that our engineering work and judgments are rendered while striving to meet the standard of care of our profession at this time.

CLOSURE

We hope the above information satisfies the project needs at this time. Please call if you have any question or need more information.

Sincerely submitted for EarthSpectives,

Hossein Rashidi

Hossein K. Rashidi, PhD, PE
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
HOSSEIN K. RASHIDI
No. C 055140
Exp. 6/30/2020
CIVIL
STATE OF CALIFORNIA

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

B.1 General

The laboratory testing was performed using appropriate American Society for Testing and Materials (ASTM) and Caltrans Test Methods (CTM).

Modified California drive samples, Standard Penetration Test (SPT) drive samples, and bulk samples collected during the field investigation were carefully sealed in the field to prevent moisture loss. The samples of earth materials were then transported to the laboratory for further examination and testing. Tests were performed on selected samples as an aid in classifying the earth materials and to evaluate their physical properties and engineering characteristics. Laboratory testing for this investigation included:

- Soil Classification: USCS (ASTM D 2487) and Visual Manual (ASTM D 2488);
- Moisture content (ASTM D 2216) and Dry Unit Weight (ASTM D 2937);
- Atterberg Limits (ASTM D 4318);
- Grain Size Distribution (ASTM D 422) & % Passing #200 Sieve (ASTM D 1140);
- Triaxial Compression: UU (ASTM D 2850);
- One-Dimensional Consolidation (ASTM D 2435);
- Expansion Index (D 4829); and
- Soil Corrosivity:
    - pH (CTM 643);
    - Water-Soluble Sulfate (ASTM D 516, CTM 417);
    - Water-Soluble Chloride(Ion-Specific Probe, CTM 422);
    - Minimum Electrical Resistivity (CTM 643).

Brief descriptions of the laboratory testing program and test results are presented below.

B.2 Soil Classification

Earth materials recovered from subsurface explorations were classified in general accordance with Caltrans' "Soil and Rock Logging Classification Manual, 2010". The subsurface soils were classified visually / manually in the field in accordance with the Unified Soil Classification System (USCS) following ASTM D 2488; soil classifications were modified as necessary based on testing in the laboratory in accordance with ASTM D 2487. The details of the soil classification system and boring records presenting the classifications are presented in Appendix A.

B.3 Moisture Content and Dry Unit Weight

The in-situ moisture content of selected bulk, SPT, and Ring samples was determined by oven drying in general accordance with ASTM D 2216. Selected California Ring samples were trimmed flush in the metal rings and wet weight was measured. After drying, the dry weight of each sample was measured, volume and weight of the metal containers was measured, and moisture content and dry density were calculated in general accordance with ASTM D 2216 and D 2937. Results of these tests are presented on the boring records in Appendix A.

B.4 Atterberg Limits

Characterization of the fine-grained fractions of soils was evaluated using the Atterberg Limits. This test includes Liquid Limit and Plastic Limit tests to determine the Plasticity Index in accordance with ASTM D 4318. Results of these tests are presented on the boring records in Appendix A, are summarized in Table B-1, and are plotted on a Plasticity Chart in this Appendix.

B.5 Grain Size Distribution and Percent Passing No. 200 Sieve:

Representative samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. The percentage of fines (soil passing No. 200 sieve) was determined for selected samples in accordance with ASTM D 1140. For selected samples, the washed material retained on No. 200 sieve was shaken through a standard stack of sieves in accordance with ASTM D 422 to determine the grain size distribution. The results of grain size distribution tests are plotted in this appendix. The relative proportion (or percentage) by dry weight of gravel (retained on No. 4 sieve), sand (passing No. 4 and retained on No. 200 sieve), and fines (passing No. 200 sieve) are listed on the boring records in Appendix A.

B.6 Triaxial Compression Test

Unconsolidated Undrained Triaxial tests were performed on selected samples in accordance with ASTM D 4767 and ASTM D 2850. The test results are summarized in this appendix.

B.7 One-Dimensional Consolidation

The consolidation characteristics of representative soil samples under incremental loading were evaluated by performing one-dimensional consolidation in general accordance with

ASTM D 2435, using a floating ring consolidometer and dead weight system. Results of the tests are presented in this appendix.

B.8 Expansion Index

The expansion potential of the site soils was estimated using the Expansion Index Test in accordance with ASTM D 4829. The results of this test presented in this appendix.

B.9 Soil Corrosivity

Tests were performed in order to determine corrosion potential of site soils on concrete and ferrous metals. Corrosivity testing included minimum electrical resistivity and soil pH (Caltrans method 643), water-soluble chlorides (Caltrans Test Method 422), and water-soluble sulfates (ASTM D 516). The test results are summarized presented in this appendix.

B.10 List of Attached Figures

The following tables and figures are attached and complete this appendix:

List of Tables

List of Figures

Boring No.	Sample No.	Sample Depth (ft)	Sample Type[1]	USCS Group Symbol	Geologic Unit[2]	SPT N_{60} (blows/ft)	Undrained Shear Strength, Su (ksf): Pocket Penetro-meter	Undrained Shear Strength, Su (ksf): Miniature Vane	Undrained Shear Strength, Su (ksf): Unconfined Compres-sion Test	Moisture Content (%)	Dry Unit Weight (pcf)	Total Unit Weight (pcf)	Atterberg Limits: LL	Atterberg Limits: PL	Atterberg Limits: PI	Grain Size Distribution (%) by dry weight: Gravel	Grain Size Distribution (%) by dry weight: Sand	Grain Size Distribution (%) by dry weight: Fines	Grain Size Distribution (%) by dry weight: Clay µ(2(	Other Test
B-1	B-1	0-5	BULK	CH																
	R-2	5	MC	CL		6	0.8													
	R-3	10	SPT	CH		6	1.5			8.0	103									
	SH-4	15	SH	CL			1.5						46	15	31					
	R-5	20	MC	CH		41	2.0			13.0	119									
	S-6	25	SPT	SC		25														
	R-7	30	MC	SW-SM		52														
B-2	B-1	0-5	BULK	CH												0.2	17.8	82		
	R-2	5	MC	CH		16	3.0			32.0	87		54	21	33					
	S-3	10	SPT	CH		7	1.5													
	SH-4	15	SH	CH			1.5		1.83											
	R-5	20	MC	CL		9	0.8			34.0	87									
	S-6	25	SPT	CL		8	2.3													
	R-7	30	MC	SW-SC		20				11.0	121									
	S-8	35	SPT	SW		18														
	R-9	40	MC	ML		18	2.8			25.0	101									
	S-10	45	SPT	ML		22	1.0													
	R-11	50	MC	ML		25	4.3			31	93									
	S-12	55	SPT	ML		20	0.8													
	R-13	60	MC	ML		17	3.0			29.0	95									
	S-14	70	SPT	CL		16	0.8													
	R-15	80	MC	SW		19														
B-3	B-1	0-5	BULK	CH																
	R-2	5	MC	CL		6	1.5			27.0	86									
	S-3	10	SPT	CH		7	1.5													
	SH-4	15	SH	CH			1.3													
	R-5	20	MC	CH		13	2.5			21	106									
	S-6	25	SPT	CL		17														
	R-7	30	MC	SW-SM		50/6"														
B-4	B-1	0-5	BULK	CL																
	R-2	5	MC	CL		6				26	91									
	R-3	10	MC	CL		11														
	R-4	15	MC	CH		8				29	94		56	22	34					
	S-5	20	SPT	CH		4														
	R-6	25	MC	CL		10				15	116									
	S-7	30	SPT	SC		27														
	R-8	35	MC	SM		34														
	S-9	40	SPT	SP		50/6"														
	R-10	45	MC	SP		50/6"				11.3	129									
	S-11	50	SPT	SP		31														
B-5	B-1	0-5	BULK	CL																
	R-2	5	MC	CL		13														
	S-3	10	SPT	CH		3							66	26	40	0	4	96		
	R-4	15	MC	CH		8			2.3											UU
	S-5	20	SPT	CH		7														
	R-6	25	MC	CL		8														
	S-7	30	SPT	CL		21														

