GROUP
DELTA

GEOTECHNICAL DESIGN REPORT
FOR THE PROPOSED RESIDENTIAL DEVELOPMENT AT
8572 Talbert Avenue
Fountain Valley, California

Prepared for

BONANNI DEVELOPMENT
5500 Bolsa Avenue, Suite 120
Huntington Beach, CA 92649

Prepared by

GROUP DELTA CONSULTANTS, INC.
32 Mauchly, Suite B
Irvine, California 92618

Group Delta Project No. IR780
December 7, 2021

GROUP DELTA

December 7, 2021

BONANNI DEVELOPMENT
5500 Bolsa Avenue, Suite 120
Huntington Beach, CA 92649

Attention: Chris Segesman

Subject: Geotechnical Design Report
               Proposed Residential Development
               8572 Talbert Avenue
               Fountain Valley, California
               Group Delta Project No. IR780

Dear Mr. Segesman:

Group Delta Consultants, Inc. (Group Delta) is pleased to submit this geotechnical design report for the proposed residential development at 8572 Talbert Avenue in Fountain Valley, California. Our scope of work was performed in general accordance with our proposal dated October 15, 2021, which was authorized on October 18, 2021. The geotechnical investigation represents the current subsurface ground conditions at the site and has been used to develop geotechnical recommendations.

We appreciate the opportunity to provide geotechnical services for this project. If you have any questions about this report, or if we can be of further service, please do not hesitate to contact us.

Sincerely,

GROUP DELTA CONSULTANTS, INC.

Asheesh Pradhan, Ph.D., P.E.   Ethan Tsai, G.E.
Staff Engineer   Associate Engineer

Distribution: Addressee   (PDF file)

32 Mauchly, Suite B, Irvine, CA 92618   TEL: (949) 450-2100
Anaheim – Irvine – Ontario – San Diego – Torrance
www.GroupDelta.com

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

**GEOTECHNICAL DESIGN REPORT
FOR RESIDENTIAL DEVELOPMENT AT
8572 TALBERT AVENUE, FOUNTAIN VALLEY, CALIFORNIA**

1.0   INTRODUCTION

This report presents our recommendations for the foundation design and construction of the proposed residential development located at 8572 Talbert Avenue in Fountain Valley, California.

1.1   Background

The Site is approximately 1.4 acres, located about 250 feet east of the intersection of Newland Street and Talbert Avenue at 8572 Talbert Avenue as shown in Figure 1. The Site is bounded by Talbert Avenue on the north, residential establishments on the west and south, greenbelt area on the east. The site is currently occupied by Courreges Ranch and has few existing single-story residential buildings.

1.2   Project Description

We understand based on a conceptual project layout developed by WHA Architects, the proposed development will consist of demolition of the existing structures and construction of new 15 units of two-story dwellings. The proposed buildings are planned to be at-grade structures. A proposed conceptual site plan is shown in the exploration plan in Figure 2.

1.3   Scope of Work

The objective of this report is to provide foundation recommendations for the design and construction of the proposed residential development at the site. Our scope of work for the site included the following tasks:

- Review of relevant United States Geological Survey (USGS) and California Geological Survey (CGS) maps and reports for the site and surrounding area as well as other geotechnical reports in the vicinity area performed by Group Delta.

- Perform a geotechnical investigation to evaluate subsurface conditions, which includes three (3) Cone Penetration Tests (CPTs) to the depths of about 50 feet below existing ground surface (bgs) and three Hollow Stem Auger (HSA) borings to depths of between 21 feet to 51 feet bgs.

- Perform laboratory testing program on selected soil samples to evaluate characteristics of the soils encountered during our field exploration

- Evaluate limited geologic and seismic hazards including surface fault rupture, liquefaction, and seismically induced landslide.

- Perform engineering analyses and provide geotechnical recommendations for excavations, placement of fill, foundation design, floor slab supports, and any construction-related issues

- Prepare this geotechnical design report

2.0   FIELD INVESTIGATION AND LABORATORY TESTING PROGRAM

Field investigation for the site was performed by Group Delta on October 28, 2021. Laboratory tests were conducted on selected and representative soil samples obtained during our field investigation. A brief description of the field investigation and laboratory testing is provided below and in Appendix A and B respectively.

2.1   Field Exploration

Before beginning the field investigation, well permits were obtained from the city of Fountain Valley. Underground Service Alert (USA) was notified of each exploration location for identifying possible subsurface utilities.

A field investigation was performed at the project site that consisted of three (3) HSA exploratory borings drilled to depths of between 21 and 51 feet bgs and three (3) CPTs advanced to a depth of about 50 feet bgs. The explorations were performed under the direct supervision of a Group Delta's field engineer, who maintained logs of the soils encountered, visually classified the material, and assisted in obtaining soil samples. Bulk samples of soil cuttings from the drilling were collected to depths of about 5 feet bgs. Relatively undisturbed ring samples were retrieved from the borings at about 5 feet depth intervals thereafter.

The locations of our field exploratory borings and CPTs are shown in Figure 2. A brief detail of the field exploration program, the logs of the borings, and CPT plots are presented in Appendix A.

2.2   Laboratory Testing

Laboratory testing was performed on selected soil samples collected from the borings to characterize the subsurface materials and to evaluate their index and engineering properties. The laboratory testing program consisted of the following.

- Soil classification
- Moisture content and dry unit weight
- Atterberg limits
- Direct shear
- Proctor Compaction Test
- Expansion Index
- Soil Corrosivity

The performed tests are identified on the boring logs in Appendix A and laboratory test results are presented in Appendix B.

3.0   SITE AND SUBSURFACE CONDITIONS

3.1   Surface Conditions

The Site is generally flat and has an approximate elevation of 44 to 46 feet. The site is currently occupied by Courreges Ranch and has few existing single-story residential buildings with paved driveways, concrete walkways, and mostly unpaved areas with trees and plantations.

3.2   Subsurface Conditions

The field explorations performed at the site indicate about 2 feet of existing fill. Native soils consist of medium dense to dense silty sand, and sand to a depth of about 18 feet. From the depth of about 18 feet to 25 feet, the soil is mostly interbeddings of very stiff to hard clay, sandy clay, silt, and medium dense to dense silty sand and sand. Below 25 feet to the explored depth of about 51 feet, the soil is predominantly dense to very dense sandy soils with occasional thin layers of clay.

3.3   Groundwater

Groundwater was measured at a depth of 35 feet in the boring B-1, CPT-1 and CPT-3 which is consistent with the mapped historic high groundwater level, as shown in Figure 3, that is about 30 feet bgs (CGS, 1997). Accordingly, the design groundwater level is assumed to be at a depth of 30 feet bgs.

4.0   LIMITED GEOLOGIC AND SEISMIC HAZARDS EVALUATION

4.1   Ground Surface Rupture

The potential hazard for ground rupture is evaluated through consideration of distance to active earthquake faults. The project site is not located within a State-identified Earthquake Fault Zone of Required Investigation (CGS, 1977). The closest active fault is the San Joaquin Hills located about 0.75 miles (1.2 kilometers) southwest of the site, as shown in Figure 4. Therefore, the potential hazard of ground surface rupture at the site is considered low.

4.2   Landslides

The site is not located within an area known for landslide hazards (CGS, 1997). Permanent cut slopes are not anticipated for the proposed improvements. The topography of the site is level. Therefore, the potential of seismically induced landslides is considered low.

4.3 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 loadings, such as those produced by an earthquake. This increase in pore water pressure can temporarily transform the soil into a fluid mass, resulting in a vertical settlement, and can also cause lateral ground deformations. Typically, liquefaction occurs in areas where there are loose to medium dense sands and silts, and where the depth to groundwater is less than 50 feet from the surface.

The site is not located in a mapped liquefaction hazard zone on the California Seismic Hazard Zone Map for Newport Beach 7.5 Minute Quadrangles (CGS, 1999), as shown in Figure 5. However, the site is located adjacent to the border of mapped liquefaction hazard zone.

Liquefaction triggering analyses was performed using simplified procedures recommended by NCEER (Youd and Idriss, 1997, 2001) for CPTs. A peak ground acceleration value for the 2,475-year return period earthquake (PGA$_M$) based on ASCE 7-16 of 0.72g. Moment median magnitude of 7.68 is used in evaluated based on the USGS based on the Dynamic U.S. 2014 (v4.2) deaggregation tool (https://earthquake.usgs.gov/hazards/interactive/). The historical highest groundwater of 30 feet was used in the analyses.

CPT soundings provide a more continuous measurement of the soil profile compared to the coarser sampling intervals in the boring records. We assessed the liquefaction potential using the computer program CLiq (GeoLogismiki, 2015). For estimating the resulting ground settlements, we used the method proposed by Zhang et al (2002 and 2004).

Total liquefaction-induced settlement is estimated to be about 0.6 inches or less. The differential settlement may be assumed to be 0.3 inches over a horizontal distance of about 30 feet.

Liquefaction triggering and liquefaction-induced settlement calculations are provided in Appendix C.

4.4 Lateral Spreading

Under cyclic loading, lateral spreading can occur on gently sloping ground or horizontal ground adjacent to bodies of water or slopes. Since the site is relatively flat and not adjacent to bodies of water or slope, lateral spreading is not a design concern.

5.0 DISCUSSIONS AND RECOMMENDATIONS

5.1 General

Based on the findings from our field explorations and engineering analyses, it is our opinion that the proposed project is feasible from a geotechnical standpoint. Following proper site grading recommendations, the structure can be supported on conventional spread footings with slab-on-grade or mat foundation.

5.2   Demolition

Before the start of earthwork, the existing site improvements will require demolition and removal, including the existing structures and foundations, slabs, hardscapes, utilities, trees, and plantations. The civil engineer should identify the presence and location of all existing utilities on and adjacent to the site. Precautions will be required to remove, relocate or protect existing utilities, as appropriate.

5.3   Removal

Fill soils were encountered at the site to the depth of about 2 feet. Deeper fills could be present anywhere within the site and could locally extend deeper. Any existing fill should be considered uncertified and should be removed and recompacted. In addition, soil loosened or disturbed during the demolition should be removed and recompacted.

To provide uniform support, the building area should be over-excavated and recompacted as structural fill to a minimum depth of 4 feet below existing ground surface to form a pad. All removals should extend a minimum of 4 feet outside the pad or a distance equal to the depth of the excavation, whichever is greater. The actual limits for removals should be determined by the project geotechnical engineer during grading based on the actual conditions encountered.

In non-structural areas such as pavements, concrete walkways, and hardscapes, the area should be over-excavated and recompacted to a minimum depth of 2 feet. The removal should extend a minimum of 2 feet from the edge of the pavements and hardscapes.

5.4   Earthwork

All grading should conform to the requirements of the 2019 California Building Code, and the general grading recommendations outlined below.

1. The grading contractor is responsible for notifying the project geotechnical engineer of a pre-grading meeting prior to the start of excavation/grading operations and any time that the operations are resumed after an interruption.

2. Prior to the start of earthwork, the project civil engineer should locate any existing utilities in the area. Existing utilities should be removed, relocated, or protected, as appropriate.

3. The bottoms of excavations should be proof rolled with heavy equipment. Any areas of loose or pumping soils should be over excavated at the direction of the project geotechnical engineer. For the areas receiving structural fill, after the proof-rolling, the native sandy soils should be scarified, brought to optimum moisture content, and compacted to at least 95 % relative compaction per the maximum dry density determined by ASTM D-1557.

4. Structural fill soil should be compacted to at least 95 % of the maximum dry density determined by ASTM D1557. Fill placed in non-structural areas such as pavements and hardscapes should be compacted to at least 90 % of the maximum dry density.

5. The on-site soils, less any debris or organic matter, may be used in required fills. All structural fill soils should be sandy soils, free of highly expansive clay, organics, debris, rocks greater than 3 inches in any dimension, and other deleterious material. All fill soils shall be approved by the project geotechnical engineer.

6. Any required import material should consist of relatively non-expansive soils with an expansion index of less than 30. The imported materials should contain sufficient fines (binder material) to be relatively impermeable and result in a stable subgrade when compacted. Import soils should be approved before being brought to the site.

7. All earthwork and grading should be performed under the observation of a Group Delta representative. Compaction testing of the fill soils shall be performed at the discretion of Group Delta and per local requirements. If specified compaction is not achieved, additional compaction effort, moisture conditioning of the fill soils, and/or removal and compaction of the below-minimum-compaction soils will be required.

8. Compaction testing of the fill soils shall be performed at the discretion of the project geotechnical engineer. Testing should be performed for approximately every 2 feet of fill thickness, or every 500 cubic yards of fill placed, whichever occurs first. If specified compaction is not achieved, additional compaction effort, moisture conditioning, and/or removal and compaction of the fill soils will be required.

9. All materials used for asphalt concrete and aggregate base shall conform to the 2018 "Green Book" or the equivalent and shall be compacted to at least 95 % relative compaction.

If in the opinion of the geotechnical engineer, contractor, or owner, an unsafe condition is created or encountered during grading, all work in the area shall be stopped until measures can be taken to mitigate the unsafe condition. An unsafe condition shall be considered any condition that creates a danger to workers, on-site structures, on-site construction, or any off-site properties or persons.

5.5   Temporary Excavations

Based on our investigation, we anticipate the excavations will be made in generally sandy materials and should be readily accomplished using conventional heavy construction equipment. The sandy soils on site are generally classified as CAL/OSHA Type C soils.

If the excavation is exposed during periods of rainfall, provisions for the collection of the runoff should be made. All surface drainage should be controlled and prevented from running down into the excavation. Ponding water should not be allowed within the excavation.

All excavation slopes and shoring systems should meet minimum requirements of the Occupational Safety and Health (OSHA) Standards. Maintaining safe and stable slopes on excavations is the responsibility of the contractor and will depend on the nature of the soils and groundwater conditions encountered and his method of excavation. Excavations during construction should be carried out in such a manner that failure or ground movement will not occur. The short-term stability of excavation depends on many factors, including slope angle, engineering characteristics of the subsurface materials, height of the excavation, and length of time the excavation remains unsupported and exposed to equipment vibrations, rainfall, and desiccation. The contractor should perform any additional studies deemed necessary to supplement the information contained in this report for the purpose of planning and executing his excavation plan. Recommendations regarding sloped temporary excavations are provided in the sections below.

Temporary excavations up to 4 feet deep may stand in vertical cuts. Deeper temporary excavations should be sloped at 1 (horizontal) to 1 (vertical), or flatter.

Surcharge loads from equipment or stockpiled material should be kept behind the top of the temporary excavations a horizontal distance of at least twice the depth of the excavation. Surface drainage should be controlled and prevented from running down the slope face. Ponding water should not be allowed within the excavation. Even with the implementation of the above recommendations, some sloughing of slopes and unstable soil zones may still occur within temporary excavations, and workmen should be adequately protected. Construction equipment and foot traffic should be kept off excavation slopes to minimize disturbance/sloughing.

5.6   Foundations Recommendations

5.6.1   Spread Footings

5.6.1.1   Bearing Capacity

Following proper site grading, the new building may be supported on shallow foundations placed in the new structural fill. Spread footings extending at least 2 feet below the floor slab or lowest adjacent grade, may be designed to impose a net dead-plus-live load pressure of 3,000 pounds per square foot. The allowable bearing pressure can be increased by one-third for temporary

loads associated with wind and seismic loading. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

All foundation excavations should be observed by the project geotechnical engineer before the placement of reinforcing steel. Any loose or soft soil found should be excavated and replaced with structural fill.

5.6.1.2   Settlement

We estimate the static settlement of the structure supported on spread footings in the manner recommended is expected to be less than ½ inch. As discussed in Section 4.3 of this report, the seismic-induced settlement is estimated to be 0.6 inches or less. Therefore, the total settlement is estimated to be 1.1 inches or less, The differential settlement is estimated to be ½ inch over a horizontal distance of 30 feet.