GROUP DELTA

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Irvine, California 92618
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TABLE B-1: Summary of Laboratory Results

Project: Related Bristol

Location: 3900 S. Bristol Street, Santa Ana

Number: IR737

ATTERBERG LIMITS

ASTM D-4318 / AASHTO T-89 / CTM 204

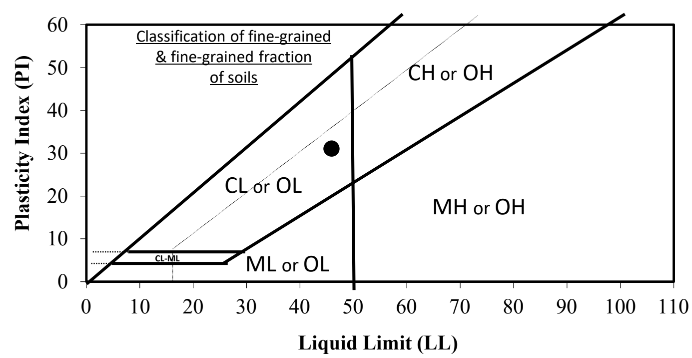
Project Name:	*Related Bristol*	Tested By :	*Eric Y.*	Date:	*01/18/21*
Project No. :	*IR737*	Data Input By:	*Eric Y.*	Date:	*01/19/21*
Boring No.:	*B-1*	Checked By:	*Mike G.*	Date:	
Sample No. :	*SH-4*	Depth (ft.) :	*15*		
Initial Moisture:		Container No.:	*AL-2*		
Description.:	*Brown Sandy Clay - CL*				

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			32	25	18	
Container No.	1	2	3	4	5	
Wet Wt. of Soil + Cont. (gm.)	32.30	31.98	37.91	39.92	40.76	
Dry Wt. of Soil + Cont. (gm.)	31.41	31.10	34.17	35.66	36.20	
Wt. of Container (gm.)	25.51	25.29	25.79	26.35	26.65	
Moisture Content (%) [Wn]	15.08	15.15	44.63	45.76	47.75	

LIQUID LIMIT	**46**
PLASTIC LIMIT	**15**
PLASTICITY INDEX	**31**

PI at "A" - Line = 0.73(LL-20) = **19.0**

One - Point Liquid Limit Calculation $LL=Wn(N/25)^{0.121}$



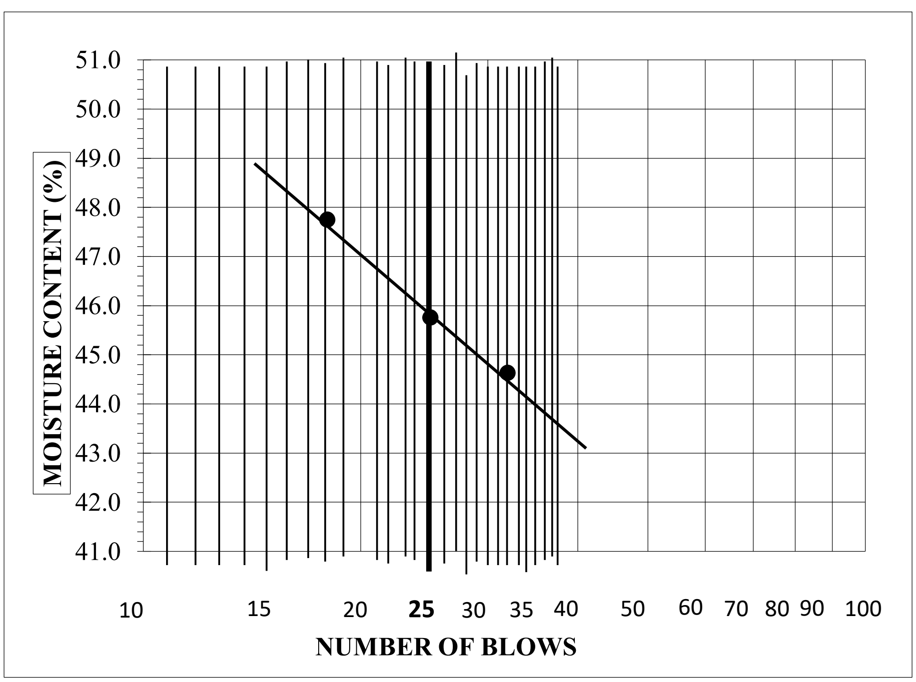
PROCEDURES USED

☐ Wet Preparation
Multipoint Wet Preparation

☒ Dry Preparation
Multipoint Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



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(714) 660-7550 fax

Figure: B-1A

ATTERBERG LIMITS

ASTM D-4318 / AASHTO T-89 / CTM 204

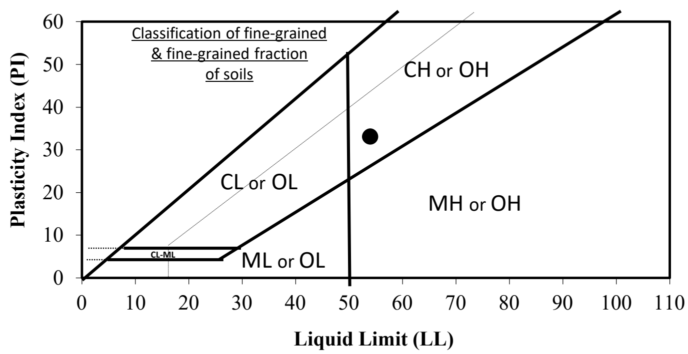
Project Name:	Related Bristol	Tested By :	Eric Y.	Date:	01/18/21
Project No. :	IR737	Data Input By:	Eric Y.	Date:	01/19/21
Boring No.:	B-2	Checked By:	Mike G.	Date:	
Sample No. :	R-2	Depth (ft.) :	5		
Initial Moisture:		Container No.:	AL-3		
Description.:	Olive Brown Fat Clay with Sand - CH				

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			33	24	17	
Container No.	6	7	8	9	10	
Wet Wt. of Soil + Cont. (gm.)	32.90	33.14	37.32	39.25	38.75	
Dry Wt. of Soil + Cont. (gm.)	31.70	31.92	32.92	34.66	33.88	
Wt. of Container (gm.)	26.00	26.13	24.52	26.22	25.23	
Moisture Content (%) [Wn]	21.05	21.07	52.38	54.38	56.30	

LIQUID LIMIT	**54**
PLASTIC LIMIT	**21**
PLASTICITY INDEX	**33**

PI at "A" - Line = 0.73(LL-20) = **24.8**

One - Point Liquid Limit Calculation $LL=W_n(N/25)^{0.121}$



PROCEDURES USED

☐ Wet Preparation
Multipoint Wet Preparation

☒ Dry Preparation
Multipoint Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test

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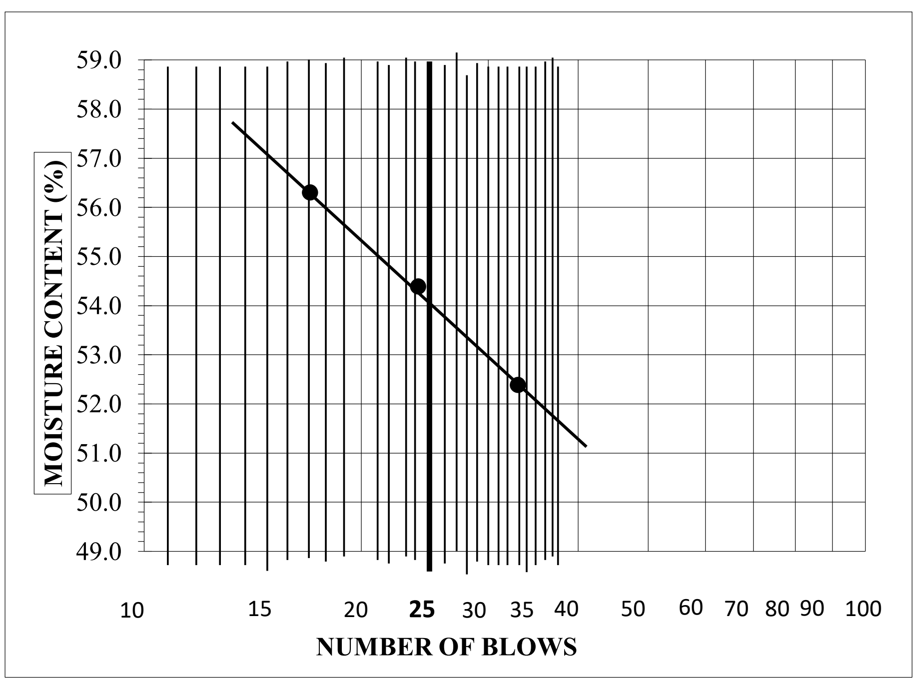


Figure: B-1B

ATTERBERG LIMITS

ASTM D-4318 / AASHTO T-89 / CTM 204

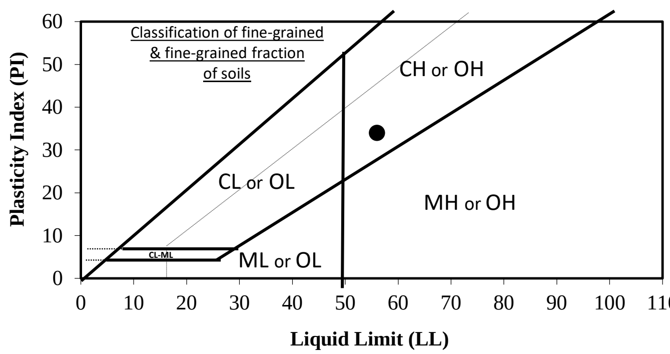
Project Name:	*Related Bristol*	Tested By :	*Eric Y.*	Date:	*02/21/20*
Project No. :	*IR737*	Data Input By:	*Eric Y.*	Date:	*02/24/20*
Boring No.:	*B-4*	Checked By:	*Mike G.*	Date:	
Sample No. :	*R-4*	Depth (ft.) :	*15*		
Initial Moisture:		Container No.:	*AL-3*		
Description.:	*Olive Brown Fat Clay with Sand - CH*				

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			34	26	17	
Container No.	11	12	13	14	15	
Wet Wt. of Soil + Cont. (gm.)	31.59	32.37	39.03	37.65	38.74	
Dry Wt. of Soil + Cont. (gm.)	30.35	31.12	34.84	33.10	33.96	
Wt. of Container (gm.)	24.71	25.44	27.08	24.92	25.69	
Moisture Content (%) [Wn]	21.99	22.01	53.99	55.62	57.80	