5.6.1.3   Lateral Resistance

Resistance to lateral loads can be provided by friction developed between the bottom of the footings and the supporting soil, and by the passive soil pressure developed on the face of the footing. For design purposes, an allowable coefficient of friction of 0.4 may be used. The allowable passive pressure for compacted structural fill may be taken as an equivalent fluid pressure of 300 pcf.

The recommended lateral load design values stated above are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values may be multiplied by a factor of 1.5. In addition, the passive pressure and coefficient of friction may be used in combination without reduction.

5.6.2   Slab on Grade

On-grade building slabs should be supported by compacted fill having an EI of 30 or less. Slabs should be designed for the anticipated loading. Slab thickness, control joints, and reinforcement should be designed by the project structural engineer and should conform to the requirements of the 2019 CBC and with the recommendations of the American Concrete Institute.

American Concrete Institute (2004) provides detailed recommendations regarding moisture protection systems. The project architect should review ACI 302.1R-04 along with the moisture requirements of the proposed flooring system and specify an appropriate moisture protection system based on the allowable moisture transmission rate of the flooring to be used. The vapor membrane should conform to ASTM E 1745 guidelines. According to ACI 302-1R-04, moisture protection may consist of 10 to 15 mil polyethylene plastic sheeting over the compacted subgrade. If necessary, 2 inches of a granular base material may be placed beneath the polyethylene plastic sheeting to provide a level surface. The granular base material should be a

clean, finely graded material with at least 10 to 30 percent passing the No. 100 sieve but not contaminated with clay, silt, or organic material. The granular material should be compacted before the placement of the vapor membrane. The vapor membrane should be protected from puncture and if damaged be repaired per the manufacturer's recommendations.

5.7   Seismic Design Parameters per CBC 2019/ASCE 7-16

Seismic design acceleration parameters were developed per the 2019 California Building Code (CBC) and ASCE 7-16 (ASCE/SEI 7-16) for the proposed project using the webtool provided by the Office of Statewide Health Planning and Development (OSHPD, 2021) and are presented in Table 1. Based on the underlying geology, subsurface exploration data, and shear wave velocity estimates from the CPTs, the site classification for seismic design is Site Class D per Chapter 20 of ASCE 7-16. The site coordinates used in our seismic hazard analysis are -117.98362 (Longitude) and 33.855614 (Latitude).

Table 1: Mapped Seismic Design Acceleration Parameters

Design Parameters	General Seismic Design Parameter (ASCE 7-16 Section 11.4)
S_s (g)	1.391
S_1 (g)	0.504
Site Class	D
F_a	1.0
F_v	1.796
S_{MS} (g)	1.391
S_{M1} (g)	0.905
S_{DS} (g)	0.927
S_{D1} (g)	0.603

Based on Section 11.4.8 of ASCE 7-16, if desired, these values may only be used if Exception 2 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 $T \geq 1.5\ T_S$: 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.

GROUP DELTA

5.8   Utility Trenches

Excavations for utility trenches should be readily accomplished with conventional excavating equipment. All shoring and excavation should comply with current OSHA regulations and be observed by the designated competent person on site.

The bedding for any new sewer and water service pipelines should be a minimum of 4 inches thick and should consist of clean sand, No. 4 concrete aggregate or gravel, and should have a sand equivalent of not less than 30. The pipe zone material, which extends to a level 12 inches above the pipe should consist of sand and should have a sand equivalent of no less than 30, and a maximum rock size of 1 inch. All imported materials should be approved by the project geotechnical engineer before being brought on site.

Trench zone backfill extends from a level 12 inches above the pipe to the finished subgrade. In general, on-site excavated materials are suitable as backfill. Any boulders or cobbles larger than 3 inches in any dimensions, or any organics or other deleterious materials, should be removed before backfilling. We recommend that all backfill should be placed in lifts not exceeding six to eight inches in thickness and be compacted to at least 90 % of relative compaction as determined by the ASTM D1557. Mechanical compaction will be required to accomplish compaction above the bedding along the entire pipeline alignments. Jetting or flooding of backfill should not be permitted.

In backfill areas, where mechanical compaction of soil backfill is impractical due to space constraints, 2-sack slurry (CLSM) may be substituted for compacted backfill.

5.9   Expansive Soils

Laboratory testing was performed on a representative sample of the near-surface material collected to the depth of 5 feet below the existing ground surface from boring B-1. The test result indicates low expansion potential, and the test result is presented in Appendix B.

5.10   Soil Corrosion Potential

A representative sample of the near-surface material collected to the depth of 5 feet below the existing ground surface from boring B-3 was tested for evaluating corrosion characteristics. The results indicate the test sample had a pH of 8.3; a water-soluble sulfate content of less than 0.01%, and a soluble chloride content of less than 0.01%. The sulfate results indicate that sulfate exposure to Portland cement is negligible.

The test sample was also found to have a minimum measured electrical resistivity of 6,800 Ohm-cm. The following correlation 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 laboratory test results, the test sample is classified as moderately corrosive to buried metals. Further evaluation/testing and recommendations for corrosion protection should be provided by a corrosion consultant.

5.11   Site Drainage

Surface drainage during construction should be controlled and directed to appropriate drainage facilities. All surface drainage should be prevented from running down along the face of the excavation. Ponding water should not be allowed within the excavation.

5.12   Pavement Design

Parking and parking access pavements may be supported on compacted fill following proper grading recommendations in Sections 5.3 and 5.4. Any loose soils should be removed and recompacted. The actual limits for subgrade preparation, removals, or compaction should be determined by the project geotechnical engineer during grading, based on the actual conditions encountered.

The preparation of the paving area subgrade should be done immediately before the placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

The paving thicknesses are based on our observations of the on-site soils conditions. An R-value of 30 was assumed for design. The R-value should be confirmed during grading.

5.12.1 Flexible Asphalt Concrete Pavement

Asphalt Concrete (AC) pavement recommendations for a 20-year design life per the Caltrans Highway Design Manual (Caltrans 2020) for Traffic Index (TI) values of 4, 5, and 6 are provided in Table 2.

Table 2. Asphalt Concrete Pavement Sections

Traffic Index	AC Pavement Thickness (inches)	Class II Aggregate Base Thickness (inches)
4	3	4
5	3.5	4.5
6	4	6.5

We recommend that the Civil Engineer select an appropriate design TI based on anticipated vehicular loading.

5.12.2 Rigid Concrete Pavement

Rigid concrete pavement thickness recommendation is based on a 20-year design life per Portland Cement Association (PCA). The required concrete pavement thickness will depend on the expected wheel loads and volume of traffic. The pavement sections were determined for the axle-load category of 1 for residential streets for sandy subgrade soils providing medium support. Assuming the modulus of rupture of 600 pounds per square inch (psi) and that the paving subgrade will consist of the on-site or comparable soils compacted to at least 90 % as recommended, the minimum recommended concrete thicknesses are presented in Table 3 below. The base thickness should be a minimum of 4 inches.

Table 3. Rigid Concrete Pavement Sections

Traffic Index	Concrete (inches)
4	5.5
5	5.5
6	6.0

For concrete entrance driveways, we recommend a pavement section of 6 inches of PCC over 6 inches of aggregate base (CAB or CMB).

Crack control joints should be constructed for all PCC slabs on a maximum spacing of 12 feet, each way.

5.12.3 Concrete Walkway and Hardscape

Concrete slabs for walkways and hardscapes can be supported on the compacted subgrade soil following recommendations in Section 5.4. Slabs should be designed for the anticipated loading. We recommend that such slabs be at least 4 inches thick and reinforced with at least 6-inch by 6-inch, W2.9 by W2.9 welded wire fabric placed at slab mid-height with crack control joints with a maximum spacing of 5-foot centers each way for sidewalks and 10-foot centers each way for

GROUP DELTA

slabs. Actual crack control joint spacing should be designed by the project structural engineer or architect in conformance with the requirements of the 2019 CBC and the PCA Engineering Bulletin for Concrete Floors on Ground.

6.0   STORMWATER INFILTRATION

The design groundwater table is at depth of 30 feet below the existing ground surface. Since the invert of the new stormwater infiltration facility must be at least 10 feet above the design groundwater, the target layer for stormwater infiltration at the site is between depths of 0 and 20 feet below existing ground.

Infiltrate rates of the onsite soils were estimated based on laboratory grain size analyses. Details of test procedures and recommended infiltration rate for design are provided below.

6.1   Laboratory Grain Size Analyses

Laboratory grain size analyses have been used to evaluate infiltration rate for the property as it is not larger than 50,000 square feet. We performed laboratory grain size tests on three selected samples. The results of the laboratory tests are presented in Appendix B.

Infiltration rate may be estimated based on the hydraulic conductivity of the soils. We have estimated the soil hydraulic conductivity using Hazen equation as shown below:

$$K = C \times (D_{10})^2$$

Where K   = Hydraulic Conductivity in cm/s
      C   = 1, Hazen's Empirical Coefficient
      D_{10} = Grain Diameter in mm Corresponding to 10% Passing

The computed hydraulic conductivity may then be assumed to be the infiltration rate of the soils.

The design infiltration rate can be determined by the computed percolation rate divided by a reduction factor (RF_t). We recommend that the following RF_t may be used for design infiltration rate:

- Laboratory Grain Size Analyses: RF_t = 2
- Site variability, number of tests, and thoroughness of subsurface investigation: RF_t = 2
- Long-term siltation, plugging, and maintenance: RF_t = 2

Accordingly, we recommend that a reduction of 6 may be used in design. A The infiltration rate based on grain-size analyses are shown in Table 4.

Table 4. Infiltration Rate based on Laboratory Grain Size Analyses

Sample No.	Soil Type	D_{10} (mm)	Computed Infiltration Rate (inch/hour)	Design Infiltration Rate (inch/hour)
B-1@5'	Sandy Silt	0.002	<0.1	<0.1
B-1@10'	Poorly Graded Sand	0.19	0.5	<0.1
B-2@10'	Silty Sand	0.002	<0.1	<0.1

Since the design infiltration rate is less than 0.3 inch per hour, therefore, we recommend that the site soils are not suitable for stormwater infiltration.

7.0   LIMITATIONS

This investigation was performed per 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 the Bonanni Development 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 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 differs 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

American Society for Testing and Materials, ASTM International (ASTM), 2008, Annual Book of Standards, Volumes 4.08 and 4.09, Soil and Rock.

California Building Code (CBC), 2019, published by International Code Council, Inc., Illinois, California.

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 003.

California Geological Survey (CGS), 1999, Earthquake Zones of Required Investigation, Seal Beach Quadrangle, Orange County, California, 1999.

California Geological Survey (CGS), 1999, Earthquake Zones of Required Investigation, Newport Beach Quadrangle, Orange County, California, 1998.

Caltrans, (2020), Caltrans Highway Design Manual.

Geologismiki, CLiq Version 2.0.6.97, CPT Liquefaction Assessment Software, 2006.

Office of Statewide Health Planning and Development (OSHPD), U.S. Seismic Design Maps, [Online] https://seismicmaps.org/ (accessed November 3, 2021).

Portland Cement Association, (1984), Thickness Design for Concrete Highway and Street Pavement, EB109P.

United States Geological Survey (USGS), 2019, Unified Hazard Tool, [Online] https://earthquake.usgs.gov/hazards/interactive/ (accessed November 3, 2021)

United States Geological Survey (USGS), Quaternary Faults in Google Earth, [Online] http://earthquake.usgs.gov/hazards/qfaults/google.php (accessed November 3, 2021)

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J., Liao, S., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R., and Stokoe, K.H., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 127, October, pp 817-833.

Zhang, G., Robertson. P.K., Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180.

WESTMINSTER

MIDWAY CITY

WESTMINSTER

GARDEN GROVE

Westminster Memorial Park

Golden West College

Smeltzer

FOUNTAIN VALLEY

The Good Shepherd Cem and Mausoleum

Lake Huntington

PROJECT SITE

HUNTINGTON BEACH

HUNTINGTON BEACH

Huntington
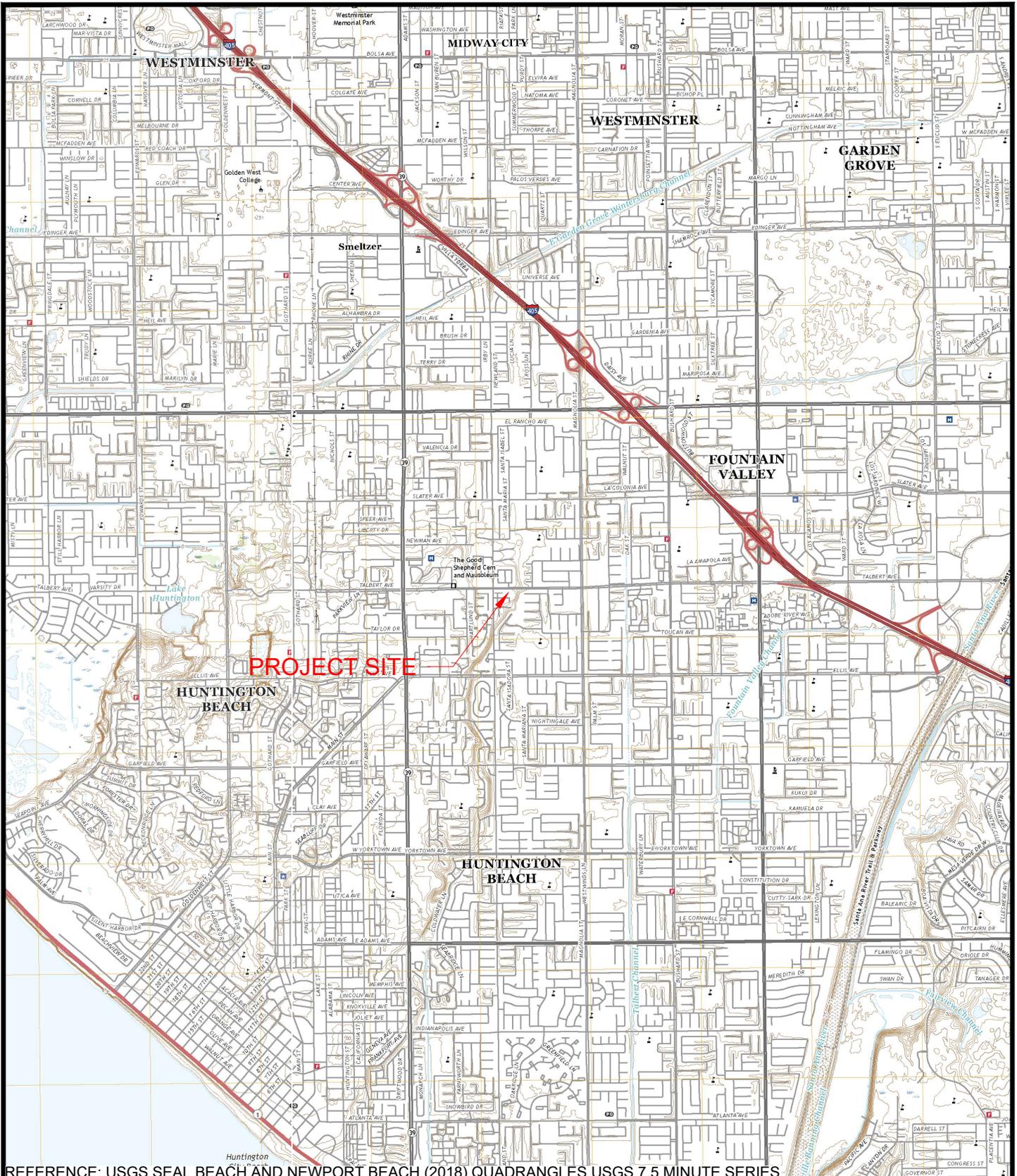
REFERENCE: USGS SEAL BEACH AND NEWPORT BEACH (2018) QUADRANGLES USGS 7.5 MINUTE SERIES