LIQUID LIMIT	**56**
PLASTIC LIMIT	**22**
PLASTICITY INDEX	**34**

PI at "A" - Line = 0.73(LL-20) = **26.3**

One - Point Liquid Limit Calculation $LL=Wn(N/25)^{0.121}$



PROCEDURES USED

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

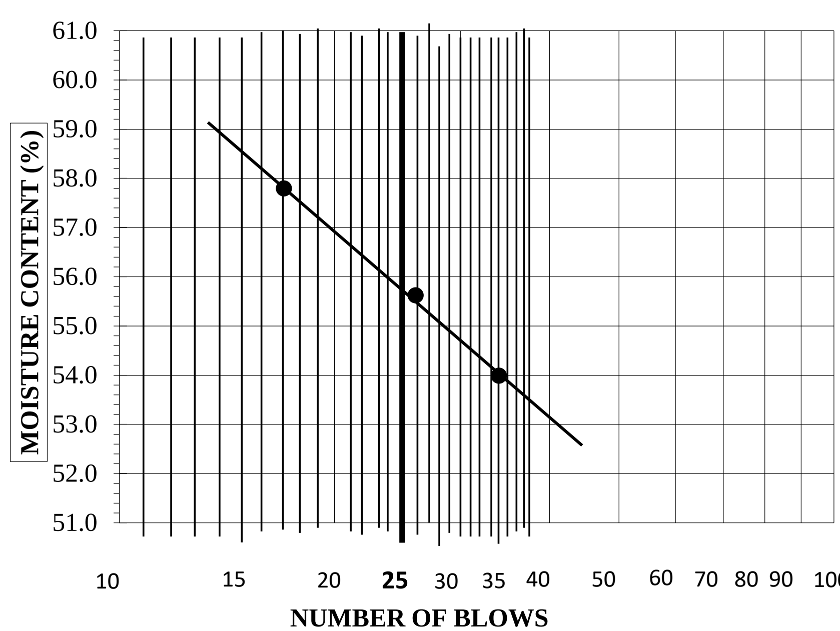


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Figure: B-1C

ATTERBERG LIMITS

ASTM D-4318 / AASHTO T-89 / CTM 204

Project Name: *Related Bristol*
Project No. : *IR737*
Boring No.: *B-5*
Sample No. : *S-3*
Initial Moisture:
Description.: *Very Dark Grayish Brown Fat Clay - CH*

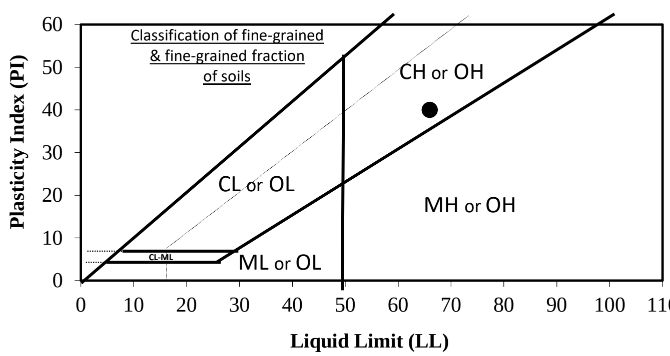
Tested By : *Eric Y.* Date: *02/21/20*
Data Input By: *Eric Y.* Date: *02/24/20*
Checked By: *Mike G.* Date:
Depth (ft.) : *10*
Container No.: *AL-4*

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			32	25	18	
Container No.	16	17	18	19	20	
Wet Wt. of Soil + Cont. (gm.)	31.81	31.37	38.97	39.81	38.57	
Dry Wt. of Soil + Cont. (gm.)	30.42	29.98	34.18	34.58	32.92	
Wt. of Container (gm.)	25.04	24.61	26.72	26.61	24.54	
Moisture Content (%) [Wn]	25.84	25.88	64.21	65.62	67.42	

LIQUID LIMIT	**66**
PLASTIC LIMIT	**26**
PLASTICITY INDEX	**40**

PI at "A" - Line = 0.73(LL-20) = **33.6**

One - Point Liquid Limit Calculation $LL=Wn(N/25)^{0.121}$



PROCEDURES USED

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

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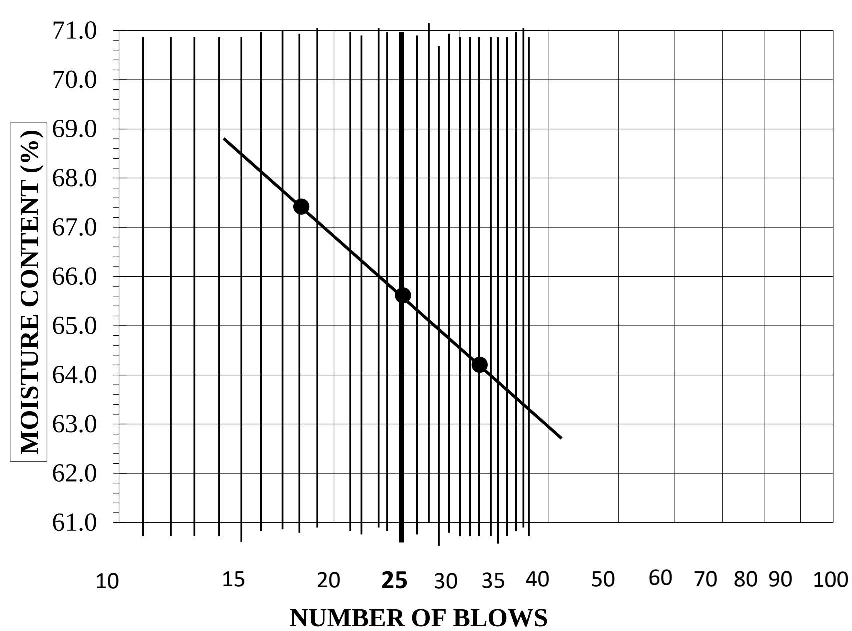
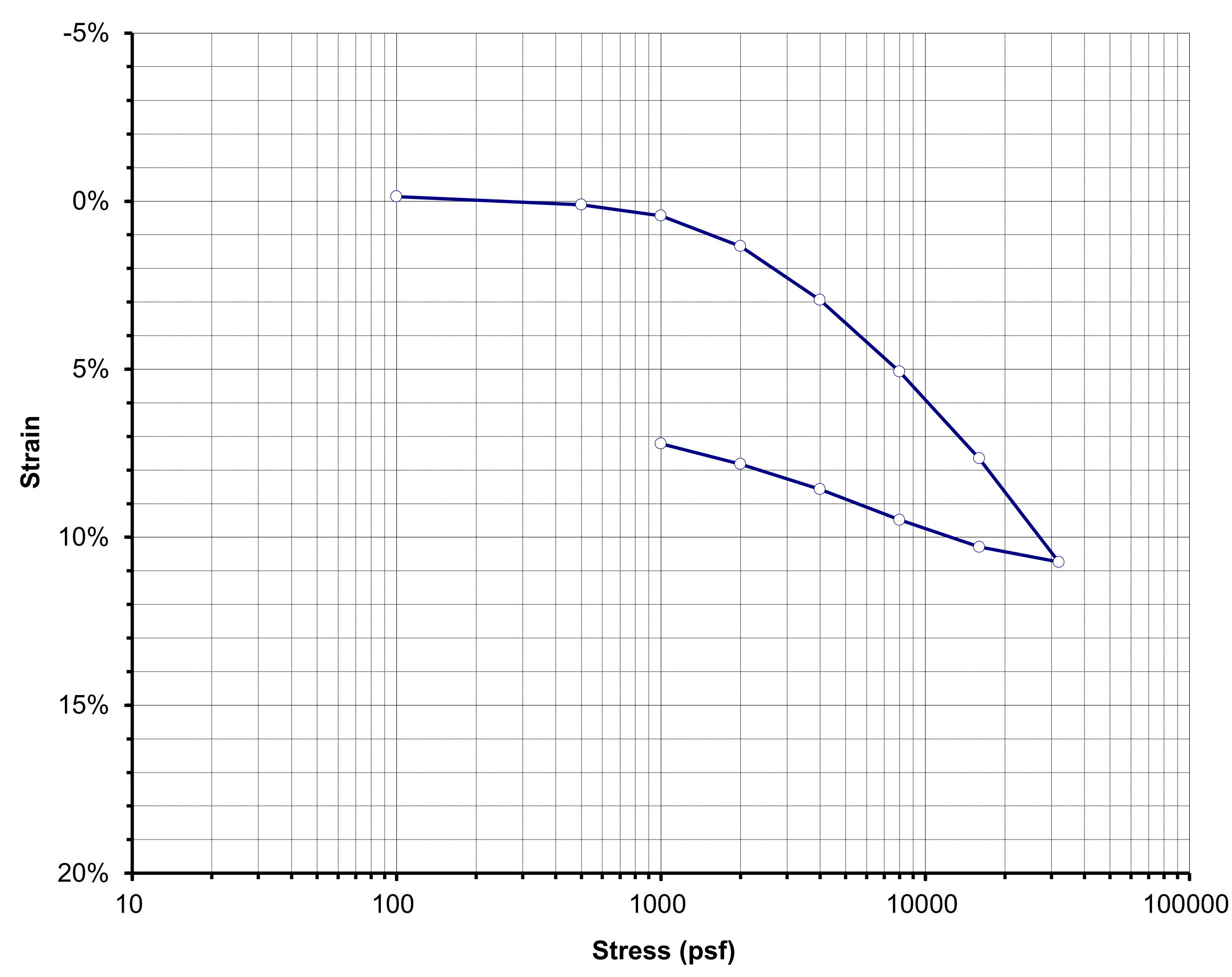