0	4000'	8000'

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
32 MAUCHLY, SUITE B
IRVINE, CALIFORNIA (949) 450-2100

FIGURE NUMBER:
1

PREPARED BY: JMT	PROJECT NAME: 8572 TALBERT AVE. FOUNTAIN VALLEY, CA	PROJECT NUMBER: IR780
REVIEWED BY: AP	SITE VICINITY MAP	

TALBERT AVE

CPT-3
B-3
CPT-2
B-2
CPT-1
B-1

LEGEND:
B-1   APPROXIMATE BORING LOCATION
CPT-1 APPROXIMATE CPT LOCATION

N

0   50'   100'

REFERENCE: GOOGLE EARTH, IMAGERY DATE: 01/22/2020

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
32 MAUCHLY, SUITE B
IRVINE, CALIFORNIA (949) 450-2100

PREPARED BY:
JMT

REVIEWED BY:
AP

PROJECT NAME:
8572 TALBERT AVE.
FOUNTAIN VALLEY, CA

GROUP DELTA

FIGURE NUMBER:
2

PROJECT NUMBER:
IR780

EXPLORATION LOCATION PLAN

PROJECT SITE
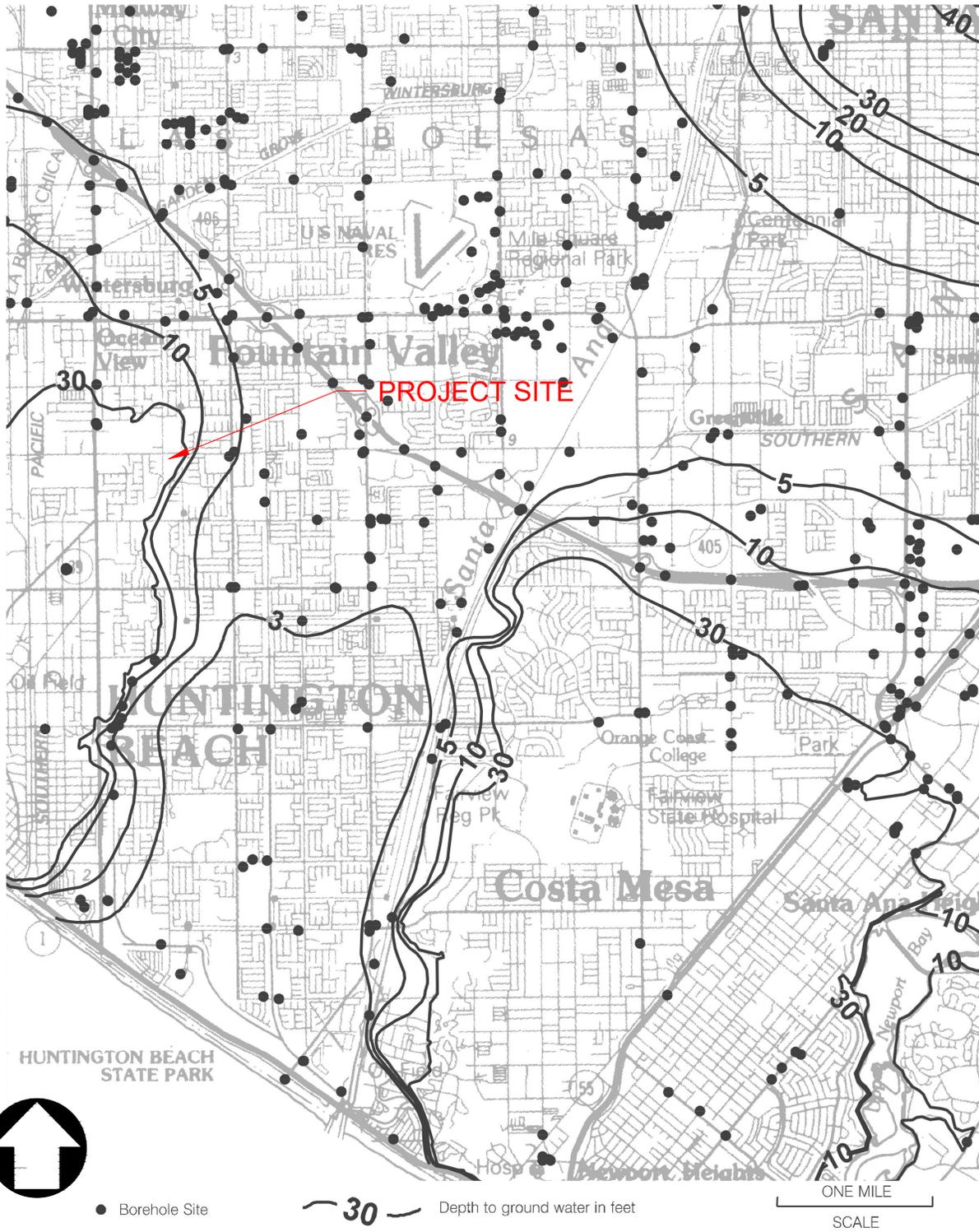
Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, Newport Beach Quadrangle.

● Borehole Site

— 30 — Depth to ground water in feet

ONE MILE

SCALE

REFERENCE: CGS, 1998, SEISMIC HAZARD ZONE
REPORT, PLATE 1.2 GROUND WATER, FOR ANAHEIM
AND NEWPORT BEACH (SHZR 003) 7.5-MINUTE
QUADRANGLE, LOS ANGELES, CALIFORNIA.

GROUP
DELTA

GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CALIFORNIA (949) 450-2100		FIGURE NUMBER: 3
PREPARED BY: JMT	PROJECT NAME: 8572 TALBERT AVE. FOUNTAIN VALLEY, CA	PROJECT NUMBER: IR780
REVIEWED BY: AP	HISTORICALLY HIGHEST GROUNDWATER MAP	

REFERENCE: USGS Quatenary Faults, Fault Sources https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a6b0aadf88412fcf

11/4/2021, 7:00:15 AM

Fault Areas

Class B
historic
late Quaternary
latest Quaternary
middle and late Quaternary

National Database

— Historic (< 150 years), well constrained location

┈┈ Historic (< 150 years), moderately constrained location
····· Historic (< 150 years), inferred location
┈┈ Latest Quaternary (<15,000 years), well constrained location
┈┈ Latest Quaternary (<15,000 years), moderately constrained location
····· Latest Quaternary (<15,000 years), inferred location
— Late Quaternary (< 130,000 years), well constrained location
┈┈ Late Quaternary (< 130,000 years), moderately constrained location

— Late Quaternary (< 130,000 years), inferred location
— Middle and late Quaternary (< 750,000 years), well constrained location
┈┈ Middle and late Quaternary (< 750,000 years), moderately constrained location
····· Middle and late Quaternary (< 750,000 years), inferred location
— Undifferentiated Quaternary (< 1.6 million years), well constrained location
┈┈ Undifferentiated Quaternary (< 1.6 million years), moderately constrained location
····· Undifferentiated Quaternary (< 1.6 million years), inferred location

1:288,895

0   2.75   5.5   11 mi
0   4.5   9   18 km

USGS, National Geographic, Esri, Garmin, HERE, UNEP-WCMC,
USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, increment P
Corp.

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
32 MAUCHLY, SUITE B
IRVINE, CALIFORNIA (949) 450-2100

PREPARED BY: JMT

REVIEWED BY: AP

PROJECT NAME:
8572 TALBERT AVE.
FOUNTAIN VALLEY, CA

REGIONAL FAULT MAP

FIGURE NUMBER:
4

PROJECT NUMBER:
IR780

GROUP DELTA

MAP EXPLANATION
EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

1906
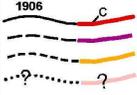
SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Note: Mitigation methods differ for each zone –
AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

PROJECT SITE
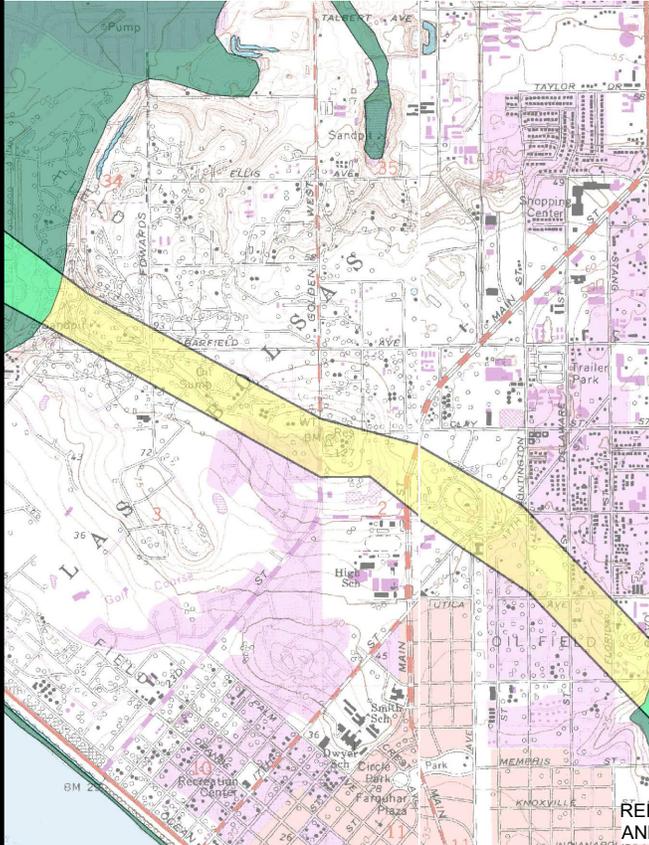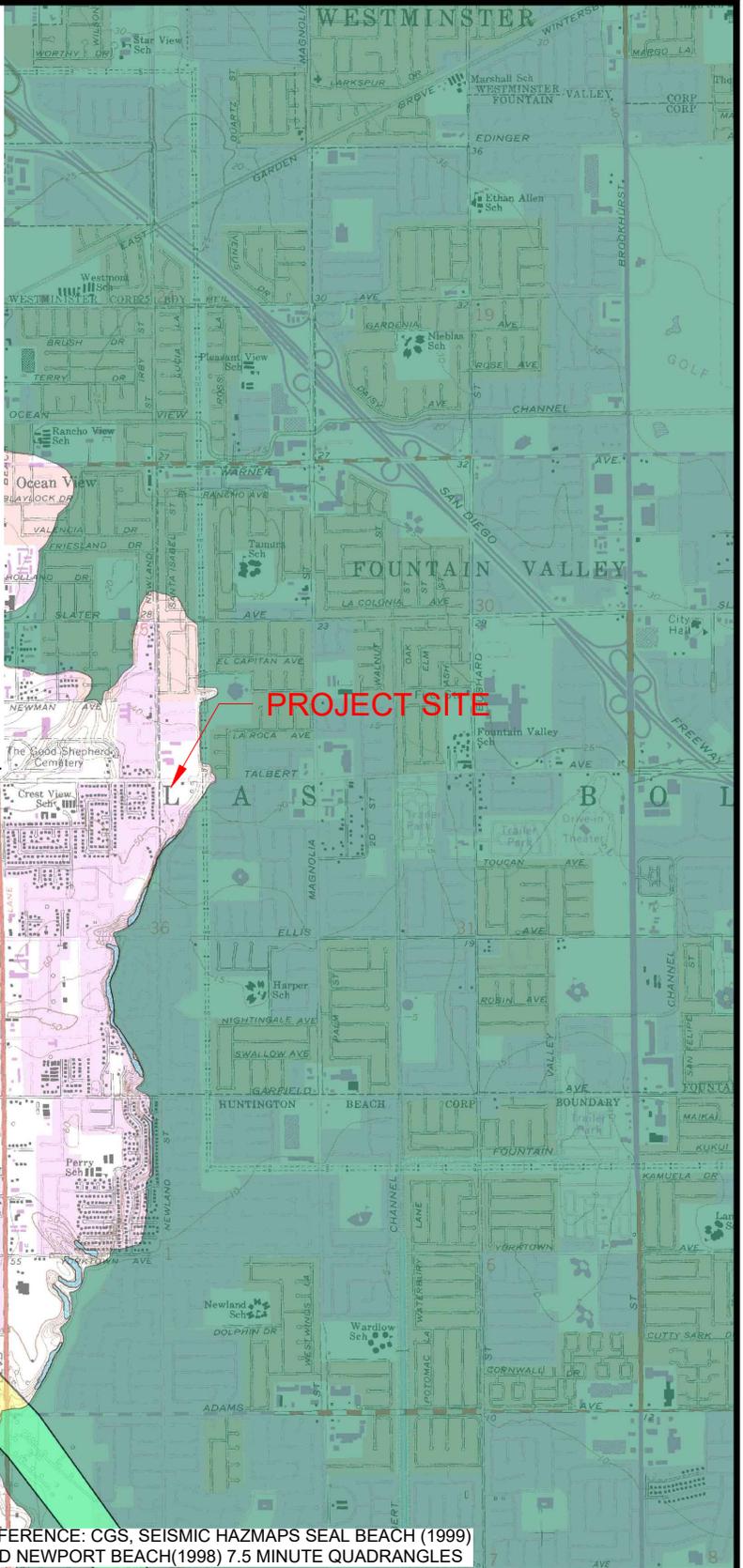
REFERENCE: CGS, SEISMIC HAZMAPS SEAL BEACH (1999) AND NEWPORT BEACH(1998) 7.5 MINUTE QUADRANGLES

0   3000'   6000'

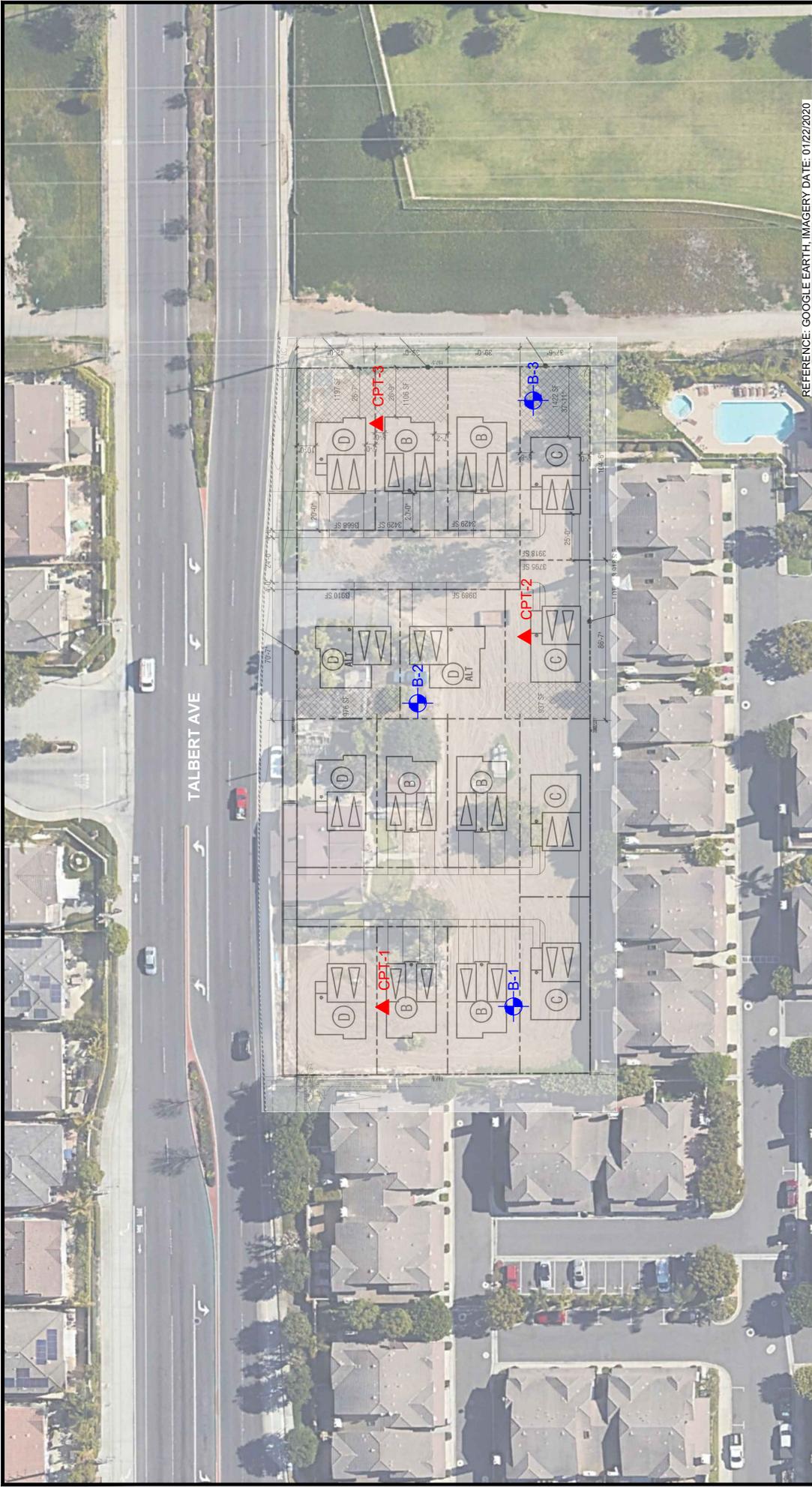| GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CALIFORNIA (949) 450-2100	FIGURE NUMBER: 5
	PREPARED BY: JMT	PROJECT NAME: 8572 TALBERT AVE. FOUNTAIN VALLEY, CA	PROJECT NUMBER: IR780
	REVIEWED BY: AP	EARTHQUAKES ZONES OF REQUIRED INVESTIGATION MAP