Figure: B-1D

CONSOLIDATION TEST RESULTS
ASTM D-2435



Boring No.	B-1	Sample Depth	15'
Sample No.	SH-4	USCS	CL

BEFORE TEST

Initial Moisture Content:	21.09%	
Initial Dry Unit Wt:	107.7	pcf
Initial Total Unit Wt.:	130.4	pcf
Initial Void Ratio:	0.6111	
Initial Degree of Saturation:	96.0%	

AFTER TEST

Final Moisture Content:	17.81%	
Final Dry Unit Wt:	116.1	pcf
Final Total Unit Wt.:	136.7	pcf
Final Void Ratio:	0.4955	
Final Degree of Saturation:	100.0%	

Water Added at: 100 psf

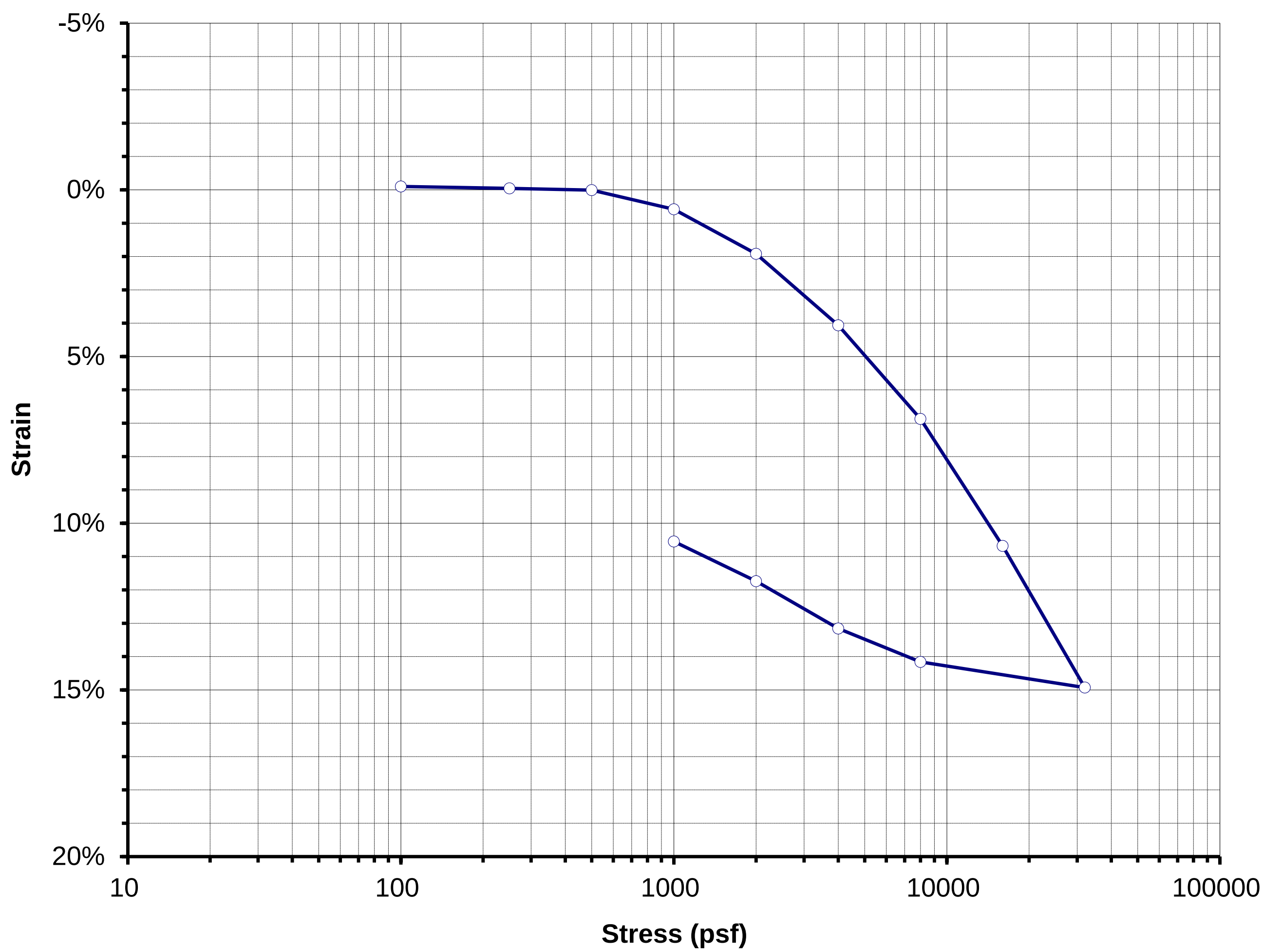
ATTERBERG LIMITS					
LL=	46	PL=	15	PI=	31