APPENDIX A
FIELD INVESTIGATION

APPENDIX A
FIELD INVESTIGATION

A.1   Introduction

The subsurface conditions at the site for the proposed residential development at 8572 Talbert Avenue in Fountain Valley, California were investigated by performing three hollow stem auger (HAS) borings and three Cone Penetration Tests (CPTs) on October 28 of 2021. The locations of the explorations are presented in Figure 2 of the main report. A summary of field explorations is presented in Table A-1.

Before beginning the exploration program, access permission and drilling permits were obtained as necessary from the city of Fountain Valley and property owners. Any available subsurface utility maps were reviewed before selecting locations for subsurface investigations. Underground Service Alert (USA) was notified and each exploration location was cleared for underground utilities. The exploration methods are described in the following sections.

A.2   Soil Drilling and Sampling

Drilling, Logging, and Soil Classification

HSA borings were performed by Group Delta's drilling subcontractor ABC Liovin Drilling under the continuous technical supervision of a Group Delta field engineer, who visually inspected the soil samples, measured groundwater levels, maintained detailed records of the borings, and visually / manually classified the soils per the ASTM D2488 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-2 through Figure A-4.

Sampling

Bulk samples of soil cuttings were collected at depths of up to 5 feet 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 per ASTM D1586 and Ring-Lined "California" Split Barrel samplers per ASTM D3550.

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.

Relatively undisturbed 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 the 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.

At each sampling interval, the drive samplers were fitted onto the 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.

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 the 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 lb 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 lb * 30 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 inches, 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, energy correction factor equal to Hammer Efficiency (%) / 60% was applied to the field blowcounts. Hammer efficiency was determined by Pile Driving Analyzer (PDA) measurement.

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

Borings	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
B-1; B-2; B-3	CME Auto	140# 30"	1.0	79.3	1.32	D_o=3.0" D_i=2.42"	0.67	1.32	0.88

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 the relative density of granular/cohesionless soils. Consistency classification of cohesive soils was based on undrained shear strength estimated by correlations to Equivalent SPT N_{60}. The correlations for consistency and relative density are shown in the Boring Record Legend in Figure A-1C. Drive sample field blow counts, SPT N^*_{60} values, and corresponding density/consistency classifications are presented on the boring records.

Borehole Abandonment

After the drilling, the borings were abandoned by backfilling the borehole with cement grout as indicated on the records. Excess cuttings were placed in 55-gallon drums, sampled and tested for contaminants, temporarily stored at an approved location, and legally disposed of off-site. Notes describing the borehole abandonment are presented at the bottom of the 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 per the Unified Soil Classification System following ASTM D2487 and ASTM D2488. Field classifications presented on the records were modified where necessary based on 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 Testing and Engineering performed three CPT soundings to depths of 50 feet below the existing ground surface. The CPTs were conducted per ASTM D5778 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 are presented in Figures A-5 to A-7. After the sounding, the apparent groundwater depth and cave-in depth were measured with weighted tape and the CPT hole was abandoned by backfilling cement grout into the hole.

A.4 List of Attached Tables and Figures

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

List of Tables

Table A-1   Summary of Field Explorations

List of Figures

Figures A-1A through A-1C   Boring Record Legend
Figures A-1D and A-1E   Key for Soil Classification
Figures A-2 through A-4   Boring Record
Figures A-5 through A-7   CPT Records and Interpretations

Table A-1: Summary of Field Explorations

Exploration No.	Date	Type	Depth (feet)	Groundwater Depth (feet)
B-1	10/28/2021	HSA	51.5	35
B-2	10/28/2021	HSA	31.5	NE
B-3	10/28/2021	HSA	21.5	NE
CPT-1	10/28/2021	CPT	50.4	35
CPT-2	10/28/2021	CPT	50.1	Cave in at 32 feet dry
CPT-3	10/28/2021	CPT	50.2	35

SOIL IDENTIFICATION AND DESCRIPTION SEQUENCE

Sequence		Refer to Section		Required	Optional
		Field	Lab		
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.	FIGURE NUMBER
GEOTECHNICAL ENGINEERS AND GEOLOGISTS	**A-1A**
PROJECT NAME	PROJECT NUMBER
Geotechnical Design Report 8572 Talbert Avenue, Fountain Valley, California	IR780

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: Geotechnical Design Report, 8572 Talbert Avenue, Fountain Valley, California

PROJECT NUMBER: IR780

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}.

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

Plasticity Index(PI) vs **Liquid Limit (LL)** chart with "U"-line and "A"-line; zones labeled CL-ML, ML or OL, CL or OL, CH or OH, MH or OH.

Classification of Fine-Grained Soil

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	None to slow	None to slow	Low to medium	Low to medium
CH	High to very high	None	High	High

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

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.

Reference:
ASTM D 2487 and 2488

Granular Soil Gradation Parameters

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

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

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 $1 \le C_c \le 3$	
GW............$C_u > 4$ and $1 \le C_c \le 3$	
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	

Geotechnical Design Report
8572 Talbert Avenue, Fountain Valley
California

Group Delta Project No. IR-780

KEY FOR SOIL CLASSIFICATION
Figure A-1E

GROUP DELTA

Figure #2

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Bonanni - 8572 Talbert Avenue	IR780	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
8572 Talbert Avenue, Fountain Valley, California	10/28/2021	10/28/2021	1 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 75	Hollow Stem Auger	G. Valdivia	Ethan Tsai

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	51.5	46	▽ 35.0 / 11.0 DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
Bulk; SPT (1.4"); MC (2.4")	$N_{60} = 1.32\ N_{SPT} = 0.88 N_{MC}$	▼ 35.0 / 11.0

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
	45		B-1									EI #200			**FILL** SANDY SILT (ML); soft; reddish brown; some fine SAND; trace GRAVEL. FINES 64 %; SAND 36 %. **NATIVE** SANDY SILT (ML); reddish brown; some fine SAND; trace GRAVEL.
5	40		S-2	9 14 10	24	32									SILTY SAND (SM); dense; brown; dry; mostly fine SAND; some fines.
10	35		R-3	8 14 17	31	27			2.1	105.9					Poorly-graded SAND (SP); medium dense; reddish brown; moist; mostly fine; some medium to coarse.
15	30		S-4	8 6 7	13	17									SANDY SILT (ML); medium stiff to stiff; yellowish brown; moist; little fine SAND; low plasticity.
20	25		R-5	6 9 12	21	18			26.4	98.8		PI			SANDY lean CLAY (CL); hard; brown; moist; mostly fines; little fine SAND; medium plasticity.
25	20		S-6	8 11 17	28	37									CLAYEY SAND (SC) to SILTY SAND (SM); dense; brown; dry; mostly fine SAND; some fines.

FIGURE

A-2 a

GDC_LOG_BORING_2016 IR780 - BONANNI - 8572 TALBERT AVE.GPJ GDC2013.GDT 11/29/21

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Bonanni - 8572 Talbert Avenue	IR780	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
8572 Talbert Avenue, Fountain Valley, California	10/28/2021	10/28/2021	2 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 75	Hollow Stem Auger	G. Valdivia	Ethan Tsai

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	51.5	46	▽ 35.0 / 11.0 DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
Bulk; SPT (1.4"); MC (2.4")	$N_{60} = 1.32$ $N_{SPT} = 0.88 N_{MC}$	▼ 35.0 / 11.0

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	15		R-7	13 26 29	55	48			7.7	103.8					SILTY SAND (SM); dense; brown; moist; mostly fine SAND.
35	10		S-8	6 7 6	13	17									SILTY SAND (SM); medium dense; brown; moist to wet; mostly fine SAND; some fines.
40	5		R-9	7 3 7	10	9			15.0	113.0					Poorly-graded SAND (SP); loose; brown; wet; mostly coarse.
45	0		S-10	9 23 50	73	96									Very dense.
50	-5		R-11	10 19 30	49	43			18.3	105.1					Dense.
55	-10														Boring was terminated at 51.5 feet below the existing ground surface (bgs). Groundwater was encountered at 35 feet bgs. Backfilled with cement grout.

GROUP DELTA CONSULTANTS
32 Mauchly, Suite B
Irvine, CA 92618

FIGURE

A-2 b

GDC_LOG_BORING_2016 IR780 - BONANNI - 8572 TALBERT AVE.GPJ GDC2013.GDT 11/29/21

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Bonanni - 8572 Talbert Avenue	IR780	**B-2**

SITE LOCATION	START	FINISH	SHEET NO.
8572 Talbert Avenue, Fountain Valley, California	10/28/2021	10/28/2021	1 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 75	Hollow Stem Auger	G. Valdivia	Ethan Tsai

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	31.5	45	∇ NE / NE DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
Bulk; SPT (1.4"); MC (2.4")	$N_{60} = 1.32\ N_{SPT} = 0.88 N_{MC}$	$\blacktriangledown$ NE / 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
			B-1									CP			**FILL** SANDY SILT (ML); soft; brown; dry; some fine SAND.
															NATIVE SANDY SILT (ML); brown; dry; some fine SAND.
5	40		R-2	13 18 22	40	35			7.9	114.5		DS			SILTY SAND (SM); dense; brown; moist; mostly fine SAND; some fines.
10	35		S-3	7 9 10	19	25									Medium dense.
15	30		R-4	6 12 13	25	22			7.3	93.8					SANDY SILT (ML); very stiff; yellowish brown; moist; little fine SAND; low plasticity.
20	25		S-5	5 12 12	24	32									Yellowish light brown.
25	20		R-6	13 19 28	47	41			12.2	121					CLAYEY SAND (SC); medium dense; brown; moist; mostly medium to fine SAND.

GROUP DELTA CONSULTANTS
32 Mauchly, Suite B
Irvine, CA 92618

FIGURE

A-3 a

GDC_LOG_BORING_2016 IR780 - BONANNI - 8572 TALBERT AVE.GPJ GDC2013.GDT 11/29/21

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Bonanni - 8572 Talbert Avenue	IR780	**B-2**

SITE LOCATION	START	FINISH	SHEET NO.
8572 Talbert Avenue, Fountain Valley, California	10/28/2021	10/28/2021	2 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 75	Hollow Stem Auger	G. Valdivia	Ethan Tsai

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	31.5	45	▽ NE / *NE*	**DURING DRILLING**

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	**AFTER DRILLING**
Bulk; SPT (1.4"); MC (2.4")	$N_{60} = 1.32$ $N_{SPT} = 0.88 N_{MC}$	▼ NE / *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
		✕	S-7	8 19 22	41	54									SILTY SAND (SM); very dense; brown; moist; mostly fine SAND; little fines.
															Boring was terminated at 31.5 feet below the existing ground surface (bgs). Groundwater was not encountered. Backfilled and tampered with soil cuttings.
35	10														
40	5														
45	0														
50	-5														
55	-10														

GDC_LOG_BORING_2016 IR780 - BONANNI - 8572 TALBERT AVE.GPJ GDC2013.GDT 11/29/21

FIGURE

A-3 b

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Bonanni - 8572 Talbert Avenue	IR780	**B-3**

SITE LOCATION	START	FINISH	SHEET NO.
8572 Talbert Avenue, Fountain Valley, California	10/28/2021	10/28/2021	1 of 1

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Liovin	CME 75	Hollow Stem Auger	G. Valdivia	Ethan Tsai

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	21.5	44	▽ NE / NE	DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
Bulk; SPT (1.4"); MC (2.4")	$N_{60} = 1.32 N_{SPT} = 0.88 N_{MC}$	▼ NE / 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
			B-1									CR			**FILL** — SANDY SILT (ML); soft; brown; dry; some fine SAND.
	40														**NATIVE** — SANDY SILT (ML); brown; dry; some fine SAND.
5			R-2	4 / 21 / 27	48	42			8.1	114.3					SILTY SAND (SM); dense; brown; moist; mostly fine SAND; some fines.
	35														
10			S-3	8 / 8 / 10	18	24									Poorly-graded SAND (SP); medium dense; yellowish brown; moist; mostly medium to fine; few fines.
	30														
15			R-4	12 / 19 / 20	39	34									Dense. No recovery.
	25		S-5	10 / 14 / 18	32	42									Dense; mostly coarse to medium.
20			S-6	6 / 9 / 13	22	29									SANDY SILT (ML); very stiff; light brown; moist; little fine SAND; low plasticity.
	20														Boring was terminated at 21.5 feet below the existing ground surface (bgs). Groundwater was not encountered. Backfilled and tampered with soil cuttings.
25															
	15														

GDC_LOG_BORING_2016 IR780 - BONANNI - 8572 TALBERT AVE.GPJ GDC2013.GDT 11/29/21

GROUP DELTA CONSULTANTS
32 Mauchly, Suite B
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-4

This software is licensed to: Group Delta Consultants, Inc.