Assumed Specific Gravity of Solids, Gs: 2.78

PRESSURE (psf)	SAMPLE STRAIN	VOID RATIO
100	-0.14%	0.613
500	0.10%	0.609
1000	0.43%	0.604
2000	1.34%	0.589
4000	2.95%	0.564
8000	5.08%	0.529
16000	7.66%	0.488
32000	10.74%	0.438
16000	10.29%	0.445
8000	9.49%	0.458
4000	8.58%	0.473
2000	7.83%	0.485
1000	7.22%	0.495

PROJECT NUMBER: IR737	PROJECT NAME: Related Bristol	FIGURE NO. B-2A	GROUP DELTA

CONSOLIDATION TEST RESULTS
ASTM D-2435



Boring No.	B-4	Sample Depth	15'
Sample No.	R-4	USCS	CH

BEFORE TEST

Initial Moisture Content:	30.29%	
Initial Dry Unit Wt:	93.6	pcf
Initial Total Unit Wt.:	121.9	pcf
Initial Void Ratio:	0.8917	
Initial Degree of Saturation:	96.3%	

AFTER TEST

Final Moisture Content:	24.64%	
Final Dry Unit Wt:	104.2	pcf
Final Total Unit Wt.:	129.9	pcf
Final Void Ratio:	0.6987	
Final Degree of Saturation:	100.0%	

Water Added at: 100 psf

ATTERBERG LIMITS					
LL=	56	PL=	22	PI=	34

PRESSURE (psf)	SAMPLE STRAIN	VOID RATIO
100	-0.10%	0.8936
250	-0.04%	0.8926
500	0.01%	0.8915
1000	0.58%	0.8807
2000	1.92%	0.8554
4000	4.06%	0.8149
8000	6.87%	0.7617
16000	10.68%	0.6897
32000	14.93%	0.6093
8000	14.16%	0.6239
4000	13.16%	0.6428
2000	11.74%	0.6696
1000	10.55%	0.6922

Assumed Specific Gravity of Solids, Gs: 2.84

PROJECT NUMBER:	IR737	PROJECT NAME:	Related Bristol	FIGURE NO. B-2B

GROUP DELTA

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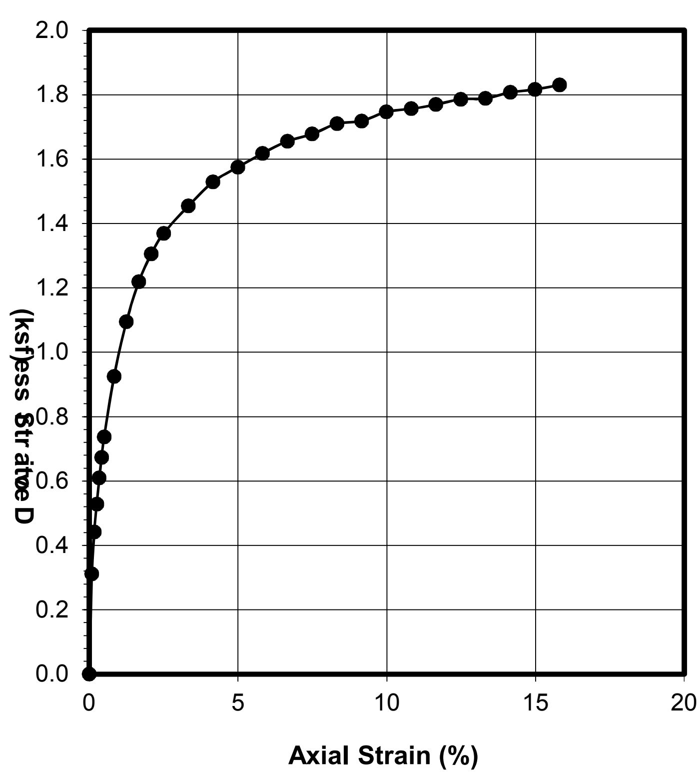
UNCONSOLIDATED UNDRAINED TRIAXIAL TEST (UU,Q)
ASTM D 2850

Client Name:	Group Delta	Tested By:	ST Date: 01/20/21
Project Name:	Related Bristol	Checked by:	AP Date: 01/22/21
Project No.:	IR737		
Boring No.:	B-2		
Sample No.:	SH-4 Depth (feet): 15		
Soil Description	Clay	Sample Type:	Mod. Cal.

Sample Diameter (inch):	2.870	Wet Unit Weight (pcf):	119.4
Sample Height (inch):	6.008	Dry Unit Weight (pcf):	90.5
Sample Weight (g):	1219.62	Moisture Content (%):	31.9
Wt. of Wet Soil+Container (g):	143.00	Void Ratio for Gs=2.7:	0.86
Wt. of Dry Soil+Container (g):	120.86	% Saturation:	100.1
Wt. of Container (g):	51.50		

TEST DATA

Cell Pressure (ksf):	1.80
Back Pressure (ksf):	0.0
Tested Total Confining Pressure (ksf):	1.80
Shear Rate (%/min):	0.3
Maximum Deviator Stress (ksf):	1.83
Ultimate Deviator Stress (ksf):	1.83
Ultimate Undrained Shear Strength (ksf):	0.92
Axial Strain @ Maximum Stress (%)	15.81



Load (lbs)	Def. (inch)	Area (sq.in)	Deviator Stress (ksf)	Axial Strain (%)
0	0.000	6.47	0.00	0.00
14	0.005	6.48	0.31	0.08
20	0.010	6.48	0.44	0.17
24	0.015	6.49	0.53	0.25
28	0.020	6.49	0.61	0.33
30	0.025	6.50	0.67	0.42
33	0.030	6.50	0.74	0.50
42	0.050	6.53	0.92	0.83
50	0.075	6.55	1.09	1.25
56	0.100	6.58	1.22	1.66
60	0.125	6.61	1.31	2.08
63	0.150	6.64	1.37	2.50
68	0.200	6.69	1.45	3.33
72	0.250	6.75	1.53	4.16
75	0.300	6.81	1.58	4.99
77	0.350	6.87	1.62	5.83
80	0.400	6.93	1.66	6.66
82	0.450	6.99	1.68	7.49
84	0.500	7.06	1.71	8.32
85	0.550	7.12	1.72	9.15
87	0.600	7.19	1.75	9.99
89	0.650	7.26	1.76	10.82
90	0.700	7.32	1.77	11.65
92	0.750	7.39	1.79	12.48
93	0.800	7.46	1.79	13.32
95	0.850	7.54	1.81	14.15
96	0.900	7.61	1.82	14.98
98	0.950	7.69	1.83	15.81