CPT name: CPT-1

CPT basic interpretation plots

Cone resistance

qt (tsf)

Depth (ft)

Friction Ratio

Rf (%)

Depth (ft)

Pore pressure

u (psi)

Insitu

Depth (ft)

SBT Plot

Ic(SBT)

Depth (ft)

Soil Behaviour Type

SBT (Robertson et al. 1986)

Depth (ft)

Clay
Sand & silty sand
Very dense/stiff soil
Very dense/stiff soil
Very dense/stiff soil
Sand & silty sand
Very dense/stiff soil
Silty sand & sandy silt
Clay
Sand & silty sand
Clay & silty clay
Clay & silty clay
Clay & silty clay
Clay
Silty sand & sandy silt
Very dense/stiff soil
Silty sand & sandy silt
Silty sand & sandy silt
Silty sand & sandy silt
Very dense/stiff soil
Clay & silty clay
Silty sand & sandy silt
Very dense/stiff soil
Clay & silty clay
Sand & silty sand
Silty sand & sandy silt
Clay
Clay
Clay & silty clay
Clay
Sand & silty sand
Sand & silty sand
Sand
Sand & silty sand
Sand & silty sand
Sand & silty sand

Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K₀ applied:	Yes
Earthquake magnitude Mw:	7.68	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.72	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CLiq v.3.3.1.14 - CPT Liquefaction Assessment Software - Report created on: 11/3/2021, 2:24:42 PM
Project file: \\192.168.100.6\Files$\Projects_AV\1700\1R780 Bonanni - 8572 Talbert Avenue\05_Field\CPT\1R780.clq

Figure 5

CPT basic interpretation plots

Cone resistance

qt (tsf)

Friction Ratio

Rf (%)

Pore pressure

u (psi)

SBT Plot

Ic(SBT)

Soil Behaviour Type

SBT (Robertson et al. 1986)

Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft
Fines correction method:	NCEER (1998)	Average results interval:	1
Points to test:	Based on Ic value	Ic cut-off value:	2.60
Earthquake magnitude M_w:	7.68	Unit weight calculation:	Based on SBT
Peak ground acceleration:	0.72	Use fill:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A

Fill weight:	N/A
Transition detect. applied:	Yes
K_σ applied:	Yes
Clay like behavior applied:	Sands only
Limit depth applied:	No
Limit depth:	N/A

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CLiq v.3.3.1.14 - CPT Liquefaction Assessment Software - Report created on: 11/3/2021, 2:24:43 PM
Project file: \\192.168.100.6\Files$\Projects_AV\I700\IR780 Bonanni - 8572 Talbert Avenue\05_Field\CPT\IR780.clq

Figure 6

This software is licensed to: Group Delta Consultants, Inc.

CPT name: CPT-3

CPT basic interpretation plots

Cone resistance

Depth (ft)

qt (tsf)

0 200 400 600

Friction Ratio

Depth (ft)

Rf (%)

0 2 4 6 8 10

Pore pressure

Depth (ft)

Insitu

u (psi)

0 2 4 6 8

SBT Plot

Depth (ft)

Ic(SBT)

1 2 3 4

Soil Behaviour Type

Depth (ft)

Organic soil
Sand & silty sand

Very dense/stiff soil

Sand & silty sand

Silty sand & sandy silt
Sand & silty sand

Silty sand & sandy silt
Silty sand & sandy silt

Sand & silty sand

Silty sand & sandy silt
Silty sand & sandy silt
Very dense/stiff soil
Clay & silty clay
Silty sand & sandy silt
Silty sand & sandy silt
Clay & silty clay
Very dense/stiff soil

Sand & silty sand

Silty sand & sandy silt
Silty sand & sandy silt
Silty sand & sandy silt
Sand & silty sand

Clay

Sand & silty sand

Silty sand & sandy silt
Clay & silty clay
Clay

SBT (Robertson et al. 1986)

0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft
Fines correction method:	NCEER (1998)	Average results interval:	1
Points to test:	Based on Ic value	Ic cut-off value:	2.60
Earthquake magnitude M$_w$:	7.68	Unit weight calculation:	Based on SBT
Peak ground acceleration:	0.72	Use fill:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A
		Fill weight:	N/A
		Transition detect. applied:	Yes
		K$_\sigma$ applied:	Yes
		Clay like behavior applied:	Sands only
		Limit depth applied:	No
		Limit depth:	N/A

CLiq v.3.3.1.14 - CPT Liquefaction Assessment Software - Report created on: 11/3/2021, 2:24:44 PM
Project file: \\192.168.100.6\Files$\Projects_AV\I700\IR780 Bonanni - 8572 Talbert Avenue\05_Field\CPT\IR780.clq

Figure 7

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

B.1   General

The laboratory testing was performed using the 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 D2487) and Visual Manual (ASTM D2488)
- Moisture content (ASTM D2216) and Dry Unit Weight (ASTM D2937)
- Atterberg Limits (ASTM D4318)
- Direct Shear (ASTM D3080)
- Proctor Compaction Test (ASTM D1557)
- Expansion Index (ASTM D4829)
- Soil Corrosivity:
    - pH (ASTM G51)
    - Water-Soluble Sulfate (ASTM D4327)
    - Water-Soluble Chloride (ASTM D4327)
    - Minimum Electrical Resistivity (ASTM G187)

A summary of laboratory tests performed is presented in Table B-1. 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 per the Unified Soil Classification System (USCS) following ASTM D2488; soil classifications were modified as necessary based on testing in the laboratory per ASTM D2487. 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

GROUP
DELTA

The in-situ moisture content of selected bulk, SPT, and Ring samples was determined by oven drying in general accordance with ASTM D2216. 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, the volume and weight of the metal containers were measured, and moisture content and dry density were calculated in general accordance with ASTM D2216 and D2937. 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 per ASTM D4318. Result of the test is presented on the boring records in Appendix A, and is plotted on plasticity chart in Figure B-1 of this Appendix.

B.5   Direct Shear

The direct shear test was as performed on a selected sample per ASTM D3080. After the initial weight and volume measurements were made, the sample was placed in a calibrated shear machine and a selected normal load was applied. The sample was then saturated and allowed to consolidate, and then was sheared under a constant strain to failure. Shear stress and sample deformations were monitored throughout the test. The test result is presented in Figure B-2.

B.6   Proctor Compaction Test

Proctor compaction test was performed on the selected sample per ASTM D1557. The sample was tested for dry density at different water content after compaction to obtain optimum moisture content that would result in maximum dry density with the standard compacting effort. The test result is presented in Figure B-3.

B.7   Expansion Index

The expansion potential of the site soil was estimated using the Expansion Index Test per ASTM D4829. The result of the test is presented in Figure B-4, and on the boring records in Appendix A.

B.8   Soil Corrosivity

Tests were performed to determine the corrosion potential of site soils on concrete and ferrous metals. Corrosivity testing included minimum electrical resistivity (ASTM G187), soil pH (ASTM G51), and water-soluble chlorides sulfates (ASTM D4327). The test results are

GROUP DELTA

\\192.168.100.6\Files$\Projects_AV\I700\IR780 Bonanni - 8572 Talbert Avenue\07_Reports\Appendices\Appendix B\IR771- Appendix B Text.docx

discussed in Section 5.10 of the main report and presented in Figures B-6a and B-6b of this appendix.

B.9   List of Attached Figures

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

List of Table

List of Figures

Table B-1: Summary of Laborator Testing Performed

Boring No.	Sample No.	Depth (feet)	Moisture (M)	Unit Weight (UW)	Passing #200 (%)	Atterberg (PI)	Direct Shear (DS)	Expansion (EI)	Proctor Compaction (CP)	Corrosion (CR)
B-1	B-1	0-5			X			X		
	S-2	5								
	R-3	10	X	X						
	S-4	15								
	R-5	20	X	X		X				
	S-6	25								
	R-7	30	X	X						
	S-8	35								
	R-9	40	X	X						
	S-10	45								
	R-11	50	X	X						
B-2	B-1	0-5							X	
	R-2	5					X			
	S-3	10								
	R-4	15	X	X						
	S-5	20								
	R-6	25	X	X						
	S-7	30								
B-3	B-1	0-5								X
	R-2	5	X	X						
	S-3	10								
	R-4	15								
	S-5	17.5								
	S-6	20								

ATTERBERG LIMITS

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

Project Name: *Bonnani - 8572 Talbert Avenue*
Project No. : *IR780*
Boring No.: *B-1*
Sample No. : *R-5*
Initial Moisture:
Description.: *Olive Brown Sandy Silty Clay - CL*

Tested By : *Eric Y.*   Date: *11/05/21*
Data Input By: *Eric Y.*   Date: *11/08/21*
Checked By: *Asheesh P.*   Date:
Depth (ft.) : *20*
Container No.: *AL-1*

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			32	24	17	
Container No.	A	B	C	D	E	
Wet Wt. of Soil + Cont. (gm.)	22.57	22.26	27.78	28.64	29.31	
Dry Wt. of Soil + Cont. (gm.)	21.26	20.98	24.01	24.56	24.80	
Wt. of Container (gm.)	15.28	15.16	15.24	15.38	15.01	
Moisture Content (%) [Wn]	21.91	21.99	42.99	44.44	46.07	

LIQUID LIMIT **44**
PLASTIC LIMIT **22**
PLASTICITY INDEX **22**

PI at "A" - Line = 0.73(LL-20) = **17.5**

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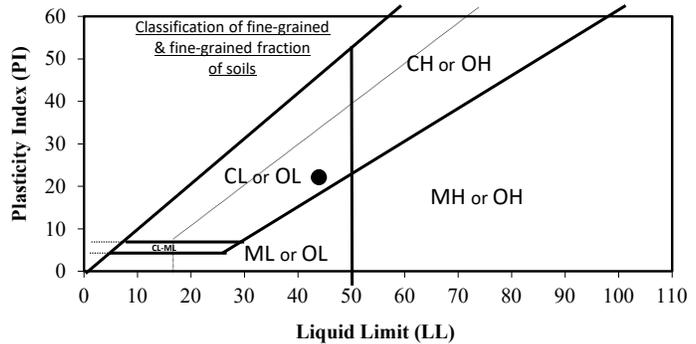
PROCEDURES USED

[ ] Wet Preparation
Multipoint Wet Preparation

[x] Dry Preparation
Multipoint Dry Preparation

[x] Procedure A
Multipoint Test

[ ] Procedure B
One-point Test
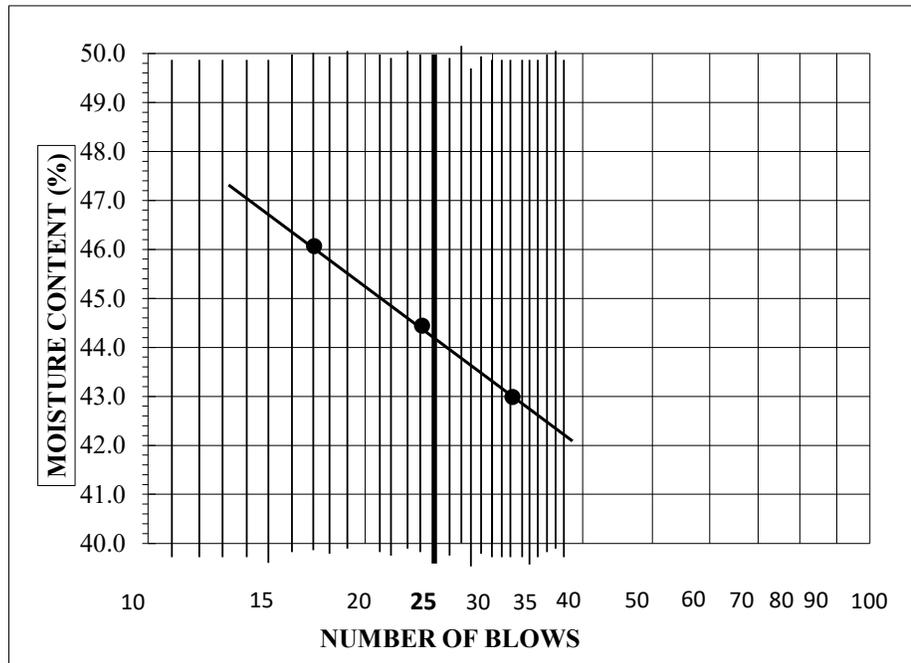
GROUP DELTA
GROUP DELTA CONSULTANTS
1320 South Simpson Circle
Anaheim, CA 92806
(714) 660-7500 office
(714) 660-7550 fax

Figure B-1
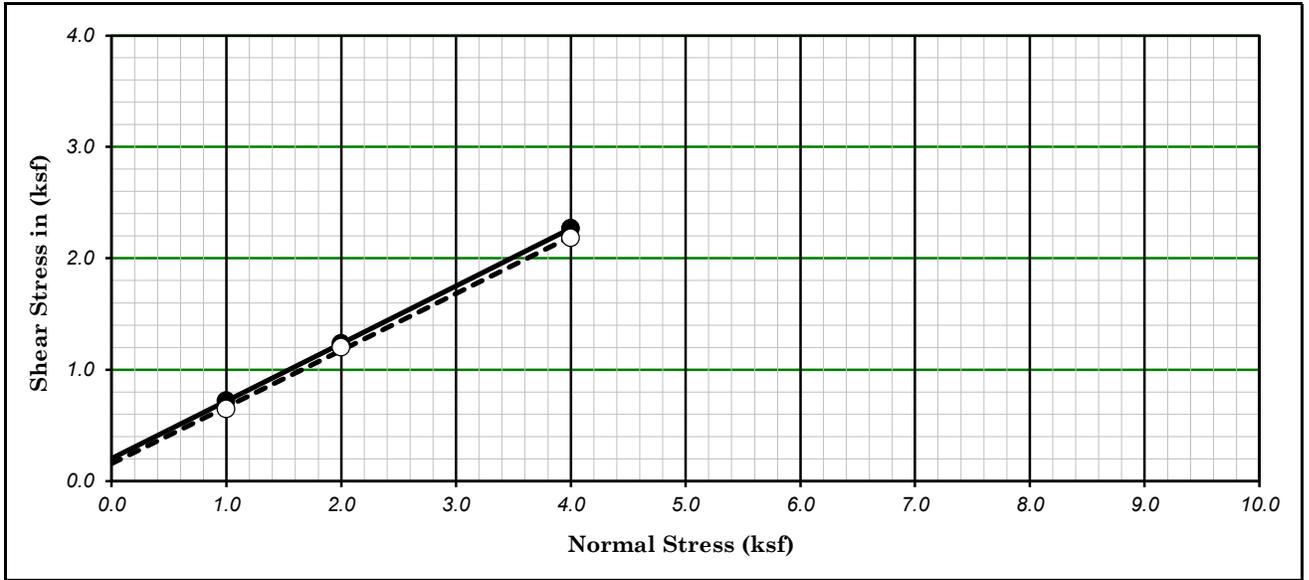
Ultimate : ○ | Shear Type : | *Saturated* | *Undisturbed* | Peak : ●
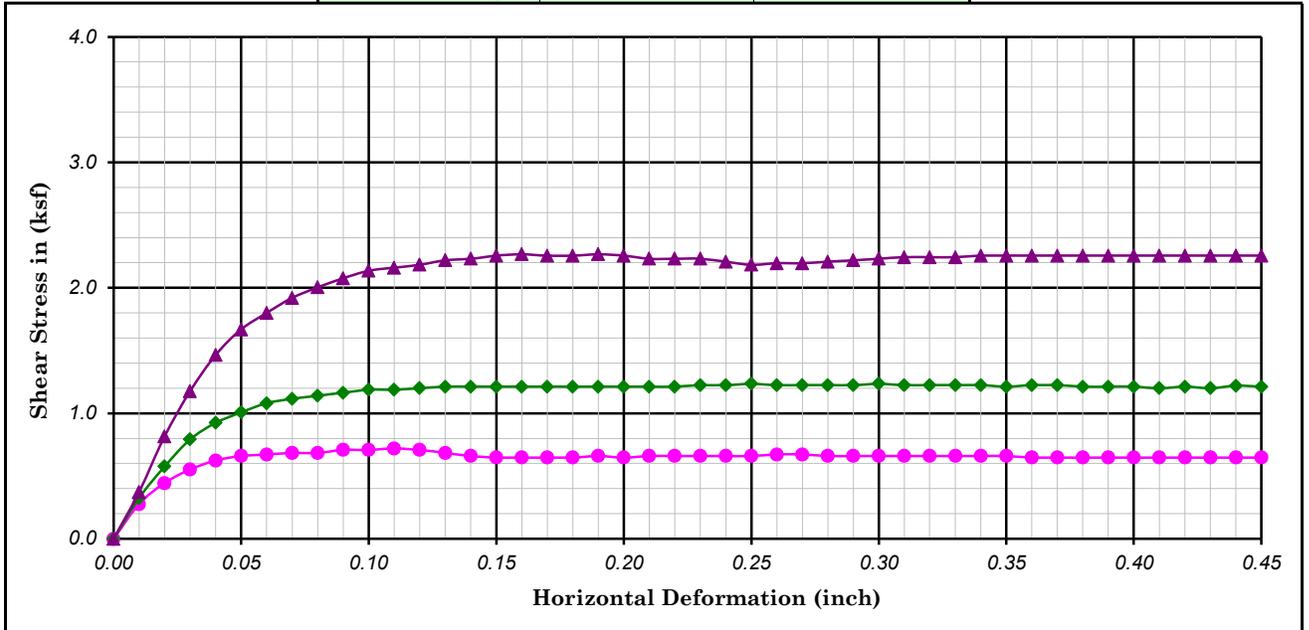
Boring No.	: *B-2*	Strength Intercept (C) :	*0.20*	(ksf)		*0.16*	(ksf)	
Sample No.	: *R-2*		*9.77*	(kPa)	**Peak**	*7.47*	(kPa)	**Ultimate**
Depth (ft/m)	: *5.0* | *1.53*	Friction Angle (φ) :	*27.29*	Degree		*26.98*	Degree	

Description : *Brown Silty Sand / Sandy Silt*   **Shear Rate** (inch/minute) : *0.0004*

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m³)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	*16.48*	*115.96*	*18.25*	*0.45*	*1.00*	*47.88*	*0.72*	*34.47*	*0.65*	*31.03*
◆	*16.56*	*116.78*	*18.38*	*0.44*	*2.00*	*95.76*	*1.24*	*59.18*	*1.20*	*57.46*
▲	*16.62*	*119.14*	*18.75*	*0.41*	*4.00*	*191.52*	*2.27*	*108.59*	*2.18*	*104.57*

GROUP DELTA

Bonnani - 8572 Talbert Avenue

DIRECT SHEAR TEST
(ASTM D -3080)

Project No. : *IR780*   **Date :** *11/08/21*

Figure B-2

STANDARD TEST METHOD FOR
MOISTURE - DENSITY RELATIONSHIP
(ASTM D1557)

REV. 1, DATED 09/19/19

PROJECT:	Bonnani - 8572 Talbert Avenue	SAMPLE ID:	SO6250
PROJECT NO.:	IR780	DATE:	November 4, 2021
TESTED BY:	Eric Y.	CHECKED BY:	Asheesh P.

SAMPLE DESCRIPTION: Brown Sandy Silt with Sandstone/Siltstone

LOCATION: B-2 Bulk-1 @ 0 - 5'

Method: Mechanical ☑ Manual ☐   **Hammer:** 10lb. ☑ 5.5 lb. ☐

A) WATER ADDED	0	2	4	6			%
B) MOLD TARE WEIGHT	1931.0	1931.0	1931.0	1931.0			grams
C) WEIGHT OF WET SOIL AND MOLD	3908.0	4040.6	4050.4	3974.6			grams
D) WET SOIL WEIGHT (C - B)	1977.0	2109.6	2119.4	2043.6			grams
E) WET DENSITY (D / V)	130.6	139.3	140.0	135.0			pcf
F) DRY DENSITY (E / [(L/100) + 1])	123.2	128.8	127.2	120.5			pcf

G) TARE WEIGHT	235.5	225.2	233.7	229.5			grams
H) WEIGHT OF WET SOIL AND TARE	1178.0	1333.5	1340.2	1308.0			grams
I) WEIGHT OF DRY SOIL AND TARE	1124.6	1249.7	1239.0	1192.5			grams
J) WEIGHT OF WATER (H - I)	53.4	83.8	101.2	115.5			grams
K) DRY WEIGHT OF SOIL (I - G)	889.1	1024.5	1005.3	963.0			grams
L) MOISTURE CONTENT (J / K)* 100）	6.0	8.2	10.1	12.0			percent
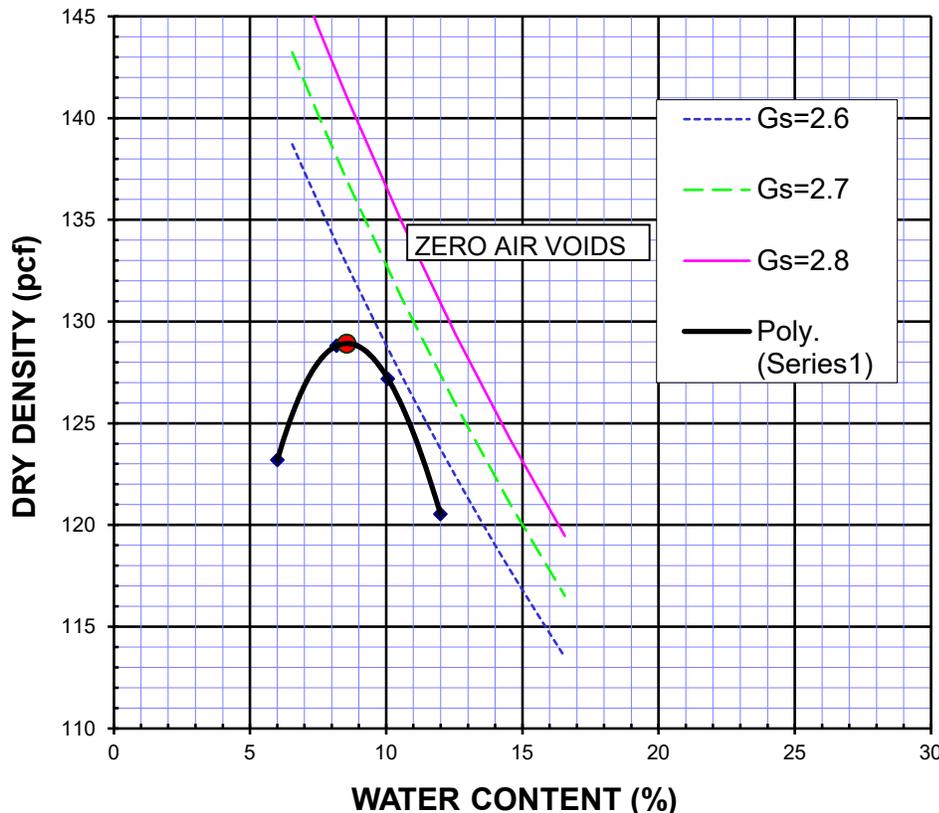
4 inch: V= 15.14 pcf/gm
6 inch: V= 33.98 pcf/gm

A	METHOD USED (A,B or C)

4 inch	MOLD USED
15.14	MOLD VOLUME CORRECTION (V)
# 4	SIEVE NUMBER
0.04%	PERCENT RETAINED

WITH ROCK CORRECTION

	MAXIMUM DENSITY [PCF]
	OPTIMUM MOISTURE [%]

WITHOUT ROCK CORRECTION

	MAXIMUM DENSITY [PCF]
128.9	
8.6	OPTIMUM MOISTURE [%]

Figure B-3

EXPANSION INDEX OF SOIL
ASTM D-4829-10 / UBC 29-2

GROUP DELTA

Lab Number: *SO6250*

Project Name	: *Bonnani - 8572 Talbert Avenue*	**Sampled By :** *G.V.*	**Date :** *10/28/2021*
Project No.	: *IR780*	**Prepared By :** *Eric Y.*	**Date :** *11/3/2021*
Boring No.	: *B-1*	**Tested By :** *Eric Y.*	**Date :** *11/4/2021*
Sample No.	: *Bulk-1*	**Calculated By :** *Eric Y.*	**Date :** *11/8/2021*
Depth (ft.)	: *0 - 5*	**Checked By :** *Asheesh P*	**Date :**
Description	: *Brown Sandy Silt with Sandstone and traces of Gravel*		

Sample Preparation

Weight of Total Soil	3185.20	Weight of Soil Retained on No. 4 Sieve	5.00	% Passing No. 4 Sieve	99.84

Trail	1	2	3	4	Tested	M & D After Test	
Container No.	SB-2					Container No.	
Weight of Wet Soil + Container (gm)	879.10					W_{et} S_{oil}+$C_{ont.}$+R_{ing}	
Weight of Dry Soil + Container (gm)	828.60					D_{ry} S_{oil}+$C_{ont.}$+R_{ing}	
Weight of Container (gm)	228.60					Wt. of Container	
Moisture Content (%)	8.42				8.42	Moisture Content	
Weight of Wet Soil + Ring (gm)	616.05						
Weight of Ring (gm) No. 2.0	198.68				198.68		
Weight of Wet Soil (gm)	417.37						
Wet Density of Soil (pcf)	125.90					Wet Density (pcf)	
Dry Density of Soil (pcf)	116.12					Dry Density (pcf)	
Precent Saturation of Soil $S_{(Meas.)}$	50.32				50.32	(%) Saturation	

Loading Machine No.				**2**

Date	Reading Time	Elapsed Time	Dial Reading	Expansion
11/04/21	10:10:00	0:10:00		0.0000
11/04/21				
11/04/21	10:20:00	0:00:00	0.3000	0.0000
Add Distilled Water to Sample				
11/04/21	11:20:00	1:00:00	0.3195	0.0195
11/04/21	12:20:00	2:00:00	0.3203	0.0203
11/04/21	13:20:00	3:00:00	0.3205	0.0205
11/04/21	14:20:00	4:00:00	0.3026	0.0026
11/04/21	15:20:00	5:00:00	0.3207	0.0207
11/04/21	16:20:00	6:00:00	0.3208	0.0208
11/05/21	7:20:00	21:00:00	0.3213	0.0213
11/05/21	8:20:00	22:00:00	0.3213	0.0213
11/05/21	9:20:00	23:00:00	0.3213	0.0213
11/05/21	10:20:00	0:00:00	0.3213	0.0213

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 (ft³)	0.00731	Specific Gravity	2.70
Rammer Weight (lb.)	5.0	Blows/Layer	15
Vertical Confining Pressure		1.0 (lbf/in²) / 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

$$E.I._{(meas)} = \frac{\text{Change in High}}{\text{Initial Thickness}} \times 1000 = \quad 21.30$$

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

21	**Low**

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

Figure B-4

GROUP DELTA

SOIL CLASSIFICATION

SAMPLE B-1
SAMPLE NUMBER: R-3
SAMPLE DEPTH: 10'

UNIFIED SOIL CLASSIFICATION: SP

DESCRIPTION: POORLY GRADED SAND

ATTERBERG LIMITS
LIQUID LIMIT: 0
PLASTIC LIMIT: 0
PLASTICITY INDEX: 0

Laboratory No. SO6250
Project No. IR780
FIGURE B-1.1

U.S. Standard Sieve Sizes

Percent Finer by Weight

Grain Size in Millimeters

←**4% Gravel**

92% Sand ↔

4% Fines→

| COARSE | FINE | COARSE | MEDIUM | FINE | SILT AND |
| GRAVEL | | SAND | | | CLAY |

U.S. Standard Sieve Sizes

Grain Size in Millimeters

Percent Finer by Weight

	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
	GRAVEL		SAND			CLAY

←0% Gravel

45% Sand →

55% Fines →

SAMPLE B-1

SAMPLE NUMBER: S-2
SAMPLE DEPTH: 5'

UNIFIED SOIL CLASSIFICATION: ML

DESCRIPTION: SANDY SILT

ATTERBERG LIMITS
LIQUID LIMIT: 0
PLASTIC LIMIT: 0
PLASTICITY INDEX: 0
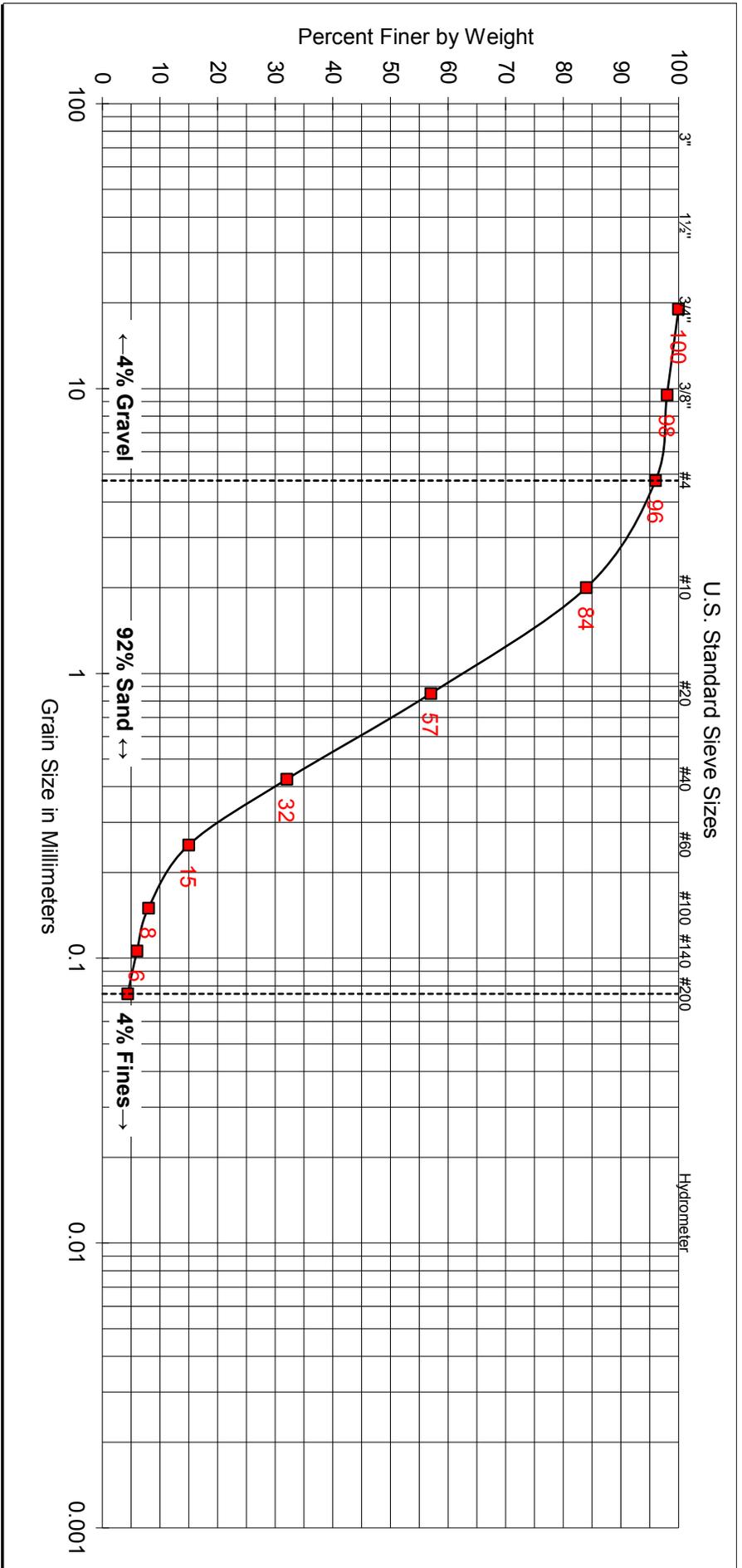
GROUP DELTA