Figure: B-3A

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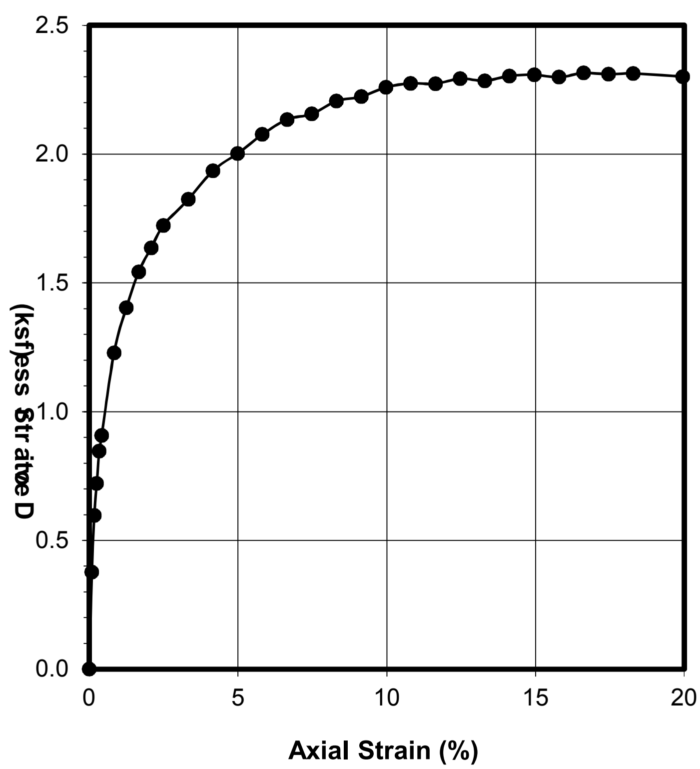
UNCONSOLIDATED UNDRAINED TRIAXIAL TEST (UU,Q)
ASTM D 2850

Client Name:	Group Delta	Tested By:	ST Date: 02/26/20
Project Name:	Related Bristol	Checked by:	AP Date: 02/27/20
Project No.:	IR737		
Boring No.:	B-5		
Sample No.:	R-4 Depth (feet): 15		
Soil Description	Fat Clay	Sample Type:	Mod. Cal.

Sample Diameter (inch):	2.415	Wet Unit Weight (pcf):	117.8
Sample Height (inch):	6.017	Dry Unit Weight (pcf):	87.6
Sample Weight (g):	852.60	Moisture Content (%):	34.5
Wt. of Wet Soil+Container (g):	994.15	Void Ratio for Gs=2.7:	0.92
Wt. of Dry Soil+Container (g):	776.00	% Saturation:	100.8
Wt. of Container (g):	143.21		

TEST DATA

Cell Pressure (ksf):	1.80
Back Pressure (ksf):	0.0
Tested Total Confining Pressure (ksf):	1.80
Shear Rate (%/min):	0.3
Maximum Deviator Stress (ksf):	2.32
Ultimate Deviator Stress (ksf):	2.30
Ultimate Undrained Shear Strength (ksf):	1.15
Axial Strain @ Maximum Stress (%)	16.62



Load (lbs)	Def. (inch)	Area (sq.in)	Deviator Stress (ksf)	Axial Strain (%)
0	0.000	4.58	0.00	0.00
12	0.005	4.58	0.38	0.08
19	0.010	4.59	0.60	0.17
23	0.015	4.59	0.72	0.25
27	0.020	4.59	0.85	0.33
29	0.025	4.60	0.91	0.42
39	0.050	4.62	1.23	0.83
45	0.075	4.64	1.40	1.25
50	0.100	4.66	1.54	1.66
53	0.125	4.68	1.64	2.08
56	0.150	4.70	1.72	2.49
60	0.200	4.74	1.82	3.32
64	0.250	4.78	1.93	4.15
67	0.300	4.82	2.00	4.99
70	0.350	4.86	2.08	5.82
73	0.400	4.91	2.13	6.65
74	0.450	4.95	2.16	7.48
77	0.500	4.99	2.21	8.31
78	0.550	5.04	2.22	9.14
80	0.600	5.09	2.26	9.97
81	0.650	5.13	2.27	10.80
82	0.700	5.18	2.27	11.63
83	0.750	5.23	2.29	12.46
84	0.800	5.28	2.28	13.29
85	0.850	5.33	2.30	14.13
86	0.900	5.38	2.31	14.96
87	0.950	5.44	2.30	15.79
88	1.000	5.49	2.32	16.62
89	1.050	5.55	2.31	17.45
90	1.100	5.60	2.31	18.28
91	1.200	5.72	2.30	19.94

Figure: B-3B

GROUP DELTA

EXPANSION INDEX OF SOIL

ASTM D-4829-10 / UBC 29-2

Lab Number: SO5951

Project Name	: Related Bristol	Sampled By : YW	Date : 1/5/2021
Project No.	: IR737	Prepared By : Eric Y.	Date : 1/14/2021
Boring No.	: B-1	Tested By : Eric Y.	Date : 1/15/2021
Sample No.	: Bulk-1	Calculated By : Eric Y.	Date : 1/19/2021
Depth (ft.)	: 0 - 5	Checked By : Mike G.	Date :
Description	: Dark Gray Fat Clay with Sand		

Sample Preparation

1 1

Weight of Total Soil	3570.40	Weight of Soil Retained on No. 4 Sieve	15.80	% Passing No. 4 Sieve	99.56

Trail	1	2	3	4	Tested	M & D After Test	
Container No.	SB-3					Container No.	
Weight of Wet Soil + Container (gm)	782.28					Wet Soil+Cont.+Ring	
Weight of Dry Soil + Container (gm)	710.95					Dry Soil+Cont.+Ring	
Weight of Container (gm)	232.96					Wt. of Container	
Moisture Content (%)	14.92				14.92	Moisture Content	
Weight of Wet Soil + Ring (gm)	560.11						
Weight of Ring (gm) No. 3.0	200.90				200.90		
Weight of Wet Soil (gm)	359.21						
Wet Density of Soil (pcf)	108.35					Wet Density (pcf)	
Dry Density of Soil (pcf)	94.28					Dry Density (pcf)	
Precent Saturation of Soil $S_{(Meas.)}$	51.14				51.14	(%) Saturation	