SOIL CLASSIFICATION

Laboratory No. SO6250
Project No. IR780
FIGURE B-1.1
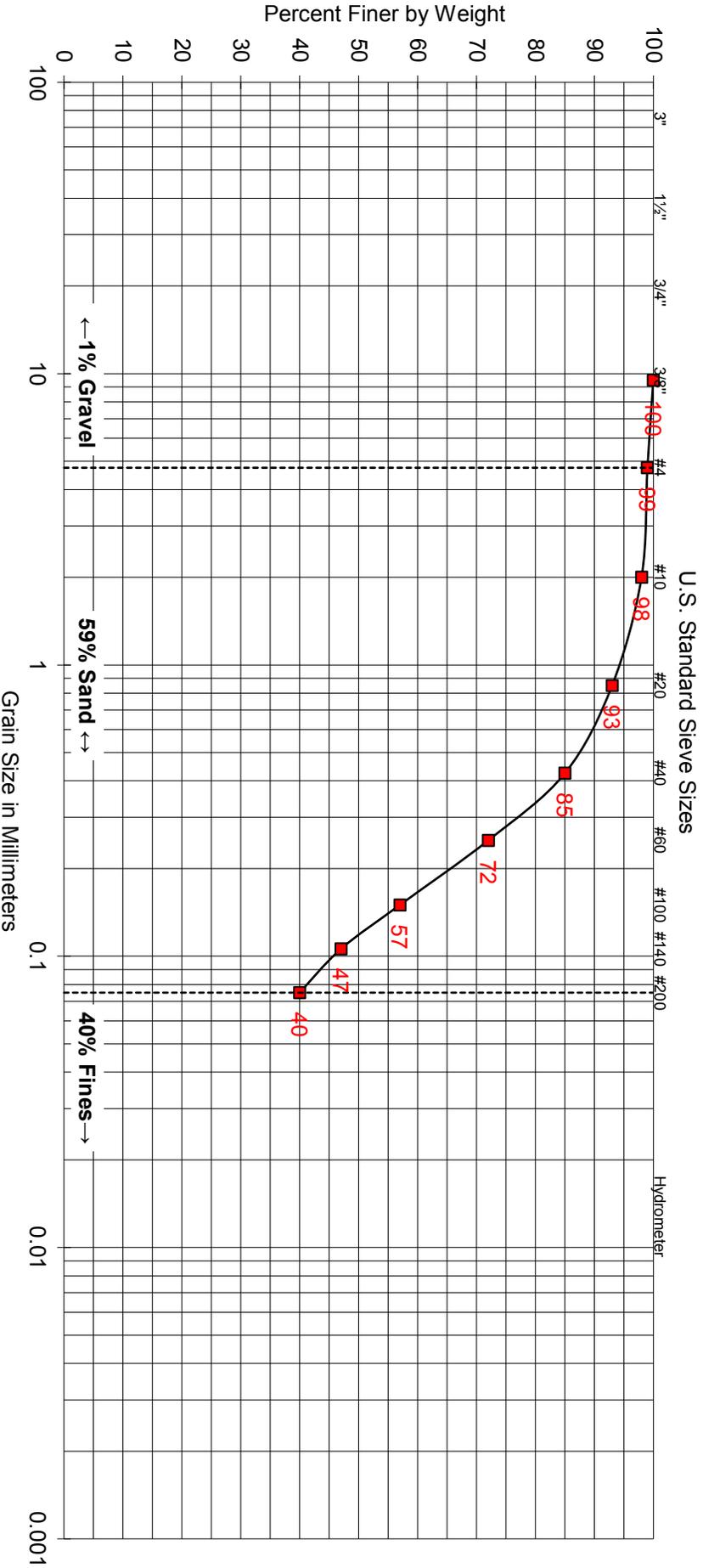
GROUP DELTA

SOIL CLASSIFICATION

SAMPLE B-2
SAMPLE NUMBER: S-3
SAMPLE DEPTH: 10'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

ATTERBERG LIMITS
LIQUID LIMIT: 0
PLASTIC LIMIT: 0
PLASTICITY INDEX: 0

Laboratory No. SO6250
Project No. IR780
FIGURE B-1.1

Percent Finer by Weight

U.S. Standard Sieve Sizes

Grain Size in Millimeters

COARSE | FINE
GRAVEL

COARSE | MEDIUM | FINE
SAND

SILT AND
CLAY

← 1% Gravel
59% Sand →
40% Fines →

100
99
98
93
85
72
57
47
40

HDR

TRANSMITTAL LETTER

DATE: November 9, 2021

ATTENTION: **Asheesh Pradhan**

TO: Group Delta Consultants
1320 South Simpson Circle
Anaheim, CA 92806

SUBJECT: Laboratory Test Data
Bonanni 8572 Talbert Avenue
Your #IR780, HDR Lab #21-1091LAB

COMMENTS: Enclosed are the results for the subject project.

James T. Keegan, MD
Corrosion and Lab Services Section Manager

Table 1 - Laboratory Tests on Soil Samples

Group Delta Consultants
Bonanni 8572 Talbert Avenue
Your #IR780, HDR Lab #21-1091LAB
9-Nov-21

Sample ID

B-3 0-5

Resistivity		Units	
as-received		ohm-cm	104,000
saturated		ohm-cm	6,800
pH			8.3
Electrical Conductivity		mS/cm	0.09

Chemical Analyses

Cations

calcium	Ca^{2+}	mg/kg	23
magnesium	Mg^{2+}	mg/kg	ND
sodium	Na^{1+}	mg/kg	25
potassium	K^{1+}	mg/kg	63
ammonium	NH_4^{1+}	mg/kg	1.4

Anions

carbonate	CO_3^{2-}	mg/kg	57
bicarbonate	HCO_3^{1-}	mg/kg	98
fluoride	F^{1-}	mg/kg	4.3
chloride	Cl^{1-}	mg/kg	ND
sulfate	SO_4^{2-}	mg/kg	11
nitrate	NO_3^{1-}	mg/kg	2.6
phosphate	PO_4^{3-}	mg/kg	6.7

Other Tests

sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

APPENDIX C
LIQUEFACTION ANALYSES

LIQUEFACTION ANALYSIS REPORT

Project title : Group Delta Consultants　　　　　　　**Location : 8572 Talbert Ave, Fountain Valley, CA**

CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	35.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	7.68	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Cone resistance — Friction Ratio — SBTn Plot — CRR plot — FS Plot

$M_w=7^{1/2}$, sigma'=1 atm base curve

Summary of liquefaction potential

Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots

CRR plot

FS Plot

Liquefaction potential

Vertical settlements

Lateral displacements

During earthq.

CRR & CSR

Factor of safety

LPI

Settlement (in)

Displacement (in)

Depth (ft)

Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft
Fines correction method:	NCEER (1998)	Average results interval:	1
Points to test:	Based on Ic value	Ic cut-off value:	2.60
Earthquake magnitude M_w:	7.68	Unit weight calculation:	Based on SBT
Peak ground acceleration:	0.72	Use fill:	No
Depth to water table (insitu):	35.00 ft	Fill height:	N/A

Fill weight:	N/A
Transition detect. applied:	Yes
K_σ applied:	Yes
Clay like behavior applied:	Sands only
Limit depth applied:	No
Limit depth:	N/A

LPI color scheme

■ Very high risk
■ High risk
■ Low risk

F.S. color scheme

■ Almost certain it will liquefy
■ Very likely to liquefy
■ Liquefaction and no liq. are equally likely
■ Unlike to liquefy
■ Almost certain it will not liquefy

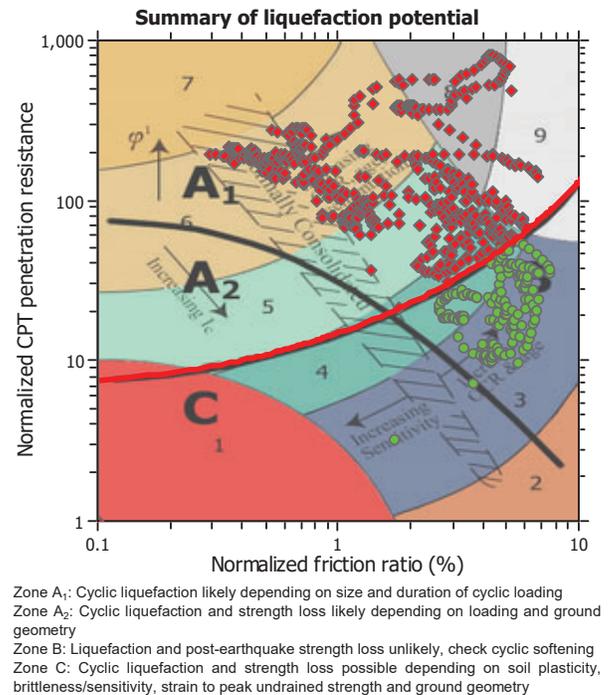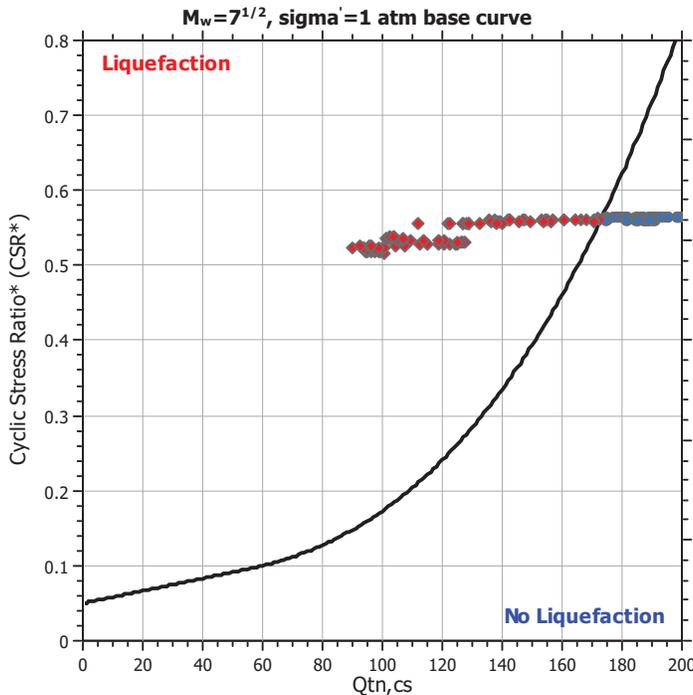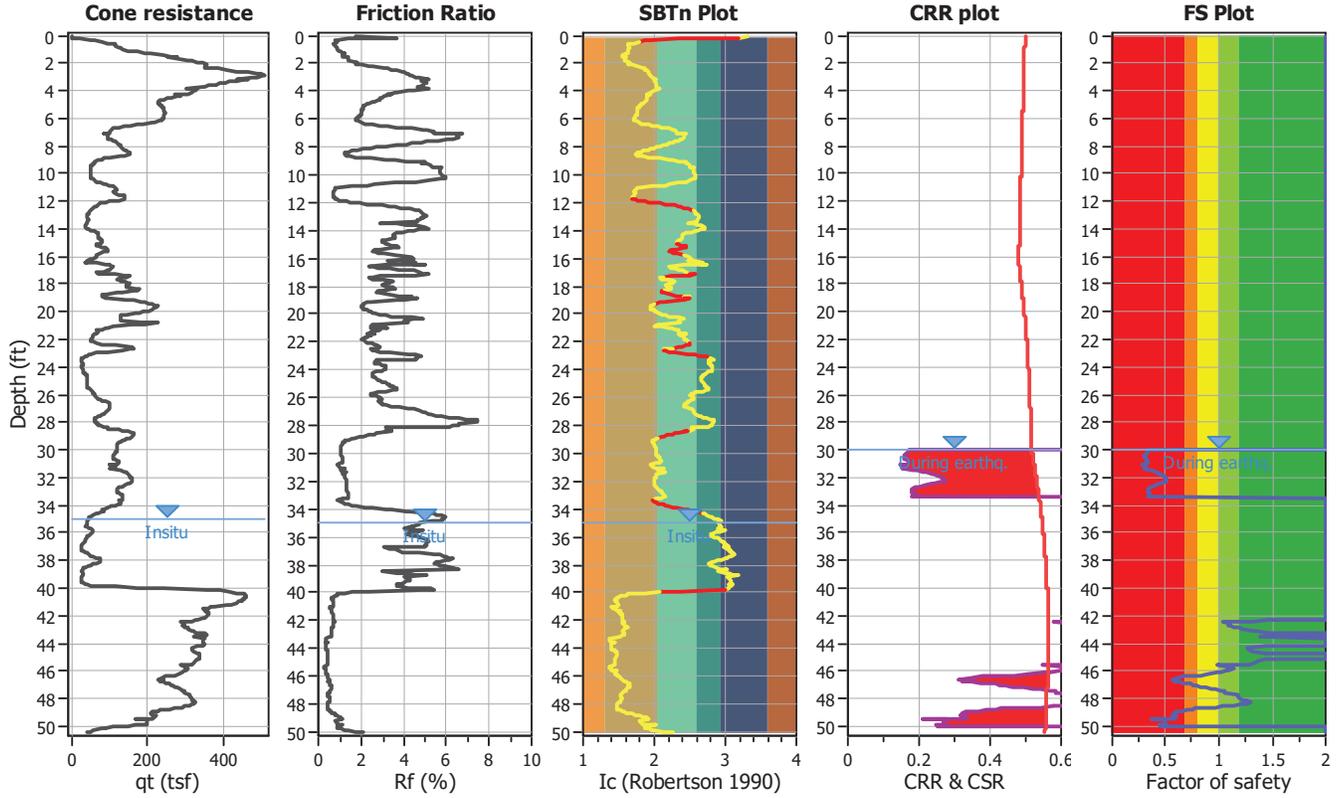Figure C-1B

LIQUEFACTION ANALYSIS REPORT

Project title : Group Delta Consultants　　　　　　**Location : 8572 Talbert Ave, Fountain Valley, CA**

CPT file : CPT-2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	35.00 ft	Use fill:	No	Clay like behavior		
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	applied:	Sands only	
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	No	
Earthquake magnitude M_w:	7.68	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A	
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based	

Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots

CRR plot

FS Plot

Liquefaction potential

Vertical settlements

Lateral displacements

During earthq.

CRR & CSR

Factor of safety

LPI

Settlement (in)

Displacement (in)

put parameters and analysis data

alysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
es correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
nts to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
F:thquake magnitude M_w:	7.68	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
ak ground acceleration:	0.72	Use fill:	No	Limit depth applied:	No
pth to water table (insitu):	35.00 ft	Fill height:	N/A	Limit depth:	N/A

LPI color scheme
- Very high risk
- High risk
- Low risk

F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

CLiq v.3.3.1.14 - CPT Liquefaction Assessment Software - Report created on: 11/9/2021, 2:45:04 PM
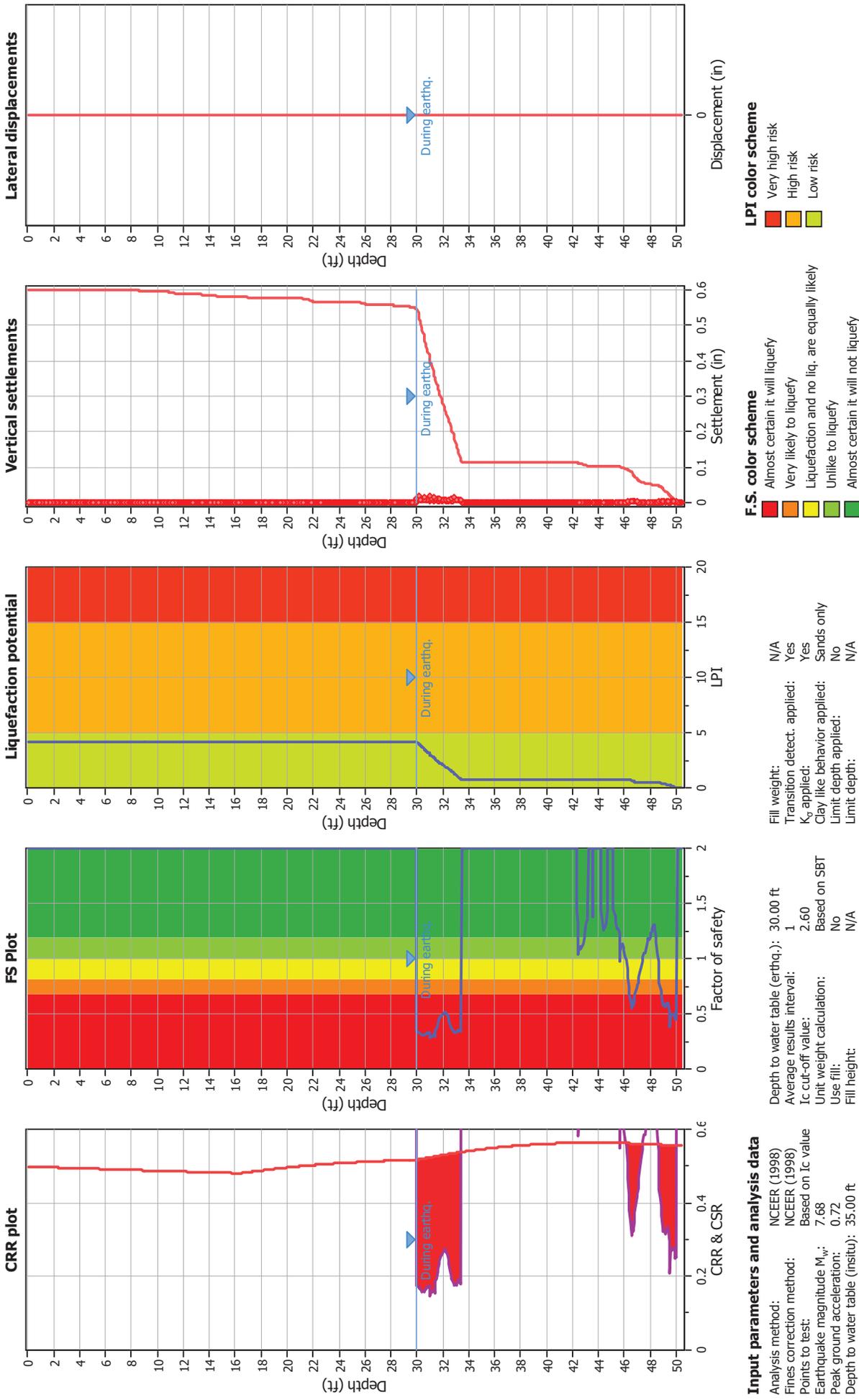ect file: \\192.168.100.6\Files$\Projects_AV\1700\1R780 Bonanni - 8572 Talbert Avenue\05_Field\CPT\1R780.clq

Figure C-2A

Figure C-2B

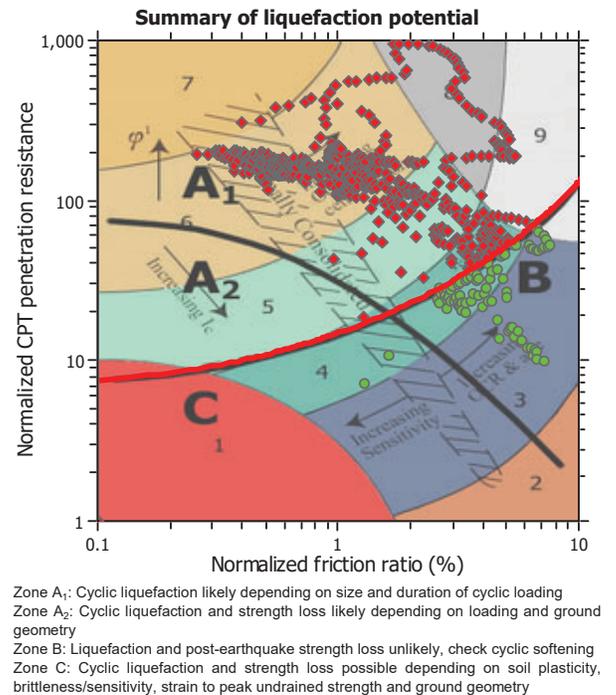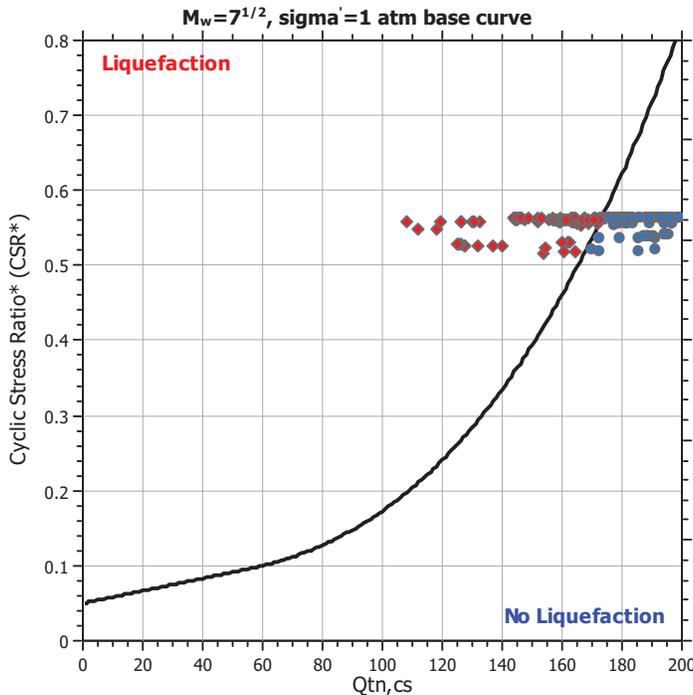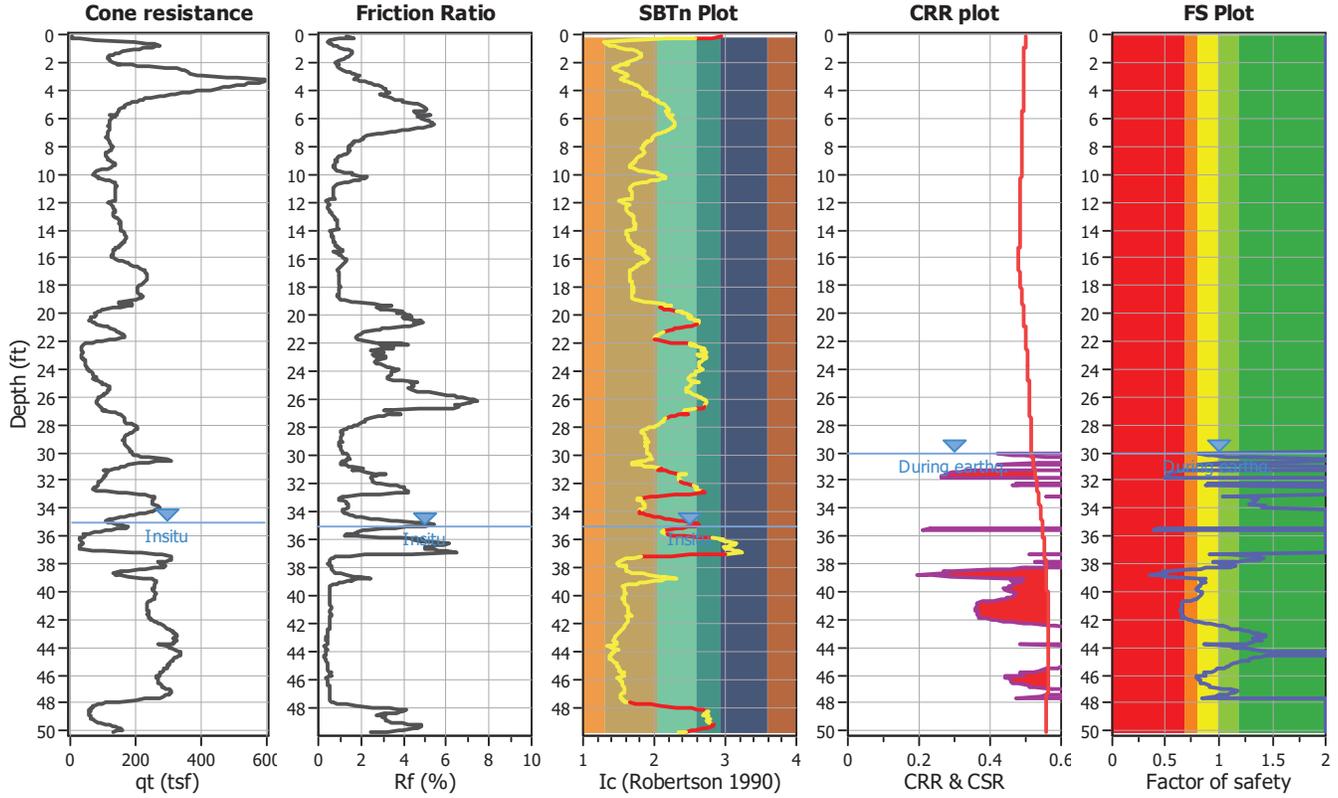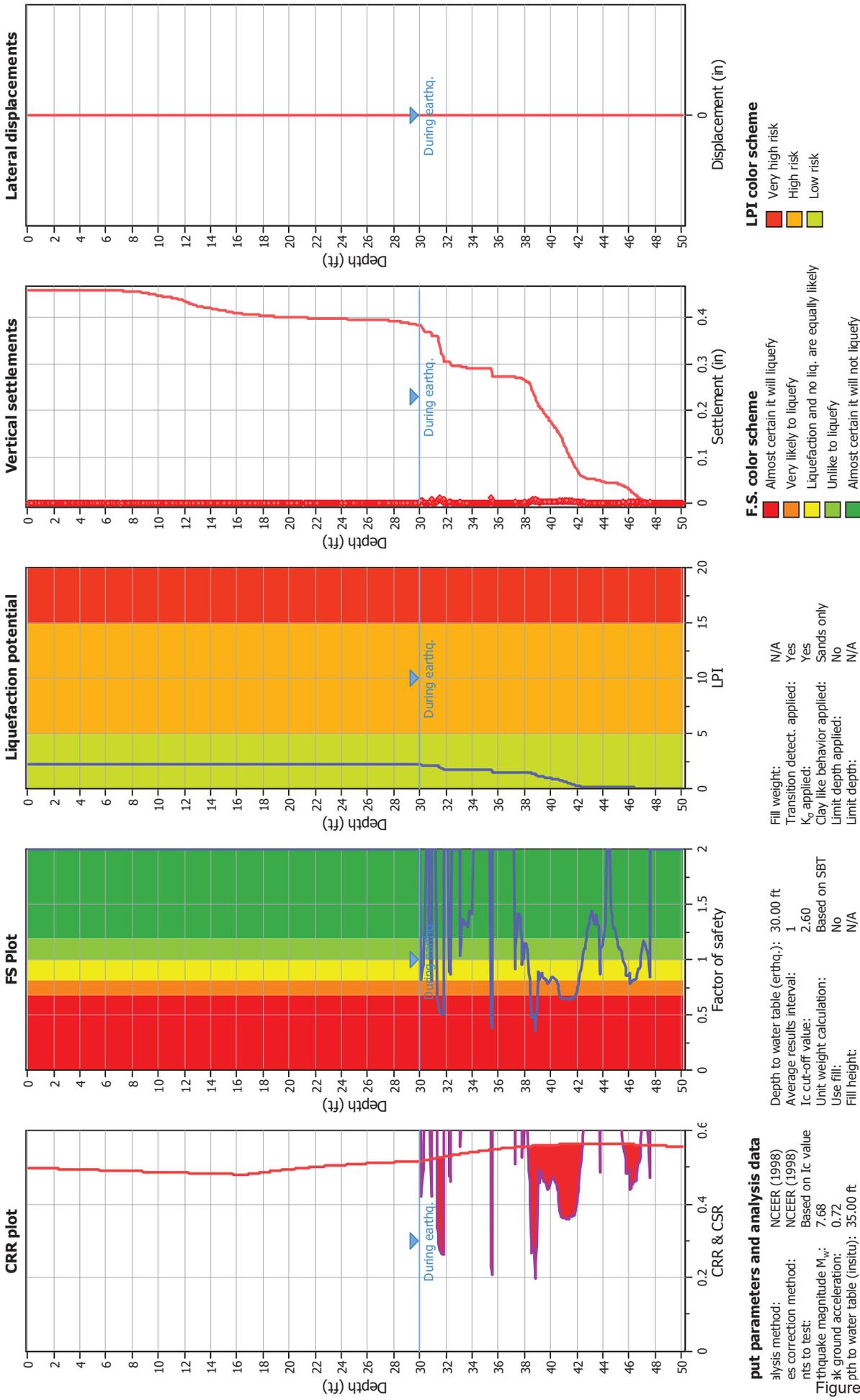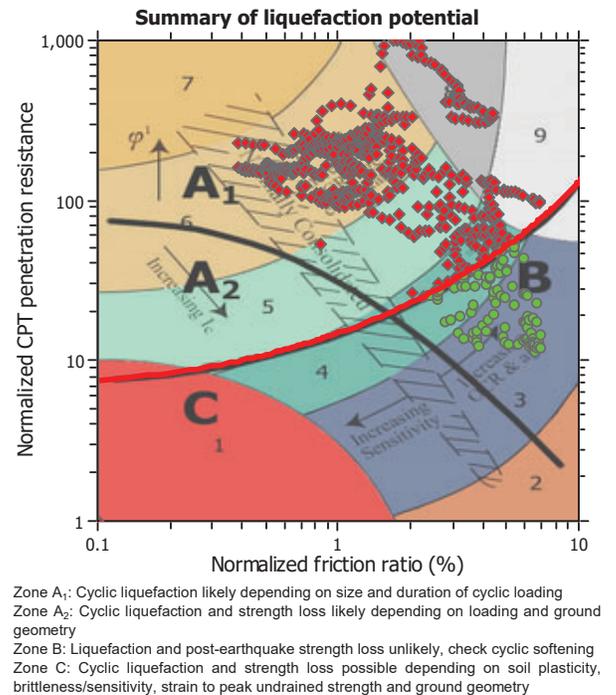## LIQUEFACTION ANALYSIS REPORT

Project title : Group Delta Consultants   **Location : 8572 Talbert Ave, Fountain Valley, CA**

CPT file : CPT-3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	35.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	7.68	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Cone resistance — Friction Ratio — SBTn Plot — CRR plot — FS Plot

$M_w=7^{1/2}$, sigma'=1 atm base curve

Summary of liquefaction potential

Zone A$_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A$_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots

Lateral displacements

During earthq.

Displacement (in)

Depth (ft)

LPI color scheme
- Very high risk
- High risk
- Low risk

Vertical settlements

During earthq.

Settlement (in)

Depth (ft)

F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

Liquefaction potential

During earthq.

LPI

Depth (ft)

FS Plot

During earthq.

Factor of safety

Depth (ft)

CRR plot

During earthq.

CRR & CSR

Depth (ft)

Input parameters and analysis data

Analysis method:	NCEER (1998)
Fines correction method:	NCEER (1998)
Points to test:	Based on Ic value
Earthquake magnitude M_w:	7.68
Peak ground acceleration:	0.72
Depth to water table (insitu):	35.00 ft

Depth to water table (erthq.):	30.00 ft
Average results interval:	1
Ic cut-off value:	2.60
Unit weight calculation:	Based on SBT
Use fill:	No
Fill height:	N/A

Fill weight:	N/A
Transition detect. applied:	Yes
K_σ applied:	Yes
Clay like behavior applied:	Sands only
Limit depth applied:	No
Limit depth:	N/A

CLiq v.3.3.1.14 - CPT Liquefaction Assessment Software - Report created on: 11/9/2021, 2:45:05 PM
Project file: \\192.168.100.6\Files$\Projects_AV\1700\1R780 Bonanni - 8572 Talbert Avenue\05_Field\CPT\1R780.clq

Figure C-3B