Loading Machine No. 3

Date	Reading Time	Elapsed Time	Dial Reading	Expansion
01/15/21	11:10:00	0:10:00		0.0000
01/15/21				
01/15/21	11:20:00	**0:00:00**	**0.3000**	**0.0000**
Add Distilled Water to Sample				
01/15/21	12:20:00	1:00:00	0.3996	0.0996
01/15/21	13:20:00	2:00:00	0.4122	0.1122
01/15/21	14:20:00	3:00:00	0.4135	0.1135
01/15/21	15:20:00	4:00:00	0.4142	0.1142
01/18/21	8:20:00	69 hrs.	0.4183	0.1183
01/18/21	9:20:00	70 hrs.	0.4183	0.1183
01/18/21	10:20:00	71 hrs.	0.4183	0.1183
01/18/21	11:20:00	72 hrs.	0.4183	0.1183

Remark :

1. Screen sample through **No. 4** *Sieve*
2. Sample should be compacted into a metal ring of the Degree of Saturation of **50 + / - 2% (48 - 52).**
3. Inundated sample in distilled water to 24 h, or until the rate of expansion > (0.0002 in./h), no less than 3 h.

Volume of Mold (ft^3)	0.00731	Specific Gravity	2.70
Rammer Weight (lb.)	5.0	Blows/Layer	15
Vertical Confining Pressure	1.0 (lbf/in^2) / 6.9 (kPa)		

$$(\%)\ S = \frac{S.G. \times W \times Dd}{Wd \times S.G. - Dd}$$

S.G.=Specific Gravity, **W**=Water Content
Dd=Dry Soil Density, **Wd**=Unit Wt. of Water

$$E.I._{(meas)} = \frac{\text{Change in High}}{\text{Initial Thickness}} \times 1000 = 118.30$$

$$\text{Expansion Index}_{(50)} = EI_{(meas.)} - (50 - S_{(meas.)}) \times \frac{65 + EI_{(meas.)}}{220 - S_{(meas.)}}$$

120	High

Expansion Index	Potential Expansion
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
> 130	Very High

Figure: B-4A

GROUP DELTA

EXPANSION INDEX OF SOIL

ASTM D-4829-10 / UBC 29-2

Lab Number: SO5652

Project Name	: Related Bristol	Sampled By :		Date :	
Project No.	: IR737	Prepared By :	Eric Y.	Date :	2/20/2020
Boring No.	: B-5	Tested By :	Eric Y.	Date :	2/21/2020
Sample No.	: B-1	Calculated By :	Eric Y.	Date :	2/24/2020
Depth (ft.)	: 0 - 5	Checked By :	Mike G.	Date :	
Description	: Olive Gray Lean Clay with Sand				

Sample Preparation

1 | 1

Weight of Total Soil	3278.60	Weight of Soil Retained on No. 4 Sieve	53.60	% Passing No. 4 Sieve	98.37

Trail	1	2	3	4	Tested	M & D After Test	
Container No.	SB-1					Container No.	
Weight of Wet Soil + Container (gm)	781.94					Wet Soil+Cont.+Ring	
Weight of Dry Soil + Container (gm)	720.72					Dry Soil+Cont.+Ring	
Weight of Container (gm)	234.56					Wt. of Container	
Moisture Content (%)	12.59				12.59	Moisture Content	
Weight of Wet Soil + Ring (gm)	582.54						
Weight of Ring (gm) No. 1.0	202.34				202.34		
Weight of Wet Soil (gm)	380.20						
Wet Density of Soil (pcf)	114.68					Wet Density (pcf)	
Dry Density of Soil (pcf)	101.86					Dry Density (pcf)	
Precent Saturation of Soil $S_{(Meas.)}$	51.92				51.92	(%) Saturation	

Loading Machine No.			1	
Date	Reading Time	Elapsed Time	Dial Reading	Expansion
02/21/20	11:00:00	0:10:00		0.0000
02/21/20				
02/21/20	11:10:00	0:00:00	0.5000	0.0000
Add Distilled Water to Sample				
02/21/20	12:10:00	1:00:00	0.5750	0.0750
02/21/20	13:10:00	2:00:00	0.5790	0.0790
02/21/20	14:10:00	3:00:00	0.5800	0.0800
02/21/20	15:10:00	4:00:00	0.5800	0.0800
02/21/20	16:10:00	5:00:00	0.5810	0.0810
02/24/20	8:10:00	69 Hrs.	0.5830	0.0830
02/24/20	9:10:00	70 Hrs.	0.5830	0.0830
02/24/20	11:10:00	72 Hrs.	0.5830	0.0830

Remark :

1. Screen sample through **No. 4** *Sieve*

2. Sample should be compacted into a metal ring of the Degree of Saturation of ***50 + / - 2% (48 - 52).***

3. Inundated sample in distilled water to 24 h, or until the rate of expansion > (0.0002 in./h), no less than 3 h.

Volume of Mold (ft^3)	0.00731	Specific Gravity	2.70
Rammer Weight (lb.)	5.0	Blows/Layer	15
Vertical Confining Pressure	1.0 (lbf/in^2) / 6.9 (kPa)		

$$(\%)\ S = \frac{S.G. \times W \times Dd}{Wd \times S.G. - Dd}$$

S.G.=Specific Gravity, W=Water Content
Dd=Dry Soil Density, Wd=Unit Wt. of Water

$$E.I._{(meas)} = \frac{\text{Change in High}}{\text{Initial Thickness}} \times 1000 = 83.00$$

$$\text{Expansion Index}_{(50)} = EI_{(meas.)} - (50 - S_{(meas.)}) \times \frac{65 + EI_{(meas.)}}{220 - S_{(meas.)}}$$

85	Medium

Expansion Index	Potential Expansion
0 - 20	*Very Low*
21 - 50	*Low*
51 - 90	*Medium*
91 - 130	*High*
> 130	*Very High*

Figure: B-4B

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CORROSION TEST RESULTS

Client Name: Group Delta
Project Name: Related Bristol
Project No.: IR737

AP Job No.: 20-0243
Date: 02/26/20

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-4	B-1	0-5	Fat Clay	371	7.7	10274	377

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested