REPORT OF GEOTECHNICAL INVESTIGATION
USD GROUP CLEAN FUELS RAIL TERMINAL
NATIONAL CITY, CALIFORNIA

Prepared for

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Attention: Mr. David Atwater

SUBJECT: REPORT OF GEOTECHNICAL INVESTIGATION
USD Group Clean Fuels Rail Terminal
National City, California

Mr. Atwater:

We are pleased to submit this draft geotechnical investigation report for Phase 1 of the proposed USD Group Clean Fuels National City Rail Terminal project. This report is based on our recent subsurface explorations, laboratory testing and geotechnical analyses within the area of the proposed Phase 1 development. Specific conclusions regarding the geologic conditions at the site, and geotechnical recommendations for remedial grading, foundation, slab, and pavement section design are provided in the following report.

We appreciate this opportunity to be of continued professional service. Feel free to contact the office with any questions or comments, or if you need anything else.

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1.0 INTRODUCTION

The following report presents our geotechnical investigation for the proposed USD Group Biofuels Transloading Facility in National City, California. The site location is shown in Figure 1A. The site vicinity is shown in more detail in Figure 1B. Selected photographs of the site are shown in Figures 1C to 1E. Plans showing the layout of the proposed development is provided in Figures 2A and 2B. The approximate locations of the 6 exploratory borings and 6 CPT soundings that we have completed at the site are shown in Figure 3. The geologic conditions in the site vicinity are depicted in Figure 4A. A geologic cross section through the site is provided in Figure 4B.

1.1 Scope of Services

This report was prepared in general accordance with the provisions of the referenced proposal (GDC, 2021). The purpose of this investigation was to characterize the geotechnical conditions at the site and provide geotechnical recommendations for grading and the design of the proposed foundations, slabs, pavements, utilities, walls and surface improvements. The recommendations provided herein are based on the findings of the subsurface explorations, laboratory tests and engineering analyses, as well as our previous experience with similar geologic conditions in the site vicinity. In summary, we provided the following scope of services.

- A geologic reconnaissance of the surface characteristics of the site, and a review of the relevant reports referenced in Section 8.0.

- A subsurface exploration of the site including 6 exploratory borings and 6 cone penetration test (CPT) soundings within the areas of planned development. The approximate boring and CPT locations are shown on the Exploration Plan, Figure 3. Boring Records and CPT interpretations are provided in the figures of Appendix A.

- Laboratory testing of selected soil samples collected from the exploratory borings. Laboratory tests included sieve and hydrometer analyses, Atterberg Limits, Expansion Index, in-situ moisture content and dry density, soil corrosion, direct shear and R-Value. The laboratory test results are presented in Appendix B.

- Engineering analysis of the field and laboratory data to help develop geotechnical recommendations for site preparation, remedial earthwork, foundation, pavement and retaining wall design, soil reactivity, and site drainage and moisture protection. Our soil liquefaction analyses are presented in Appendix C.

- Preparation of this geotechnical report summarizing our findings, conclusions and geotechnical recommendations for the site development.

1.2 Site Description

The subject site is located along the eastern edge of the San Diego Bay, as shown on the Site Location Map, Figure 1A. The site is situated within National City at 837 19th Street, as shown on the Site Vicinity Plan, Figure 1B. Selected photographs showing the existing site conditions are provided in Figures 1C to 1E. The photograph locations and orientations are shown in Figure 3.

The site consists of an approximately 5-acre property that is currently undeveloped land. We understand that the site was formerly used for both railroad and industrial purposes. Much of the site is surfaced with a few inches of gravel. Harrison Avenue crosses through the center of the site and is surface with deteriorated asphalt concrete pavement. Various subsurface utilities also exist on site that will need to be relocated as part of the site redevelopment.

The southern (Phase 1) portion of the site is relatively flat lying, with gentle sheet grades that typically slope down to the northwest. Existing surface elevations in the Phase 1 area range from a low of about 13 feet above mean sea level (MSL) in the northwest portion of Phase 1A, to a high of about 18 feet MSL near the southeast corner of the site. Existing grades in the Phase 2 area are highly irregular and vary from about 12 feet MSL on the south, to about 5 feet MSL on the north.

1.3 Proposed Development

The proposed development will include a renewable diesel fuel facility that will be constructed in two phases, as shown on the Proposed Development, Figures 2A and 2B. Phase 1 of the development will include various new improvements that will allow fuel to be transferred directly from rail to trucks at a volume of about 15,000 barrels per day (see Figure 2A). The Phase 2 development plan indicates that several new rail spurs will also be extended from the site to the north onto an existing BNSF corridor (see Figure 2B). We understand that the Phase 2 development will commence after environmental remediation is completed in that area.

The Phase 1 development will include five new loading areas, power poles, various bermed spill containment areas, new parking and truck pavements, and various fences and gates among other improvements. An aboveground pipeline between the tracks will allow the fuel to be pumped over the tracks on a pipe bridge to three truck loading lanes. We understand that the power poles, pipe bridge, and other improvements may be founded on cast-in-drilled hole (CIDH) pile foundations. We also understand grades should remain relatively unchanged and that only a few feet of fill will be required along the southern and eastern property boundaries to attain plan grades. Up to about 6 feet of additional fill will be needed to raise track grades in the Phase 2 development area.

We anticipate that site development will begin by demolishing the existing surface improvements and removal of deleterious materials throughout the area of planned development. Existing subsurface utilities that will be abandoned or that may otherwise interfere with the planned excavations and proposed improvements will be removed and/or relocated. Remedial earthwork will then be conducted to prepare the new improvement areas.

2.0 FIELD AND LABORATORY INVESTIGATION

The field investigation included a visual and geologic reconnaissance of the site, the drilling of 6 exploratory borings with a truck mounted drill rig, and the completion of 6 cone penetration tests (CPT) between March 22nd and 23rd, 2022. The maximum depth of exploration was about 31½ feet. The approximate boring and CPT locations are shown on the Exploration Plan, Figure 3. Boring Records and CPT interpretations are provided in the figures of Appendix A.

Various soil samples were collected from the borings for laboratory testing and analysis. The testing program included gradation, hydrometer and Atterberg Limits to aid in material classification using the Unified Soil Classification System (USCS). Tests were conducted on ring samples to help estimate the in-situ dry density and moisture content of the soils we encountered at the site. Index tests were conducted on bulk samples to help evaluate the soil expansion potential and corrosivity. Direct shear tests were conducted to aid in soil strength characterization. R-Value tests were conducted to aid in pavement section design. The test results are presented in Appendix B.

2.1 Infiltration Testing

Four field infiltration tests were proposed as an optional part of this geotechnical investigation. The precise test locations and target test depths will be finalized through coordination with the project Civil Engineer. The borehole percolation test method will be used. Our previous experience with similar clayey soil types suggest that the factored vertical infiltration rates may be less than 0.05 inches per hour (including a Safety Factor of 2.0). Note that a factored infiltration rate of less than 0.05 inches per hour is indicative of a “No Infiltration” condition per the 2016 National City BMP Design Manual. Field infiltration testing will be conducted upon request.

3.0 GEOLOGY AND SUBSURFACE CONDITIONS

The site is located within the coastal plain section of the Peninsular Ranges geomorphic province of southern California and is underlain at depth by Old Paralic Deposits (Map Symbol – Qop_6). The surface of the site is covered with Young Alluvium (Qya) associated with the Sweetwater River which flows into the bay north of the site as shown on the Regional Geologic Map, Figure 4A (Kennedy, 2005). A geologic cross section is provided in Figure 4B. The approximate cross section location is shown on Figure 3. Logs describing the subsurface conditions encountered in the explorations are provided in Appendix A. The geologic materials are described below.

3.1 Old Paralic Deposits

The entire site is underlain at depth by Pleistocene-age Old Paralic Deposits. Most of the CPT soundings met with refusal near the geologic contact between the alluvium and the Old Paralic Deposits. As observed in Boring B-2, the Old Paralic Deposits primarily consist of silty sandstone (SM) to the maximum depths we explored. The Old Paralic Deposits have a relatively high shear strength and low compressibility. The corrected Standard Penetration Test (SPT) blow counts (N_{60}) we collected within the Old Paralic Deposits ranged from 30 to 43, indicating a dense condition.

3.2 Alluvium

Alluvium (Qa) was encountered in most of our explorations at depths ranging from about 10 to 20 feet below existing surface grades. The alluvial soils we observed in the borings primarily consisted of clean sands such as poorly-graded sand and well-graded sand (SP, SP-SM and SW). Lesser amounts of silty sand and sandy silt (SM and ML) were also observed. The corrected Standard Penetration Test (SPT) blow counts (N_{60}) we collected within the Alluvium generally ranged from 17 to 38 and averaged 28, indicating a medium dense to dense condition. Direct shear tests suggest that the dense alluvium has a relatively high strength on the order of 42^{o} with 200 lb/ft^2 cohesion.

3.3 Fill

Roughly 9 to 11 feet of undocumented fill was observed in our explorations directly overlying the young alluvium. The undocumented fill soils that we observed generally consisted of a clayey sand with gravel and sandy lean clay (SC and CL). The deeper fill soils included sandy silt (ML). The fill contained little subangular gravel, as well as some trash and demolition debris including wood, plastic, glass and metal fragments. The CPT data indicates that the clayey fill is highly variable with undrained shear strength (Su) ranging from 2 to 6 KSF, indicating a very stiff to hard clay.

Laboratory tests conducted on shallow samples of the clayey fill indicated a low plasticity (Liquid Limit of 21 to 22), and a very low to low expansion potential (Expansion Index less than 50). The fill soils appear to be very corrosive to buried metals. R-Value tests indicate that the clayey soil will provide poor support for truck loads. Laboratory tests suggest that the dense silty fill soils have a drained shear strength on the order of 35^{o} with 250 lb/ft^2 cohesion. By comparison, the surficial clayey fill soils have a lower drained shear strength of roughly 27^{o} with 300 lb/ft^2 cohesion.

The existing pavement sections at the site were measured in Borings B-1 and B-5. The existing pavements vary from 3 to 8-inches of asphalt concrete with no underlying base. Other portions of the site are surfaced with several inches of coarse gravel or railroad ballast.

3.4 Groundwater

Groundwater was encountered in our explorations at depths ranging from about 14½ to 16½ feet below grade, which correspond to elevations ranging from about ½ to 1½ feet MSL. We have also reviewed historic data from ten groundwater monitoring wells located immediately north of the subject site (PTS, 2005). These ten wells were monitored on 60 occasions between the years of 2000 and 2004. The groundwater elevations varied from 0.8 to 1.4 feet and averaged 1.1 feet MSL.

The groundwater table does not appear to be influenced by tidal fluctuations in the San Diego Bay. However, changes in rainfall, irrigation or site drainage may produce seepage or locally perched groundwater within the fill or alluvium underlying the site. Accordingly, future excavations may encounter zones of wet soil and seepage. Due to the difficulty in predicting the location of perched groundwater, such conditions are typically mitigated if and where they occur.

4.0 GEOLOGIC HAZARDS

The subject site is not located within an area previously known for significant geologic hazards. Evidence of past landslides, liquefaction or active faulting was not encountered in our geotechnical investigation or literature review. The main geologic hazards at the site will be associated with the potential for strong ground motion due to a seismic event on the nearby Rose Canyon fault zone. Known active faults located within 100 kilometers (62 miles) of the site are shown in the Regional Fault Map, Figure 5A. Nearby faults within the San Diego Bay are depicted on the Local Fault Map, Figure 5B. Each of the potential geologic hazards at the site is described in more detail below.

4.1 Ground Rupture

Ground rupture is the result of movement on an active fault reaching the ground surface. The site is not located within an Alquist-Priolo Earthquake Fault Zone, and no indication of Holocene active or potentially active faulting was found during our investigation or literature review. The nearest known active fault is located within the San Diego Bay roughly 2 miles (3 kilometers) west of the site, as shown in Figures 5A and 5B. Consequently, the potential for ground rupture to adversely impact the site is considered to be low.

4.2 Seismicity

The site may be subjected to strong ground shaking from nearby large magnitude earthquakes occurring during the expected life span of the project. The site is located at latitude 32.6645° north and longitude 117.1129° west. The United States Geologic Survey maintains an interactive website that provides Next Generation Attenuation (NGA) probabilistic analyses based on the site location and shear wave velocity. Based on an estimated shear wave velocity for Site Class D, the peak ground accelerations (PGA) with a 2, 5 and 10 percent probability of being exceeded in a 50-year period at the site are estimated at approximately 0.633g, 0.443g and 0.311g, respectively.

By comparison to the probabilistic PGA values described above, the Maximum Considered and Design Earthquakes from the 2019 California Building Code (CBC) are 0.518g and 0.346g, respectively (see attached Table 1). The strong ground shaking hazard may be managed by structural design per the governing edition of the California Building Code. Seismic design parameters are provided in the recommendations section of this report.

4.3 Liquefaction and Dynamic Settlement

Liquefaction involves the sudden loss in strength of a saturated, cohesionless soil (sand and non-plastic silts) caused by the build-up of pore water pressure during cyclic loading, such as that produced by an earthquake. This increase in pore water pressure can temporarily transform the soil into a fluid mass, resulting in sand boils, settlement and 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 ground surface.

In summary, three simultaneous conditions are required for liquefaction:

- Historic high groundwater within 50 feet of the ground surface
- Liquefiable soils such as loose to medium dense sands
- Strong shaking, such as that caused by an earthquake

The granular loose to medium dense alluvial deposits at the site are susceptible to liquefaction due to a strong earthquake on a nearby active fault zone. Liquefaction analyses were conducted using a peak ground acceleration of 0.644g, corresponding to the 2019 CBC site modified MCE level peak ground acceleration (PGA_M). Groundwater levels were estimated at about 15 feet below existing grades for all these analyses. The liquefaction and settlement analyses are shown in Appendix C.

Our analyses indicate that the total dynamic settlement at the site will typically range from about ½ to 1 inch as shown in the figures of Appendix C. According to state guidelines, a differential settlement equal to roughly one-half of the total settlement may be conservatively assumed for structural design (SCEC, 1999). Therefore, we estimate that the post-liquefaction differential settlement of the proposed improvements will be on the order of ½ inch in 40 feet.

4.4 Landslides and Slope Instability

Evidence of ancient landslides or slope instabilities was not observed during our literature review or site reconnaissance. The site is essentially flat. Provided that our geotechnical recommendations are properly implemented during construction, and that shoring is used for vertical excavations, it is our opinion that slope instability should not adversely impact the proposed development.

4.5 Tsunamis, Seiches and Flooding

The site is located in close proximity to the San Diego Bay, with surface grades that vary from about 5 to 18 feet above mean sea level (MSL). The relatively close proximity to the bay suggests that the potential may exist for flooding in the event that an earthquake induced tsunami or seiche were to impact the San Diego Bay. However, the existence of the offshore barrier islands and the configuration of the continental shelf in the San Diego vicinity have historically provided relief from tsunamis. The ten largest tsunamis that occurred within the Pacific Ocean over the last 100 years did not significantly impact the San Diego Bay area.

The California Emergency Management Agency's Tsunami Inundation Map indicates that the site is located slightly above the estimated tsunami inundation area. Previous studies by the Army Corps of Engineers suggest that a 500-year tsunami within the Pacific Ocean may result in a water surface runup of about 5 to 8 feet above the existing bay surface elevations in the site vicinity (U.S. Army, 1974). The site is not located below any confined bodies of water and is not located within a FEMA 100-year flood zone. Consequently, the potential for earthquake induced flooding at the site is low.

5.0 CONCLUSIONS

The proposed improvements should be feasible from a geotechnical perspective, provided that appropriate measures are implemented during design development and earthwork construction. Several geotechnical conditions will need to be addressed.

- We anticipate that the lightly loaded foundations for the new minor structure will bear directly on a relatively shallow depth of structural compacted fill overlying the existing alluvial soils. A 4-foot-deep remedial excavation is recommended for the building pad and any other settlement sensitive improvement areas. Any moderately or highly expansive clay exposed by the remedial excavations should be removed from the improvement areas.

- A variety of structures may be founded on cast-in-drilled-hole (CIDH) pile foundations. Alternative recommendations are provided for 18 to 48-inch diameter CIDH piles between 2 and 30 feet in length. Allowable bearing capacities are provided for shallow foundations and short CIDH piles. For piles over 10-feet in length, the presence of groundwater may prohibit the development of end bearing. Axial capacities are provided for longer piles based on skin friction only. Wet construction methods will be needed for pile excavations that extend below groundwater (below about 5 feet MSL).

- The on-site soils are generally considered suitable for reuse in compacted fills, with the exception of any soils deemed to be contaminated based on environmental studies completed by others. The existing asphalt concrete does contain hydrocarbons, and may therefore not be suitable for reuse on site depending on the property owner.

- Laboratory tests indicate that the near surface soils at the site primarily consist of clayey sand with gravel and lean clay (SC and CL) with a low expansion potential (EI<50). However, it should be noted that some moderately or highly expansive clay (EI>90) may also exist on site. Additional testing should be conducted during fine grading to confirm that any fill placed within the building and improvement areas consists of low expansion soil (EI<50). Imported fill should have a very low expansion potential (EI<20).

- Laboratory tests indicate that the on-site soils typically present a negligible potential for sulfate attack to concrete structures. However, the soils do appear to be *corrosive* to buried metals. Typical corrosion control measures should also be incorporated into the design. A corrosion consultant may be contacted for specific recommendations.

- The potential for active faults, seismic settlement or floods to adversely impact the site is considered remote. Other hazards that may impact site development include strong ground shaking from an earthquake on a nearby active fault. This hazard may be managed by structural design in accordance with the applicable building code.

6.0 RECOMMENDATIONS

The remainder of this report presents recommendations for earthwork construction and the design of the proposed improvements. These recommendations are based on empirical and analytical methods typical of the standards of practice in southern California. If these recommendations do not cover a specific feature of the project, please contact our office for revisions or amendments.

6.1 Design Development and Plan Review

We recommend that the grading, foundation and improvement plans be reviewed by Group Delta during the design development phase of this project. We anticipate that substantial changes in the development may occur from the preliminary design concepts used for this investigation. Such changes typically will require additional geotechnical evaluation and modifications to the geotechnical recommendations provided in this report.

6.2 Excavation and Grading Observation

Foundation and grading excavations should be observed by the project geotechnical consultant. During grading, the geotechnical engineer's representative should provide observation and testing services continuously. Such observations are considered essential to identify field conditions that differ from those anticipated by this investigation, to adjust designs to the actual field conditions, and to determine that the remedial grading is accomplished in general accordance with the recommendations presented in this report. The recommendations provided in this report are contingent upon Group Delta Consultants providing these services. Our personnel should perform sufficient testing of fill and backfill during grading and improvement operations to support our professional opinion as to compliance with the compaction recommendations.

6.3 Earthwork

Grading and earthwork should be conducted in accordance with the requirements of the current California Building Code and the City of National City. The following recommendations are provided regarding specific aspects of the proposed earthwork and improvement operations. These recommendations should be considered preliminary and subject to revision based on the conditions observed by the geotechnical consultant during grading.

6.3.1 Site Preparation

General site preparation should begin with the removal of deleterious materials from the site. Deleterious materials include existing structures, retaining walls, foundations, slabs, asphalt concrete pavements, vegetation, demolition debris and contaminated soil (if encountered). Existing subsurface utilities that will be abandoned should be removed and the excavations backfilled and compacted as described in Section 6.3.4. Alternatively, abandoned pipes may be grouted with a two-sack sand-cement slurry under the observation of the geotechnical consultant.

6.3.2 Improvement Areas

At least two feet of compacted fill with an Expansion Index of 50 or less is recommended beneath all new concrete sidewalks and exterior flatwork areas, as well as all areas that will receive additional ballast placement for construction of the new railroad spur lines. To accomplish this objective, the upper 24-inches of soil immediately below slab subgrade or ballast elevations should be excavated and stockpiled on site. The exposed subgrade soil should then be scarified 12 inches, brought to optimum moisture, and compacted as described in Section 6.3.4. Low expansion (EI<50) imported or on-site soil should then be placed and compacted to the planned subgrade or ballast elevations. Compaction should be conducted immediately prior to placing concrete or ballast.

6.3.3 Building Areas

The main geotechnical constraint within the proposed building area consists of the presence of potentially compressible undocumented fill and surficial alluvial soils. For the proposed building pad area, all existing undocumented fill and alluvium should be excavated to a minimum depth of 4-feet below finish pad grade and replaced as compacted fill. The over-excavation should include all areas within 5-feet of the building foundation perimeter, including any isolated column foundations. The stockpiled soil from the over-excavations that is free of deleterious materials may be replaced as uniformly compacted fill to the planned finish pad grades.

In addition to the over-excavation and compaction of the surficial soil, a low expansion soil (with an Expansion Index of 50 or less) is recommended beneath the new building slab-on-grade. Some of the on-site soil may meet this criterion. Additional sampling and testing of the soil placed within 4-feet of finish pad grade should be conducted by the geotechnical consultant during grading to confirm that low expansion soils are placed within this zone.

6.3.4 Fill Compaction

All fill and backfill should be placed at slightly above optimum moisture content using equipment that is capable of producing a uniformly compacted product. The minimum recommended relative compaction is 90 percent of the maximum dry density at slightly above optimum moisture content per ASTM D1557. Sufficient observation and testing should be performed by the geotechnical consultant during grading so that an opinion can be rendered as to the compaction achieved. Rocks greater than 6 inches in maximum dimension should not be used in structural compacted fill.

Imported fill sources should be observed prior to hauling onto the site to determine the suitability for use. In general, imported fill materials should consist of granular soil with less than 35 percent passing the No. 200 sieve based on ASTM C136 and an Expansion Index less than 20 based on ASTM D4829. Samples of the import should be tested by the geotechnical consultant in order to evaluate the suitability of these soils for their proposed use. During grading operations, soil types may be encountered by the contractor that do not appear to conform to those discussed within this report. The geotechnical consultant should be notified to evaluate the suitability of these soils.

A two-sack sand and cement slurry may be used as an alternative to compacted fill soil. It has been our experience that slurry is often useful in confined areas which may be difficult to access with typical compaction equipment. A minimum 28-day compressive strength of 100 psi is recommended for the two-sack sand and cement slurry. Samples of the slurry should be fabricated and tested for compressive strength during construction.

6.3.5 Subgrade Stabilization

All excavation bottoms should be firm and unyielding prior to placing fill. In areas of saturated or "pumping" subgrade, a geogrid such as Tensar BX-1200 or Terragrid RX1200 may be placed directly on the excavation bottom, and then covered with at least 12 inches of minus ¾-inch aggregate base. Once the excavation is firm enough to attain the required compaction within the base, the remainder of the excavation may be backfilled using either compacted soil or aggregate base.

6.3.6 Surface Drainage

Foundation and slab performance depends greatly on how well surface runoff drains from the site. The ground surface should be graded so that water flows rapidly away from the structure and top of slope without ponding. The surface gradient needed to achieve this may depend on the prevailing landscaping. Planters should be built so that water will not seep into the foundation, slab, or pavement areas. If roof drains are used, the drainage should be channeled by pipe to storm drains, or discharge at least 10 feet from buildings. Irrigation should be limited to the minimum needed to sustain landscaping. Excessive irrigation, surface water, water line breaks, or rainfall may cause perched groundwater to develop within the underlying soil.

6.3.7 Storm Water Management

We understand that various bioretention basins and swales may be proposed on site to promote on-site infiltration for storm water Best Management Practice (BMP). In order to help determine the feasibility of full or partial on-site infiltration, the infiltration rates would typically be estimated at one or two locations within each BMP area using either the borehole percolation test method or a double ring infiltrometer. Infiltration testing may be conducted for this project once the precise BMP locations are determined. Based on our previous experience, we anticipate that the factored infiltration rates will likely be less than 0.05 inches per hour for the on-site clays, which is indicative of a "No Infiltration" design condition per the 2016 National City BMP Design Manual.

6.3.8 Temporary Excavations

Temporary excavations may be needed to construct the planned improvements. All excavations should conform to Cal-OSHA guidelines. The design, construction, maintenance and monitoring of all temporary slopes is the responsibility of the contractor. The contractor should have a competent person evaluate the geologic conditions encountered during excavation to determine permissible temporary slope inclinations and other measures as required by Cal-OSHA.

The following OSHA Soil Types may be assumed for assessment of temporary excavations.

Geologic Unit	Cal/OSHA Soil Type
Fill	Type B[1]
Alluvium	Type B[1]

1. Not subject to vibration, with no groundwater seepage into the excavation.

6.4 Foundation Recommendations

The foundations for the new buildings should be designed by the project structural engineer using the following geotechnical parameters. Recommendations are provided below for conventional shallow foundations or cast-in-drilled-hole (CIDH) pile foundations. These recommendations only provide minimum geotechnical criteria, and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or the structural engineer. The following recommendations should be considered preliminary, and subject to revision based on the conditions observed by the geotechnical consultant during fine grading.

6.4.1 Shallow Foundations

Assuming that the site is graded per our recommendations, we anticipate that new building foundations will bear directly on a relatively uniform depth of compacted fill. Shallow foundations should be at least 12 inches wide and 24 inches deep (see Figure 6). The following parameters may be used for both shallow foundation and short pile design purposes (see Figure 7).

Allowable Bearing: 2,500 lbs/ft^2. The allowable bearing pressure may be increased by 500 lbs/ft^2 for each additional foot of depth, up to a maximum value of 5,000 lbs/ft^2. A ⅓ increase in the allowable bearing is permitted for short-term wind or seismic loads.

Minimum Footing Width: 12 inches

Minimum Footing Depth: 24 inches below lowest adjacent soil grade

6.4.2 Deep Foundations

Cast-in-drilled-hole (CIDH) pile foundations will be used to support various new improvements. We anticipate that 18 to 48-inch diameter CIDH piles may be used. For our axial analyses, each pile was assumed to be spaced at 4 pile diameters such that group effects could be neglected. Axial capacity charts for 18 to 48-inch diameter CIDH piles are provided in Figure 7. The allowable capacities for piles shorter than 10-feet are derived from end bearing only (with a Safety Factor of 3.0).

The axial capacities provided in Figure 7 for piles longer than 10-feet do not include end bearing and are derived solely from skin friction (with a Safety Factor of 2.0). Pile excavations that extend more than 10-feet below existing grades may encounter groundwater, unstable bottom conditions, or possibly caving (below an elevation of about 5 feet MSL). The Contractor should be prepared to use wet methods to stabilize pile excavations that extend below groundwater, including filling the excavations with drilling slurry, or installing temporary casings (if needed). Concrete should be tremied into the stabilized pile excavations with a maximum drop height of 5 feet.

6.4.3 Settlement

Total and differential settlement of shallow building foundations loaded to the allowable bearing capacities provided above are not expected to exceed one inch and ¾-inch in 40 feet, respectively. We estimate that longer CIDH piles loaded to the allowable axial capacities presented in Figure 7 will experience less than ½ inch of total settlement due to axial loads. The entire site may also experience dynamic settlement following a strong earthquake, as described in Section 4.3.

6.4.4 Lateral Resistance

Lateral loads against the structures may be resisted by friction between the bottoms of footings, short piles, pile caps or slabs and the surrounding soil, as well as passive pressure from the portion of vertical foundation members embedded into compacted fill or alluvium. A coefficient of friction of 0.30 and a passive pressure of 300 psf per foot of depth may be used. The allowable friction and passive pressure values incorporate Safety Factors of 1.5 and 2.0 or more, respectively.

Preliminary LPILE analyses are provided for single 2, 3 and 4-foot diameter CIDH piles in Figures 8A to 8C. For the analyses, the piles were assumed to be 20 to 30-feet long to maintain pile tip fixity and composed of 4,000 psi reinforced concrete. Free head conditions were evaluated for ½, ¾ and 1-inch lateral displacement at the pile head. Additional LPILE analyses may be provided as the design development progresses, and the actual pile loads, sizes and reinforcement are known.

6.4.5 Seismic Design

Structures should be designed in general accordance with the seismic provisions of the 2019 California Building Code (CBC) for Seismic Design Category D. Based on the conditions we encountered in the subsurface explorations throughout the site, it is our opinion that a Site Class D would be most applicable to the site conditions per the 2019 CBC. Seismic parameters were developed using the referenced OSHPD online Seismic Design Maps Tool (OSHPD, 2022). The recommended 2019 CBC Design and MCE_G spectra for a Site Class D are provided in Table 1.

6.5 On-Grade Slabs

Building slabs should be at least 5 inches thick. The final slab thickness, control joints, and reinforcement should be designed by the structural engineer and should conform to the requirements of the current California Building Code.

6.5.1 Moisture Protection for Slabs

Moisture protection should comply with requirements of the current CBC, American Concrete Institute (ACI 302.1R-15) and the desired functionality of the interior ground level spaces. The Architect typically specifies an appropriate level of moisture protection considering allowable moisture transmission rates for the flooring or other functionality considerations. Moisture protection may be a "Vapor Retarder" or "Vapor Barrier" that use membranes with a thickness of 10 and 15 mil or more, respectively. ACI 302.1R-15 provides a flow chart to determine when and where these membranes should be used. Note the CBC specifies a Capillary Break, as defined and installed per the California Green Building Standards, with a Vapor Retarder.

6.5.2 Exterior Slabs

Exterior slabs and sidewalks should be at least 4 inches thick. Crack control joints should be placed on a maximum spacing of 10-foot centers, each way, for slabs, and on 5-foot centers for sidewalks. The potential for differential movements across the control joints may be reduced by using steel reinforcement. Typical reinforcement for exterior slabs would consist of 6x6 W2.9/W2.9 welded wire fabric placed securely at mid-height of the slab.

6.5.3 Expansive Soils

The near surface fill soils we observed in the subsurface investigation primarily consisted of clayey sand and lean clay (SC and CL). Laboratory tests and our previous experience suggests that these materials typically have a low expansion potential (EI<50), based on commonly accepted criteria. However, some moderately expansive clay may also exist on site in areas that were not explored. The Expansion Index test results are summarized in Figure B-2 in Appendix B.

6.5.4 Reactive Soils

In order to assess the sulfate exposure of concrete in contact with the site soils, samples were tested for water-soluble sulfate content, as shown in Figure B-3. The test results indicate that the on-site soils typically have a *negligible* potential for sulfate attack based on commonly accepted criteria. The sulfate content of the finish grade soils should be confirmed during fine grading.

In order to assess the reactivity of the site soils with buried metals, the pH, resistivity and chloride content were also determined (see Figure B-3). These tests suggest that the on-site soils may be *corrosive* to buried metals. Typical corrosion control measures should be incorporated into design, such as providing minimum clearances between reinforcing steel and soil, or sacrificial anodes for buried metal structures. It is the responsibility of the design build team to confirm that proper corrosion control measures are incorporated into the design and implemented during construction. A corrosion consultant may be contacted for specific recommendations.

6.6 Earth-Retaining Structures

Backfilling retaining walls with expansive soil can increase lateral pressures well beyond normal active or at-rest pressures. We recommend that retaining walls be backfilled with granular soil that has an Expansion Index of 20 or less (EI<20). Some of the on-site soil may meet this criterion. The select backfill zone should include all fill placed within a 1:1 plane extending back and up from the base of the wall. Retaining wall backfill should be compacted to at least 90 percent relative compaction based on ASTM D1557. Backfill should not be placed until the retaining walls have achieved adequate strength. Heavy compaction equipment should not be used.

For general design of retaining walls bearing on at least 2-feet of granular compacted fill, an allowable bearing capacity of 2,500 lbs/ft^2, a coefficient of friction of 0.30, and a passive pressure of 300 psf per foot of depth is recommended.

6.6.1 Cantilever Walls

Cantilever retaining walls with level granular backfill may be designed using an active earth pressure approximated by an equivalent fluid pressure of 35 lbs/ft^3 (see Figure 9A). The active pressure should be used for walls free to yield at the top at least ½ percent of the wall height. These pressures do not include groundwater forces. All retaining walls should contain adequate backdrains to relieve hydrostatic pressures. Typical wall drainage details are provided in Figure 9B.

Any surcharges located within a 1:1 plane extending back and up from the base of the retaining wall should also be accounted for in the design. Retaining walls situated adjacent to vehicular traffic areas may be designed to resist a uniform lateral surcharge pressure of 100 lb/ft^2 resulting from a typical 300 lb/ft^2 traffic surcharge acting behind the wall.

6.6.2 Seismic Wall Loads

Per the provisions of the 2019 California Building Code (CBC), seismic design is required for all earth retaining structures over 6 feet in height. The site modified MCE_G level peak ground acceleration (PGA_M) for the site is 0.644g, as shown in the attached Table 1. Design level loads are traditionally used for seismic design of retaining walls (PGA_M/1.5~0.429g), as described in Section 1803A.5.12 of the 2019 CBC. A fraction of the Design level peak ground acceleration is typically used for pseudo-static seismic wall design to account for yielding of the walls.

We have provided seismic retaining wall design parameters based on a pseudo-static seismic load of 0.27g, corresponding to 1 to 2 inches of seismic deformation. The recommended seismic increment of 26 lb/ft^3 for yielding walls is shown in the attached Figure 9A.

6.7 Pavement Design

For all pavement areas, the upper 12 inches of subgrade should be scarified immediately prior to constructing the pavements, brought to optimum moisture, and compacted to at least 95 percent of the maximum density per ASTM D1557. Aggregate base should also be compacted to 95 percent relative compaction. Aggregate base should conform to the Standard Specifications for Public Works Construction (*SSPWC),* Section 200-2. Asphalt concrete should conform to Section 400-4 of the *SSPWC* and should be compacted to 91 and 97 percent of the Rice density per ASTM D2041.

6.7.1 Asphalt Concrete

To aid in preliminary design, R-Value tests were conducted in general accordance with CTM 301 using soil samples collected from the site during the field investigation. The test results varied from 21 to 23 (see Figures B-5.1 and B-5.2 in Appendix B). The final pavement section designs should be based on R-Value testing of the actual pavement subgrade soils collected during fine grading.

Asphalt concrete pavement design was conducted in general accordance with the Caltrans Design Method. We anticipate that a Traffic Index ranging from 5.0 to 9.0 may apply to new pavement areas. The project civil engineer should review the assumed Traffic Indices to determine if and where they apply to the various pavement areas proposed at the site. Based on an R-Value of 21 from our tests, and the assumed range of Traffic Indices, the following pavement sections apply.

PAVEMENT TYPE	ADTT[1]	TRAFFIC INDEX	ASPHALT SECTION	BASE SECTION
Passenger Car Parking (Only)	<1	5.0	3 Inches	7 Inches
Light Truck Traffic Areas	4	6.0	4 Inches	8 Inches
Medium Truck Traffic Areas	16	7.0	4 Inches	12 Inches
Heavy Truck Traffic Areas	50	8.0	5 Inches	14 Inches
Very Heavy Traffic Areas	136	9.0	6 Inches	15 Inches

1) NOTE: ADTT is Allowable Daily Truck Traffic for a 20-year design life, assuming one 18-Kip Equivalent Single Axle Load (ESAL) per truck.

6.7.2 Portland Cement Concrete

Concrete pavement design was conducted in general accordance with the simplified design procedure of the Portland Cement Association. This methodology is based on a 20-year design life. For design, it was assumed that aggregate interlock would be used for load transfer across control joints. The concrete was assumed to have a minimum flexural strength of 600 psi. For design, the subgrade materials were assumed to provide "low" support, based on the R-Value tests. Based on these assumptions, we recommend that the PCC pavement sections for truck traffic areas consist of at least 7 inches of concrete placed over 6 inches of compacted aggregate base. For a Traffic Index of 8.0, at least 8 inches of concrete over 6 inches of aggregate base is recommended. For a Traffic Index of 9.0, 9 inches of concrete over 12 inches of aggregate base should be used.

Crack control joints should be constructed for all PCC pavements on a maximum spacing of 10 feet, each way. Concentrated truck traffic areas, such as truck aprons and loading docks, should be reinforced with at least number 4 bars on 18-inch centers, each way.

6.8 Pipelines

The planned addition may include various pipelines such as water, storm drain and sewer systems. Geotechnical aspects of pipeline design include lateral earth pressures for thrust blocks, modulus of soil reaction, and pipe bedding. Each of these parameters is discussed separately below.

6.8.1 Thrust Blocks

Lateral resistance for thrust blocks may be determined by a passive pressure value of 300 lbs/ft^2 per foot of embedment, assuming a triangular distribution. This value may be used for thrust blocks embedded into compacted fill soils as well as the formational materials.

6.8.2 Modulus of Soil Reaction

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines. For the purpose of evaluating deflection due to the load associated with trench backfill over the pipe, a value of 1,500 lbs/in^2 is recommended for the general conditions, assuming granular bedding material is placed around the pipe (USBR, 1977).

6.8.3 Pipe Bedding

Typical pipe bedding as specified in the *Standard Specifications for Public Works Construction* may be used. As a minimum, we recommend that pipes be supported on at least 4 inches of granular bedding material such as minus ¾-inch crushed rock or disintegrated granite. Where pipeline excavations exceed a 15 percent gradient, we do not recommend that open graded rock be used for bedding or backfill because of the potential for piping and internal erosion. For sloping utilities, we recommend that coarse sand or sand-cement slurry be used for the bedding and pipe zone. The slurry should consist of a 2-sack mix having a slump no greater than 5 inches.

6.8.4 Filter Fabric Separator

It has been our experience that soil may migrate into void spaces within an open graded gravel over time. A ¾-inch Minus Crushed Rock may have 50 percent void space or more, creating the potential for migration of a large volume of soil into the gravel voids. This migration of soil may take several years to occur, and is generally recognized only when surface manifestations develop, such as settlement of the pavement around a manhole or over a utility trench. In order to reduce the potential for distress to settlement sensitive improvements at the site, we recommend that a filter fabric separator (such as Mirafi 140N or an approved similar product) be placed between the soil and any open graded gravel used around storm drain pipes and manholes that are constructed within roadways, or beneath areas finished with concrete flatwork or pavers.

7.0 LIMITATIONS

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in similar localities. No warranty, express or implied, is made as to the conclusions and professional opinions included in this report.

The findings of this report are valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on this or adjacent properties. In addition, changes in applicable or appropriate standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

8.0 REFERENCES

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TABLES

TABLE 1 - 2019 CBC ACCELERATION RESPONSE SPECTRA

INPUT

S_S=	1.296	g = short period (0.2 sec) mapped spectral response acceleration MCE Site Class B (**ASCE 7-16 Section 11.4.2 and Figure 22-1**)
S_1=	0.433	g = 1.0 sec period mapped spectral response acceleration MCE Site Class B (**ASCE 7-16 Section 11.4-2 and Figure 22-2**)
Site Class=	D	= Site Class definition based on **2019 California Bulding Code**
F_a=	1.000	= Site Coefficient applied to S_s to account for soil type (**ASCE 7-16 Table 11.4-1**)
F_v=	1.867	= Site Coefficient applied to S_1 to account for soil type (**ASCE 7-16 Table 11.4-2**)
T_L=	8.00	sec = Long Period Transition Period (**ASCE 7-16 Figure 11.4-1**)

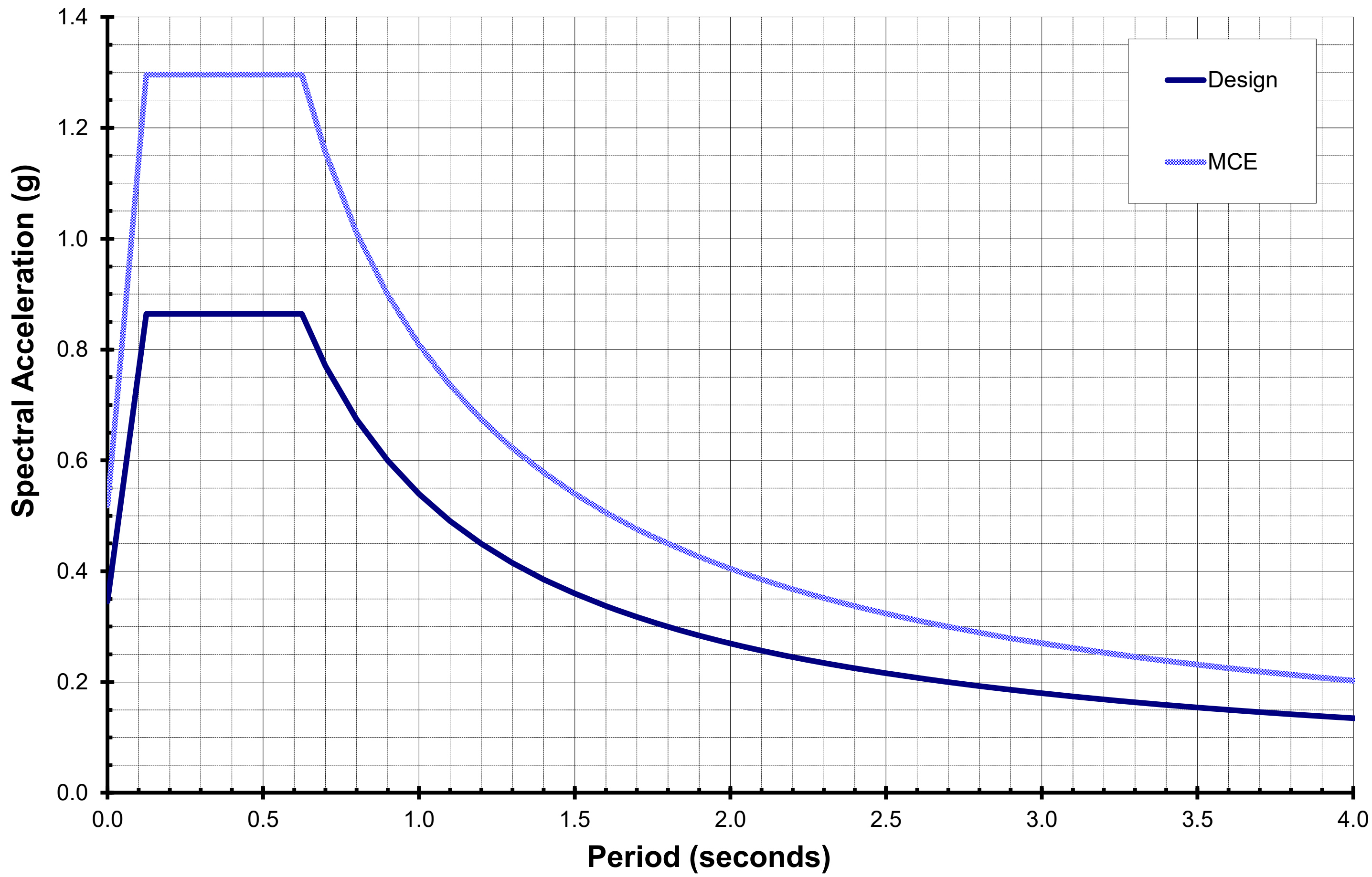
OUTPUT

S_{MS}=	1.296	= site class modified short period (0.2 sec) MCE spectral response acceleration = F_a x S_s (**ASCE 7-16 Equation 11.4-1**)
S_{M1}=	0.809	= site class modified 1.0 sec period MCE spectral response acceleration = F_v x S_1 (**ASCE 7-16 Equation 11.4-2**)
S_{DS}=	0.864	= site class modified short period (0.2 sec) Design spectral response acceleration = 2/3 x S_{MS} (**ASCE 7-16 Equation 11.4-3**)
S_{D1}=	0.539	= site class modified 1.0 sec period Design spectral response acceleration = 2/3 x S_{M1} (**ASCE 7-16 Equation 11.4-4**)
T_0=	0.125	sec = 0.2 S_{D1}/S_{DS} = Control Period (left end of peak) for ARS Curve (**ASCE 7-16 Section 11.4.6**)
T_S=	0.624	sec = S_{D1}/S_{DS} = Control Period (right end of peak) for ARS Curve (**ASCE 7-16 Section 11.4.6**)

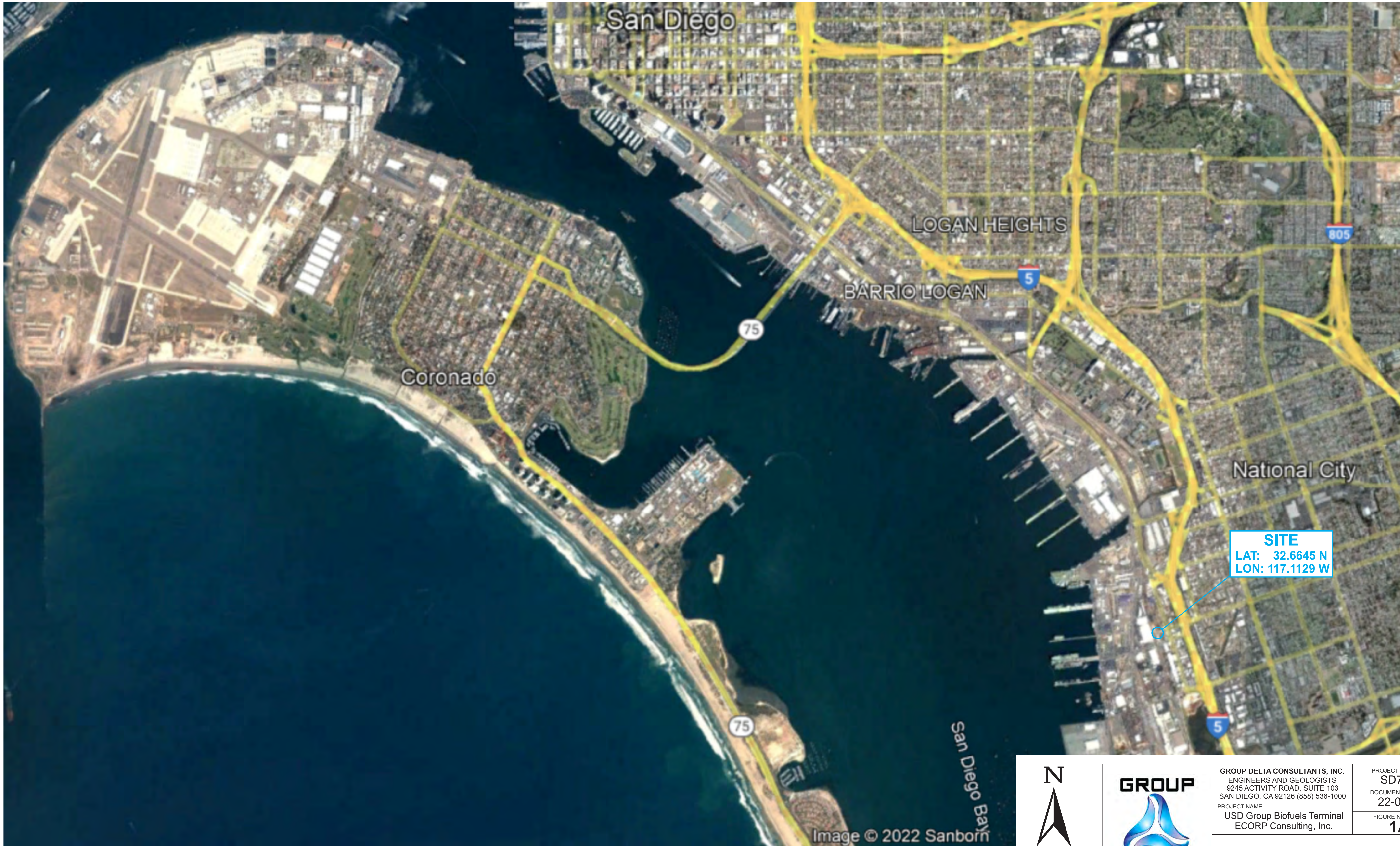
Site Latitude:	**32.6645**
Site Longitude:	**-117.1129**
Seismic Design Category:	**D**
Site Modified Peak Ground Acceleration (PGA_M):	**0.644**

SPECTRUM CALCULATION

T (seconds)	Design Sa (g)	MCE Sa (g)
0.000	0.346	0.518
0.125	0.864	1.296
0.624	0.864	1.296
0.700	0.771	1.156
0.800	0.674	1.011
0.900	0.599	0.899
1.000	0.539	0.809
1.100	0.490	0.736
1.200	0.450	0.674
1.300	0.415	0.622
1.400	0.385	0.578
1.500	0.360	0.539
1.600	0.337	0.506
1.700	0.317	0.476
1.800	0.300	0.450
1.900	0.284	0.426
2.000	0.270	0.405
2.100	0.257	0.385
2.200	0.245	0.368
2.300	0.235	0.352
2.400	0.225	0.337
2.500	0.216	0.324
2.600	0.207	0.311
2.700	0.200	0.300
2.800	0.193	0.289
2.900	0.186	0.279
3.000	0.180	0.270
3.100	0.174	0.261
3.200	0.169	0.253
3.300	0.163	0.245
3.400	0.159	0.238
3.500	0.154	0.231
3.600	0.150	0.225
3.700	0.146	0.219
3.800	0.142	0.213
3.900	0.138	0.207
4.000	0.135	0.202
4.100	0.132	0.197
5.000	0.108	0.162



FIGURES



N

NO SCALE

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PROJECT NAME
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ECORP Consulting, Inc.

PROJECT NUMBER
SD724

DOCUMENT NUMBER
22-0036

FIGURE NUMBER
1A

SITE LOCATION MAP

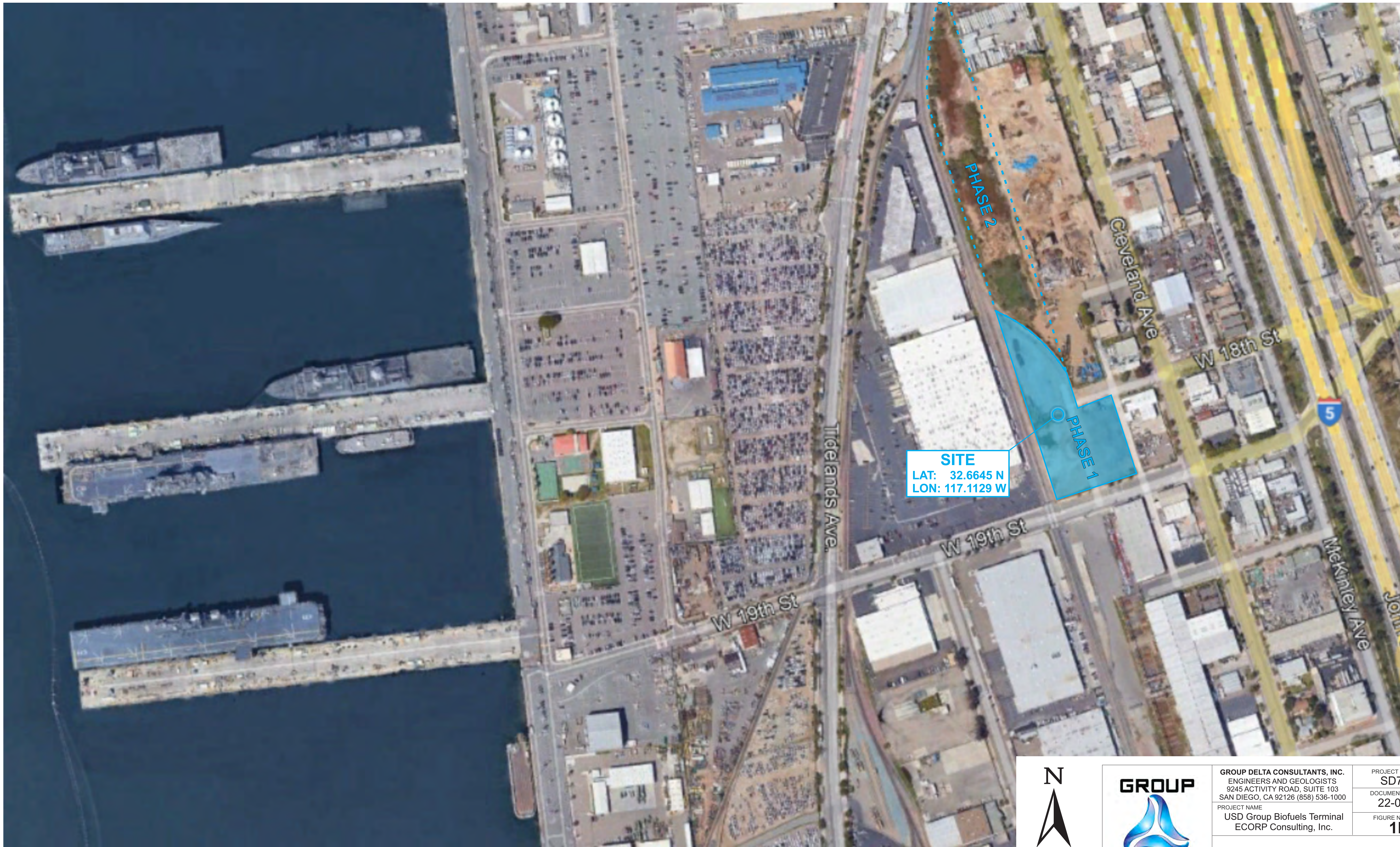

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FIGURE NUMBER
1B

SITE VICINITY PLAN

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DOCUMENT NUMBER	22-0036
FIGURE NUMBER	1C

SITE PHOTOGRAPHS

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1D

SITE PHOTOGRAPHS

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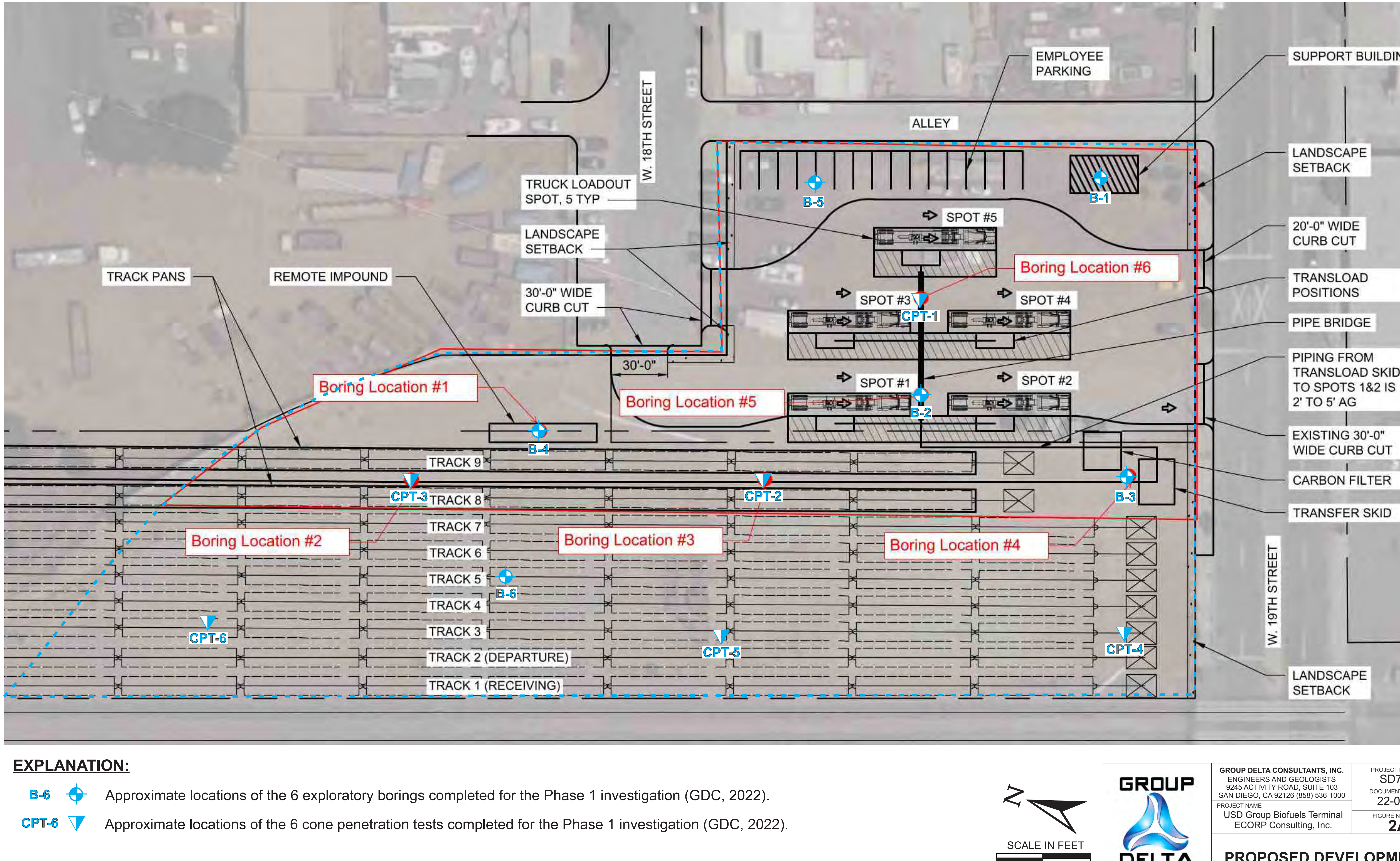
PROJECT NUMBER	SD724
DOCUMENT NUMBER	22-0036
FIGURE NUMBER	1E

SITE PHOTOGRAPHS



EXPLANATION:

B-6 Approximate locations of the 6 exploratory borings completed for the Phase 1 investigation (GDC, 2022).

CPT-6 Approximate locations of the 6 cone penetration tests completed for the Phase 1 investigation (GDC, 2022).

Reference: TKDA (2022). National City, California Rail Terminal Fuels Transload, Civil Site Plan, Proposed Soil Boring Locations, February 11.

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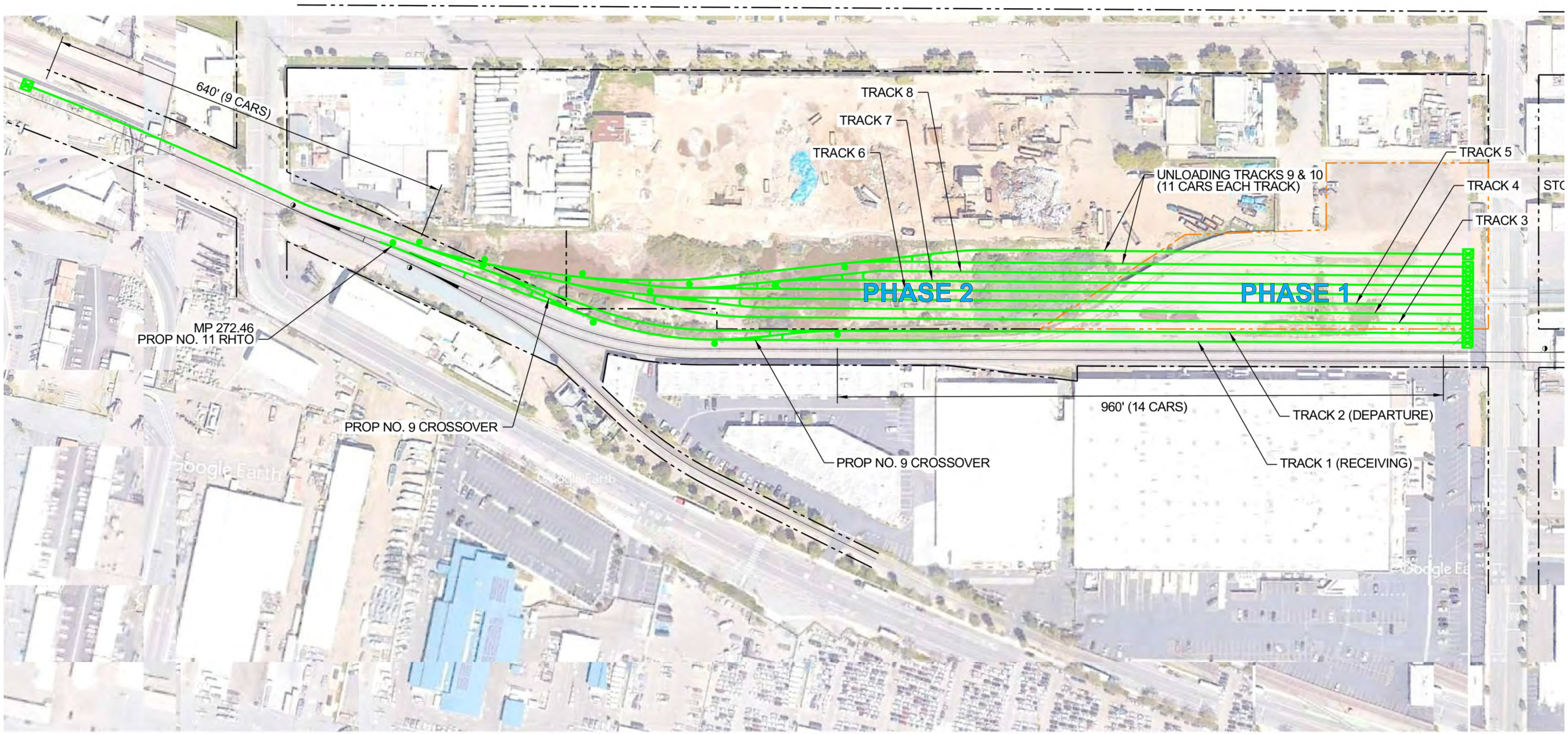
PROJECT NUMBER SD724
DOCUMENT NUMBER 22-0036
FIGURE NUMBER 2A

PROPOSED DEVELOPMENT



EXPLANATION:

PHASE 1 Approximate location of Phase 1 of the proposed development as described in this report.

PHASE 2 Approximate location of Phase 2 of the development (to be investigated at at future date).

Reference: USD Group (2021). National City Biofuels, Transloading Terminal, San Diego Sub, MP-272.62, National City, CA, Exhibit 14, October 22

N

NO SCALE

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	PROJECT NAME USD Group Biofuels Terminal ECORP Consulting, Inc.	FIGURE NUMBER 2B
	PROPOSED DEVELOPMENT	

EXPLANATION:

B-6 — Approximate locations of the 6 exploratory borings completed for the Phase 1 investigation (GDC, 2022).

CPT-6 — Approximate locations of the 6 cone penetration tests completed for the Phase 1 investigation (GDC, 2022).

N

SCALE IN FEET

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DOCUMENT NUMBER
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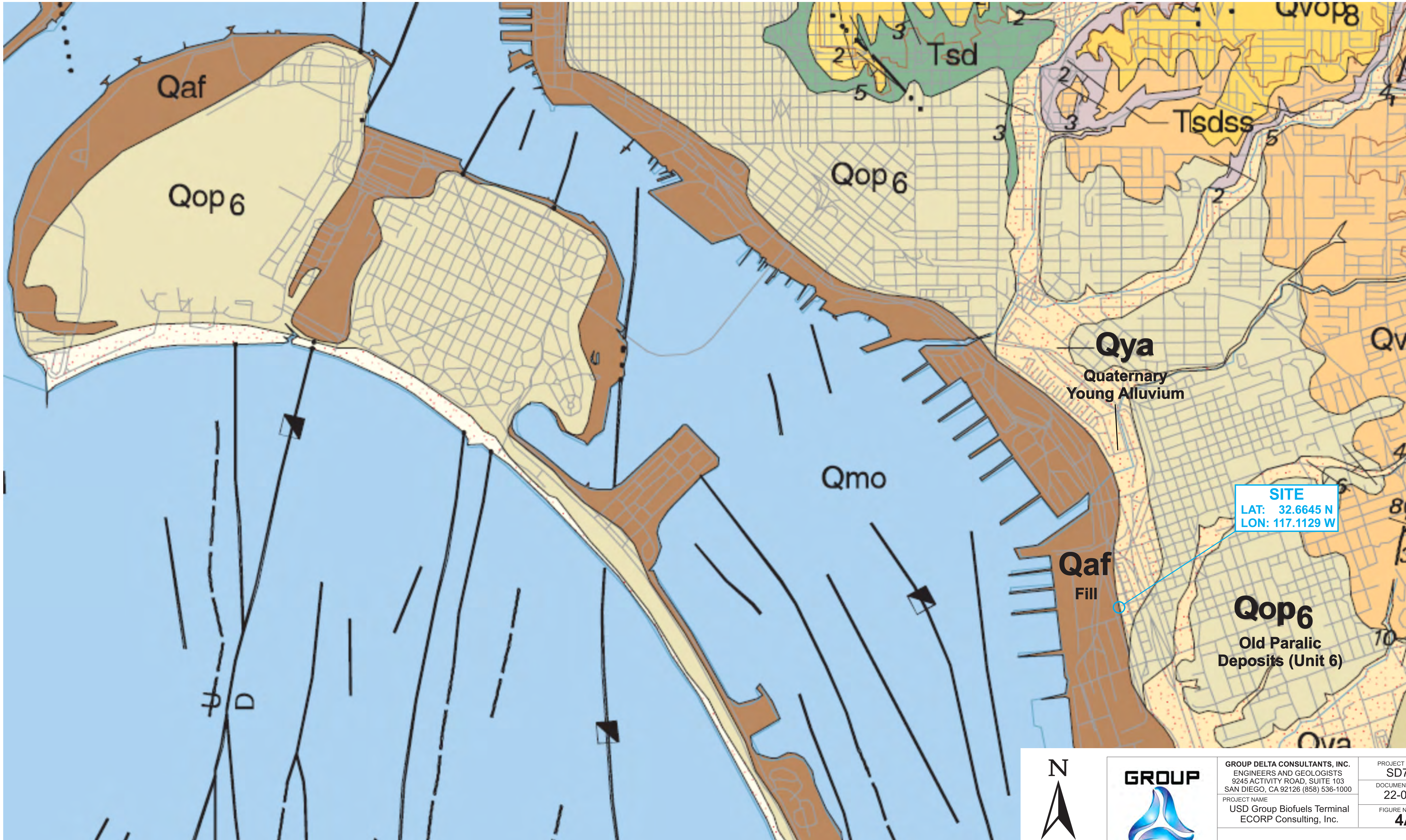
FIGURE NUMBER
3

EXPLORATION PLAN



Reference: Kennedy & Tan (2005). *Geologic Map of the San Diego 30' X 60' Quadrangle, California,* Scale 1:100,000.

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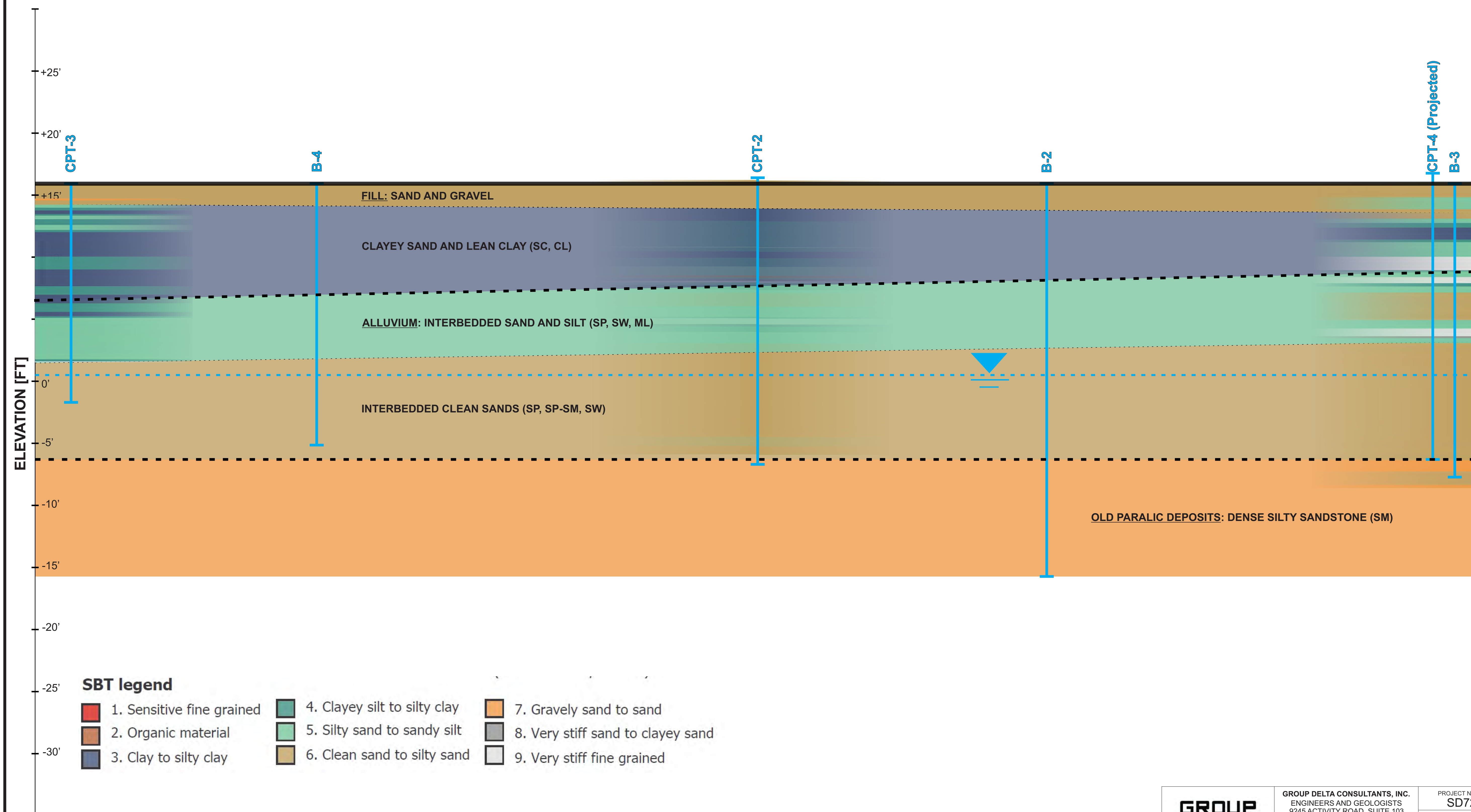
DOCUMENT NUMBER
22-0036

FIGURE NUMBER
4A

REGIONAL GEOLOGIC MAP



NOTE: Vertical Scale Exaggeration (5:1).

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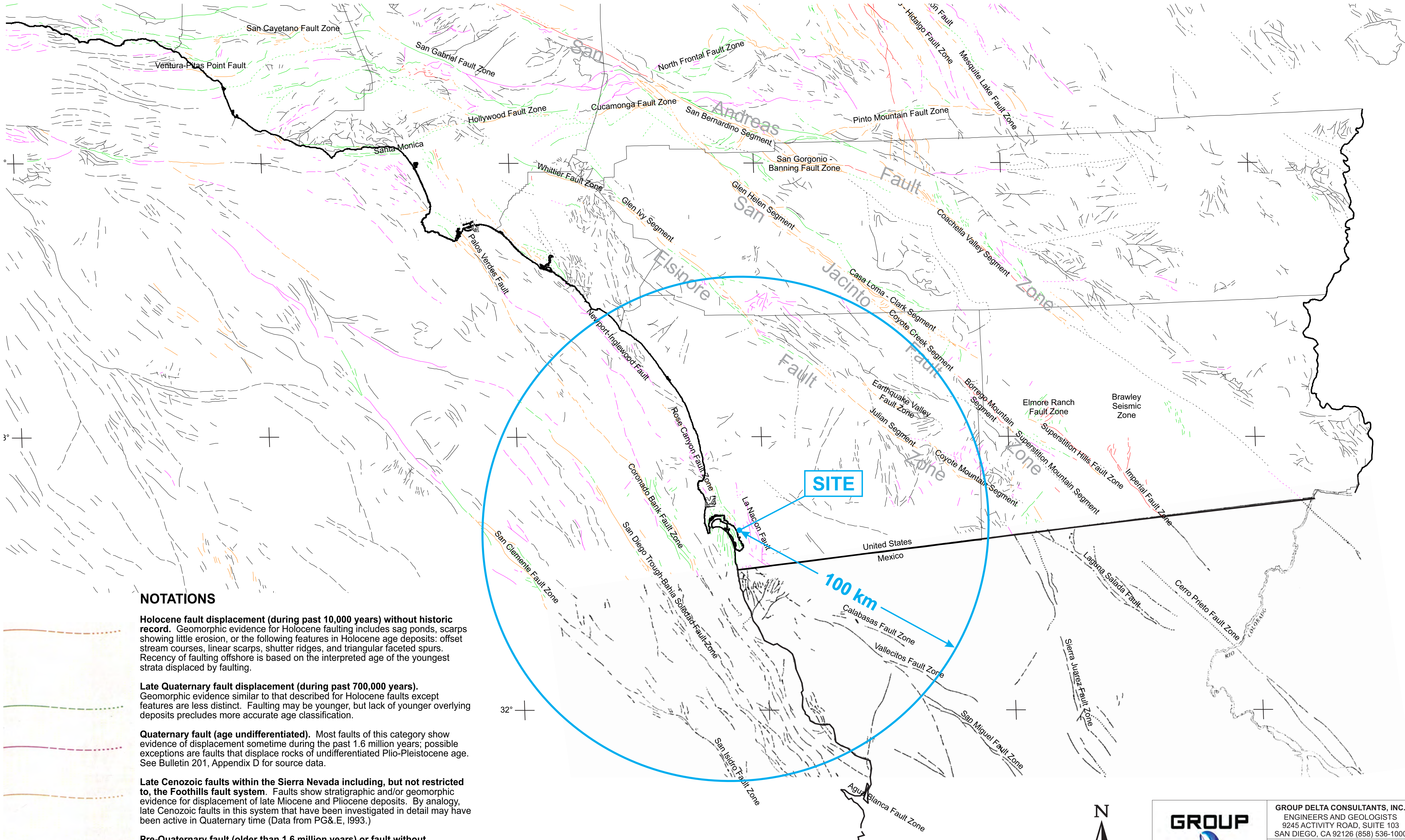
		DOCUMENT NUMBER 22-0036
	PROJECT NAME USD Group Biofuels Terminal ECORP Consulting, Inc.	FIGURE NUMBER 4B
	CROSS SECTION A-A'	



NOTATIONS

Holocene fault displacement (during past 10,000 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults that displace rocks of undifferentiated Plio-Pleistocene age. See Bulletin 201, Appendix D for source data.

Late Cenozoic faults within the Sierra Nevada including, but not restricted to, the Foothills fault system. Faults show stratigraphic and/or geomorphic evidence for displacement of late Miocene and Pliocene deposits. By analogy, late Cenozoic faults in this system that have been investigated in detail may have been active in Quaternary time (Data from PG&E, I993.)

Pre-Quaternary fault (older than 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissance nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

<u>**REFERENCE**</u>: Jennings, C.W. (1994). *Fault Activity Map of Callifornia and Adjacent Areas,* CDMG Geologic Data Series, Map No. 6.

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DOCUMENT NUMBER	22-0036
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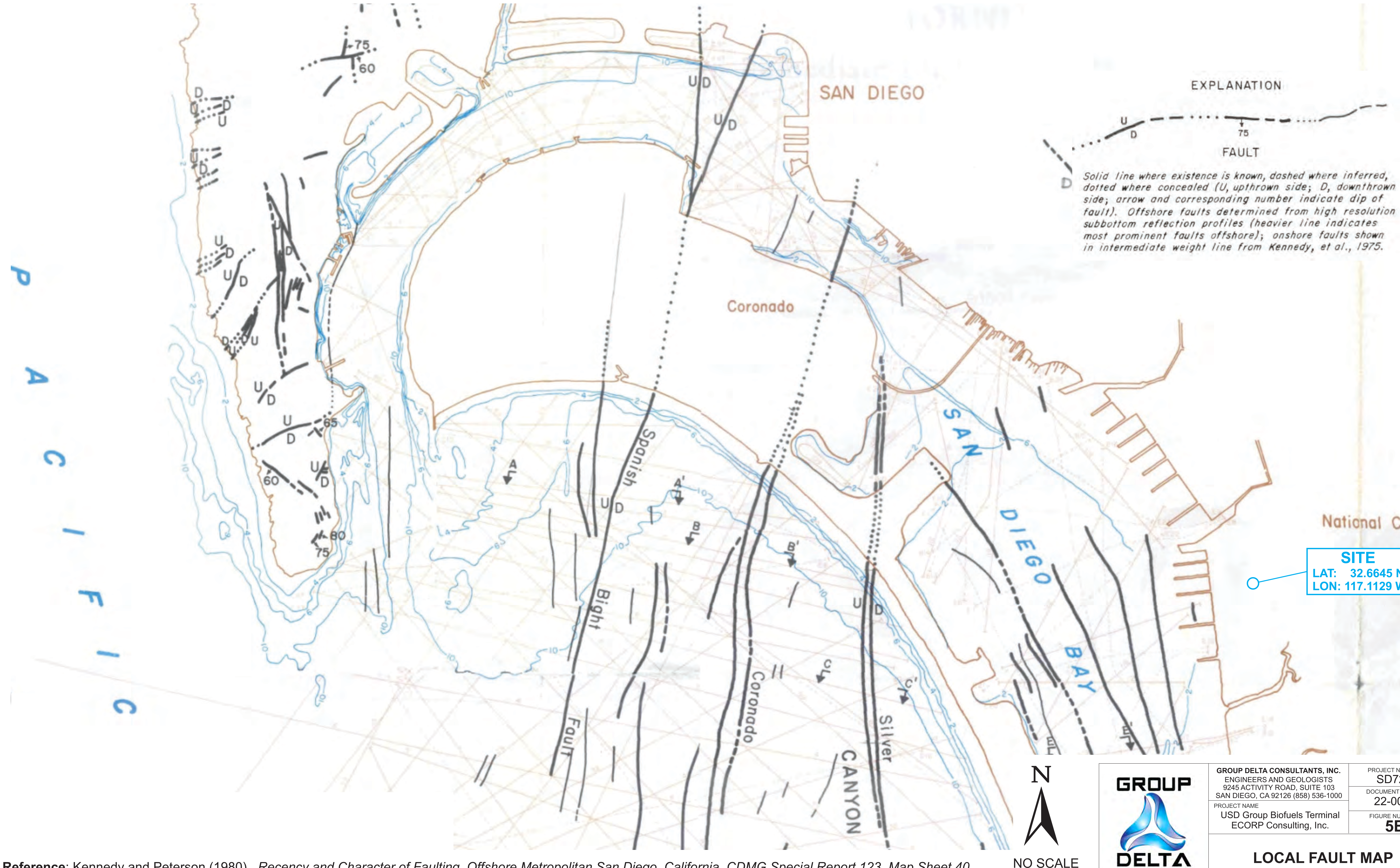
FIGURE NUMBER	5A

REGIONAL FAULT MAP



Reference: Kennedy and Peterson (1980). *Recency and Character of Faulting, Offshore Metropolitan San Diego, California, CDMG Special Report 123, Map Sheet 40.*

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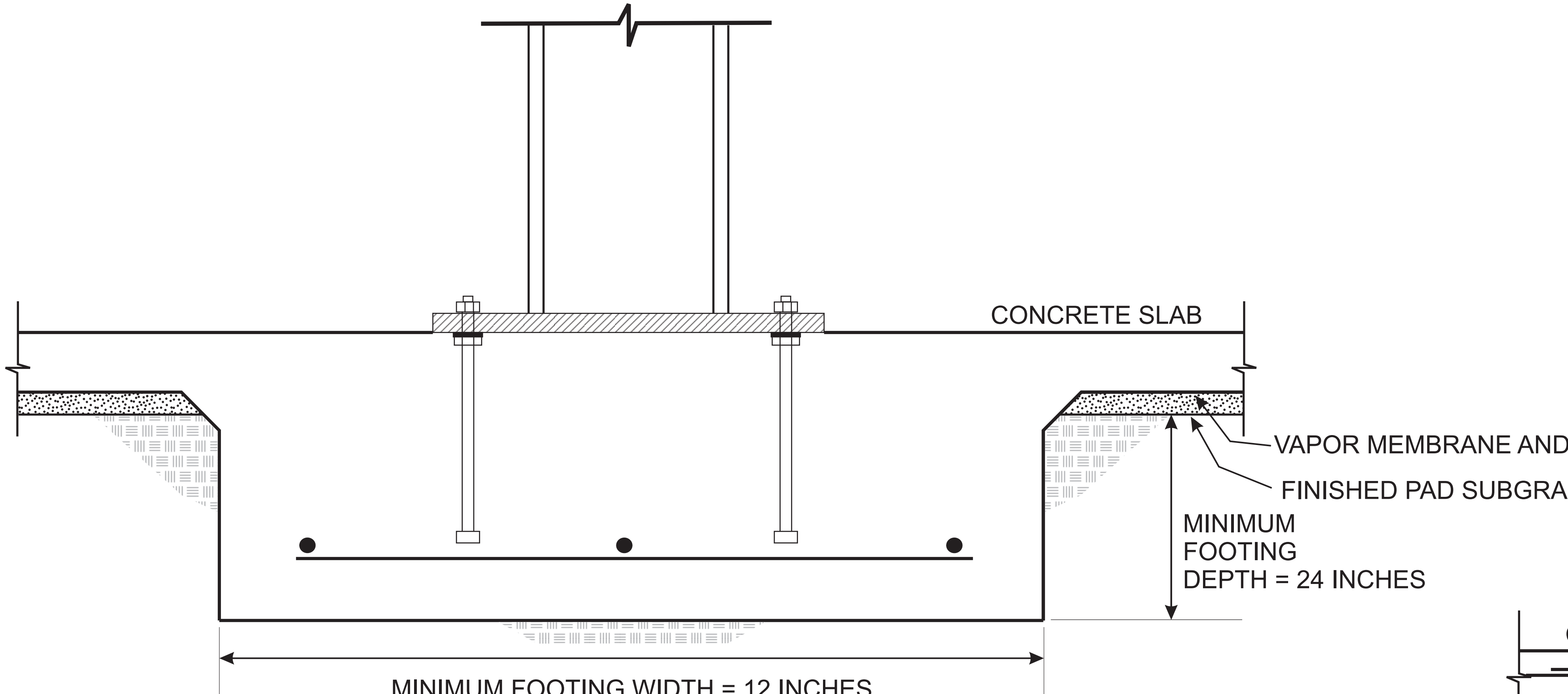
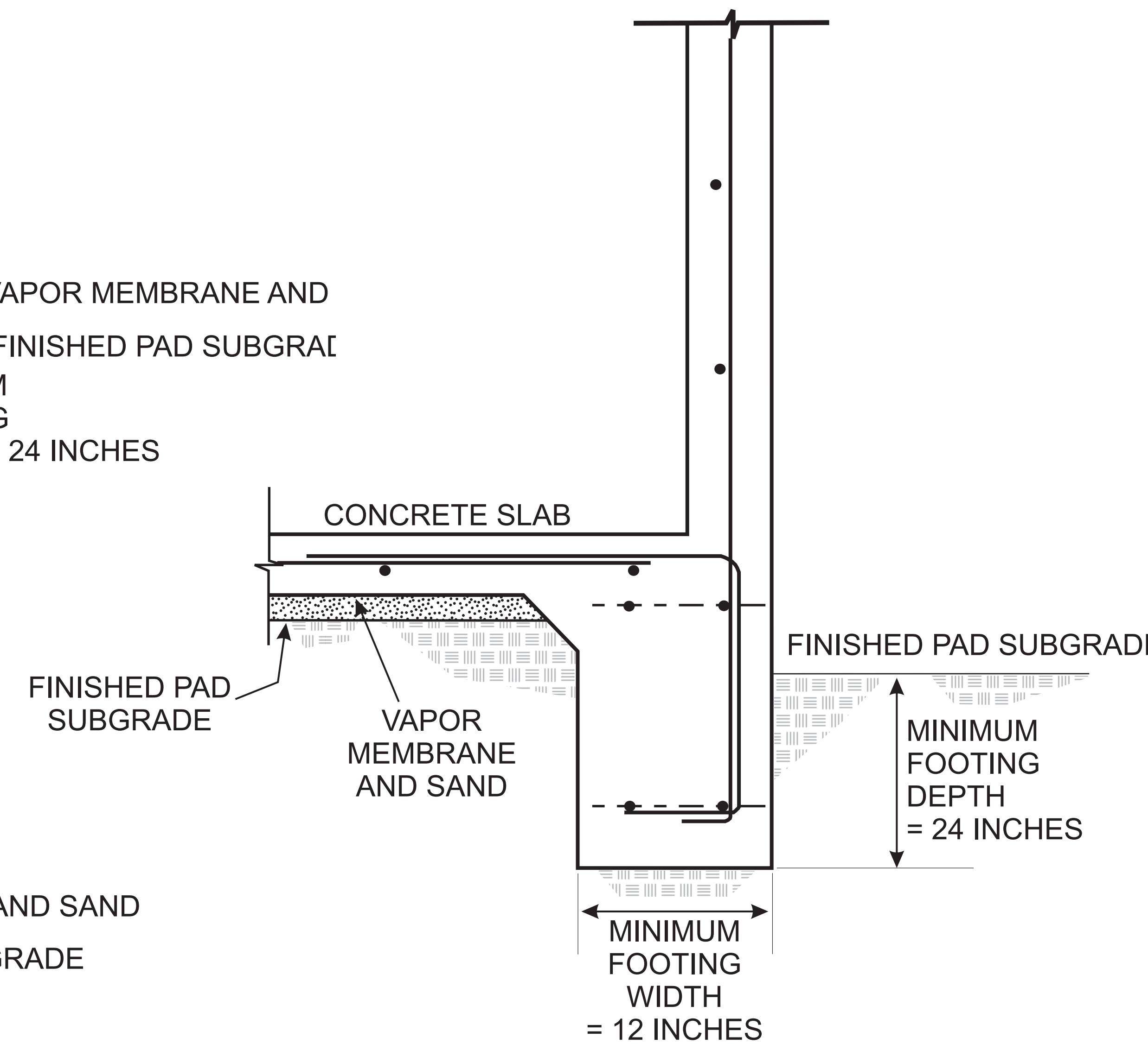
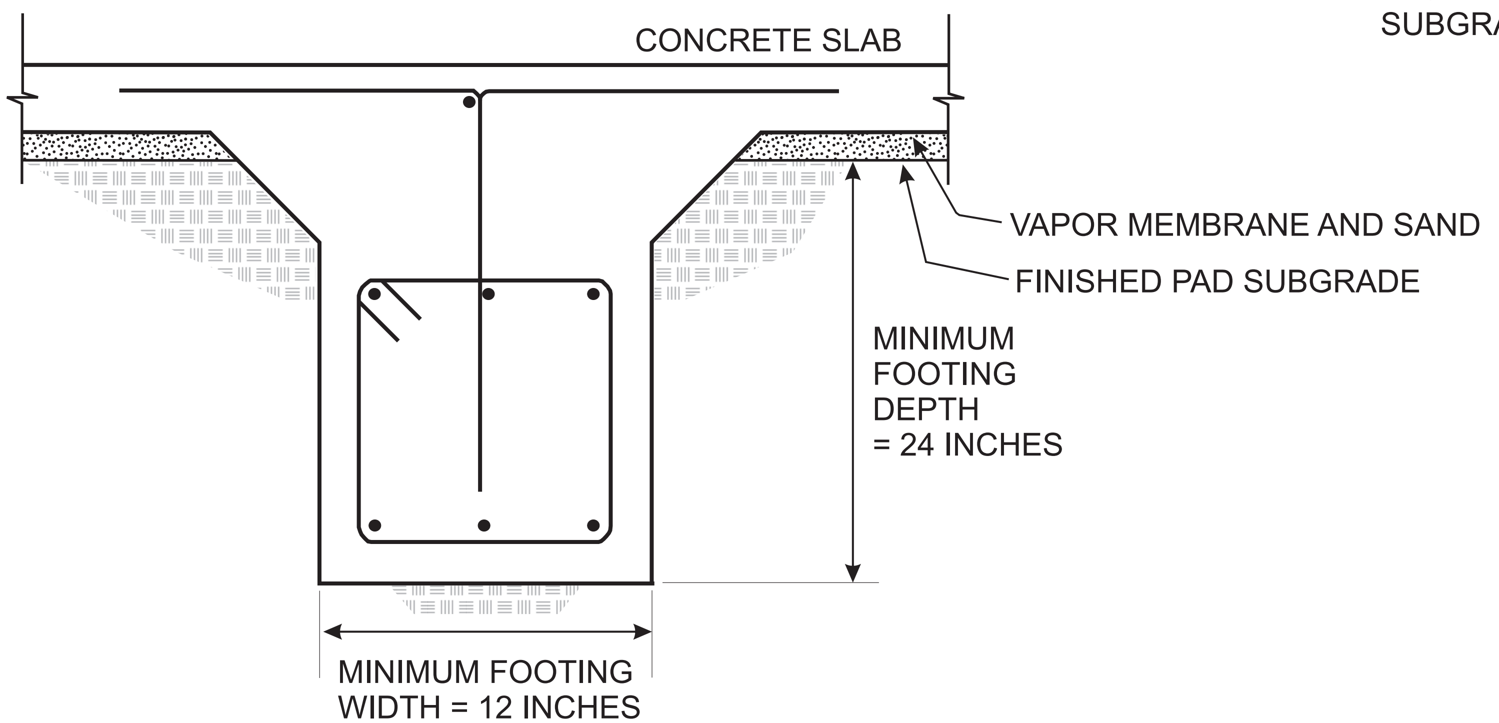
	PROJECT NAME USD Group Biofuels Terminal ECORP Consulting, Inc.	FIGURE NUMBER 5B
	LOCAL FAULT MAP	



SQUARE FOOTING



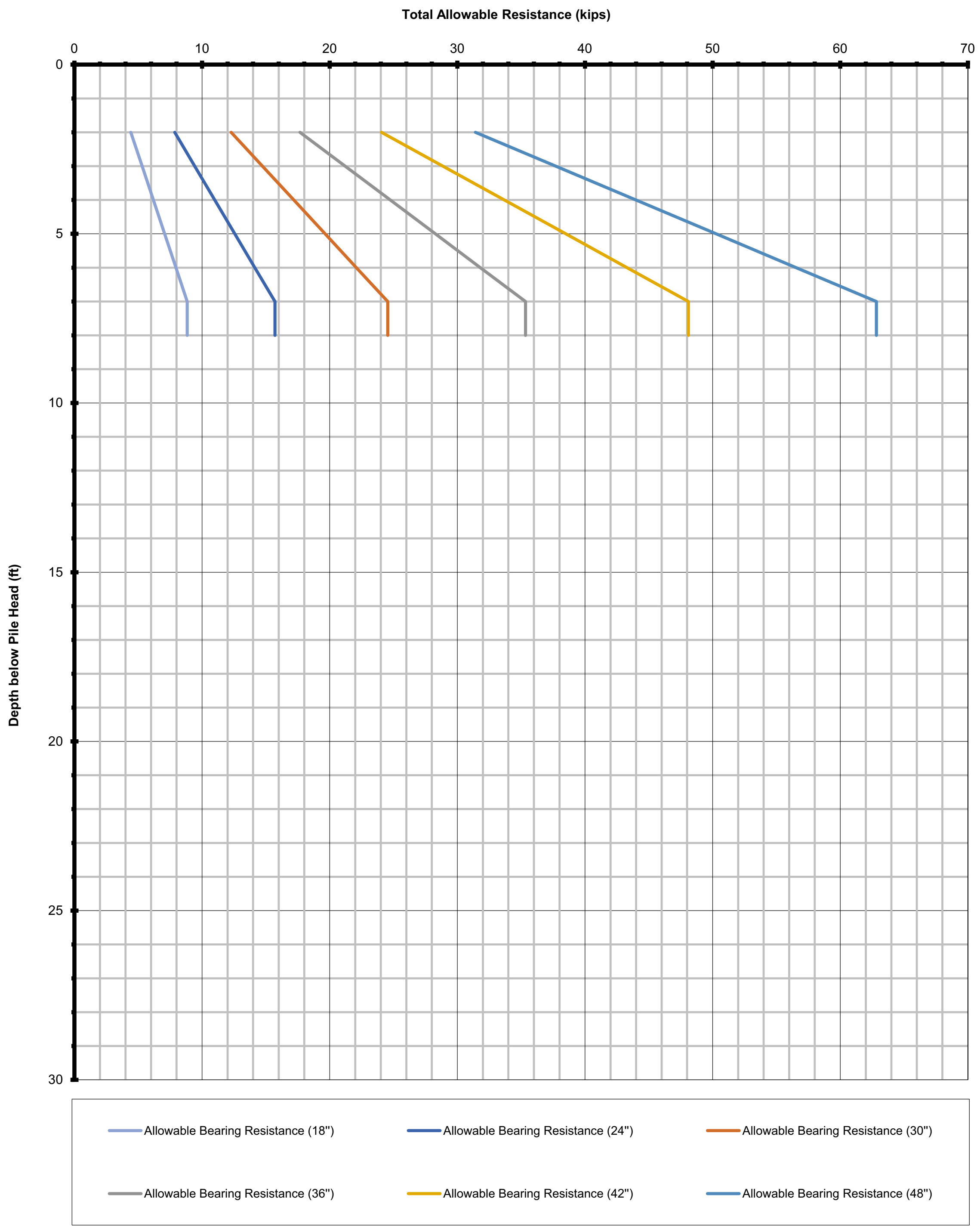
EXTERIOR CONTINUOUS FOOTING



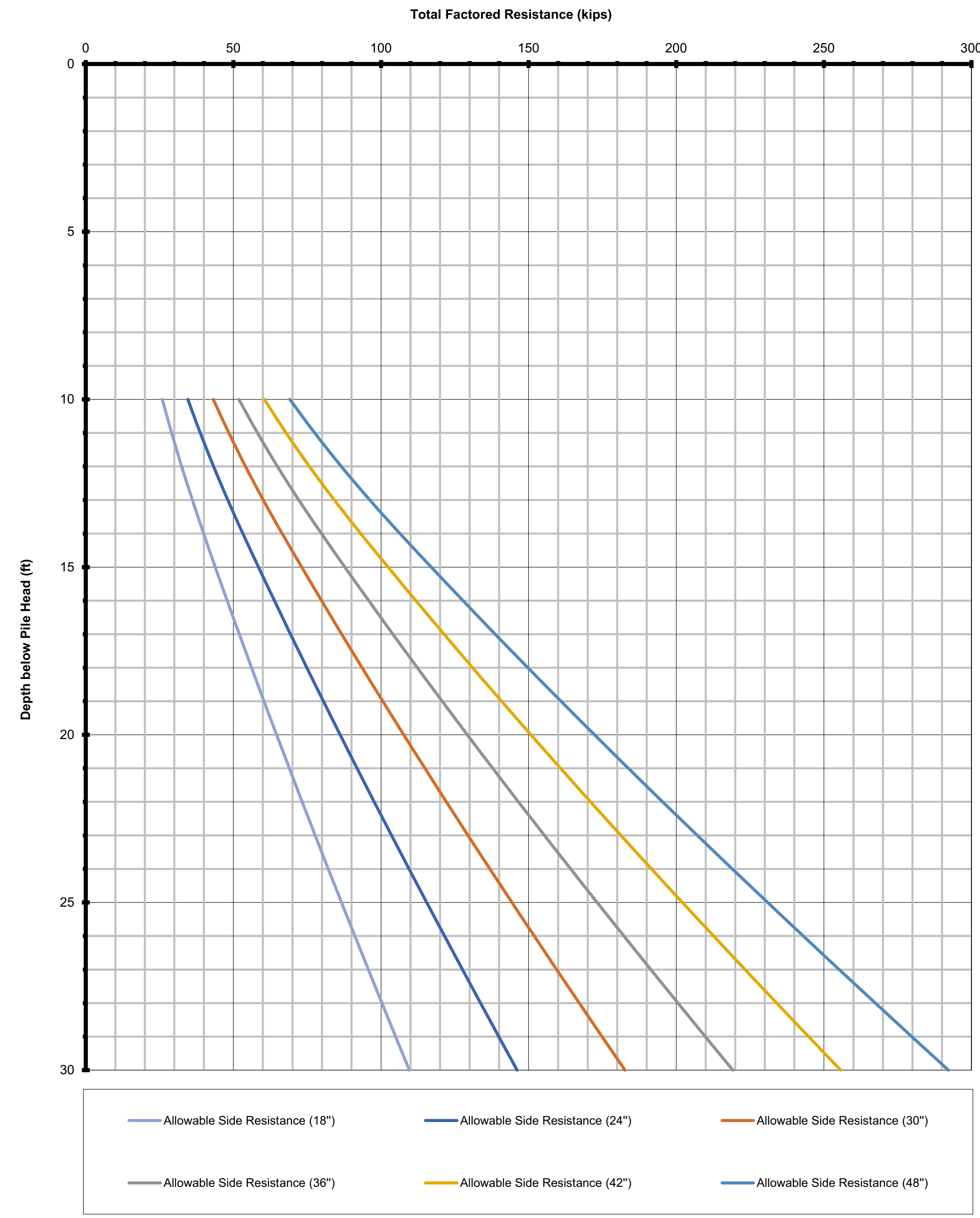
INTERIOR CONTINUOUS FOOTING

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD724 DOCUMENT NUMBER 22-0036
	PROJECT NAME USD Group Biofuels Terminal ECORP Consulting, Inc.	FIGURE NUMBER 6
	SHALLOW FOUNDATIONS	

SHORT PILE CAPACITIES



LONG PILE CAPACITIES



NOTES:

1) Short piles (L<10') are assumed to derive capacity from end bearing in firm and dry soil. A thorough cleaning of the excavation bottoms will be needed.

2) Long piles (L>10') are assumed to derive capacity from skin friction only. Wet methods should be used to stabilize pile excavations below groundwater.

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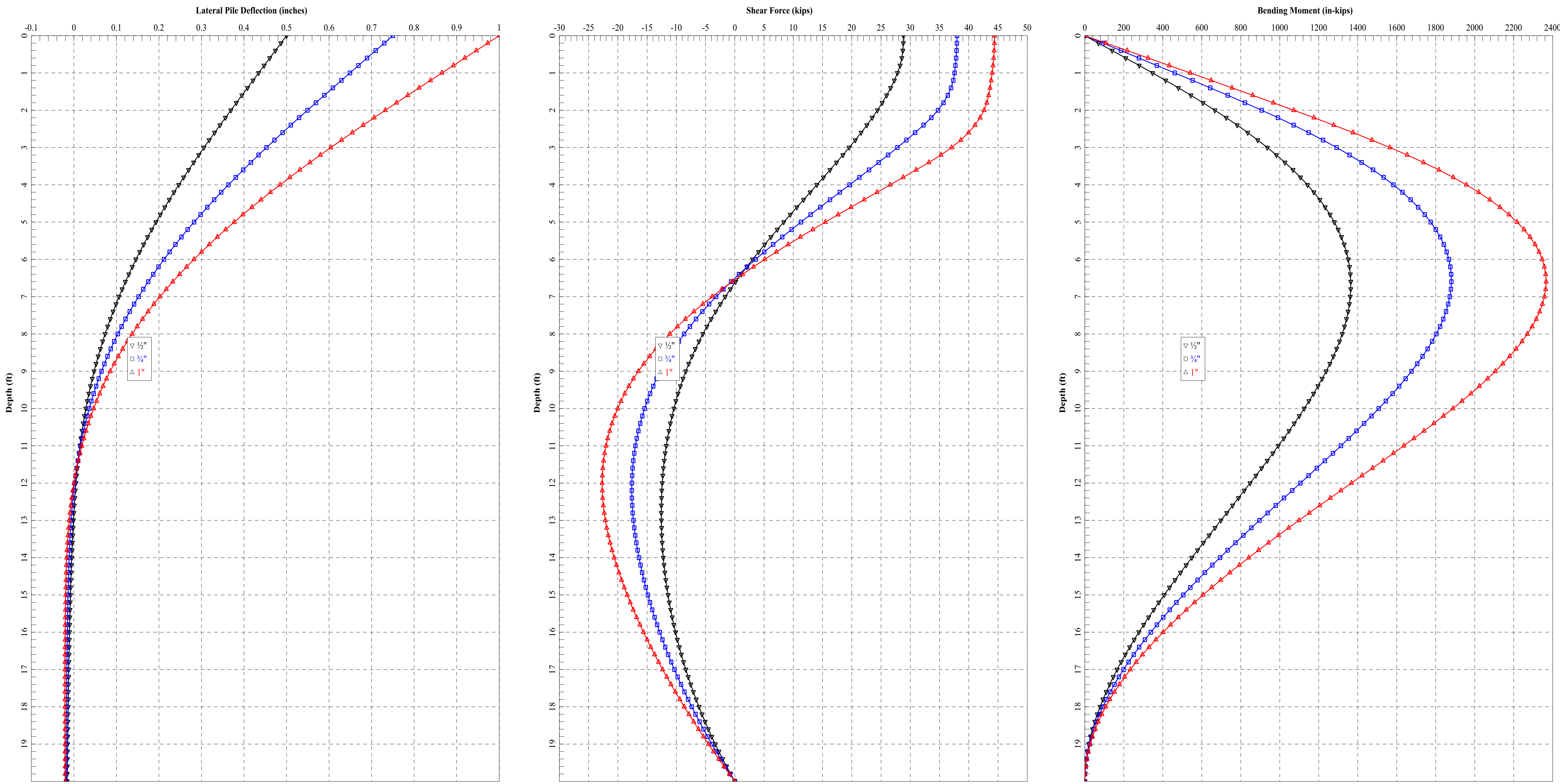
PROJECT NAME	USD Group Biofuels Terminal ECORP Consulting, Inc.
FIGURE NUMBER	7

AXIAL PILE CAPACITY



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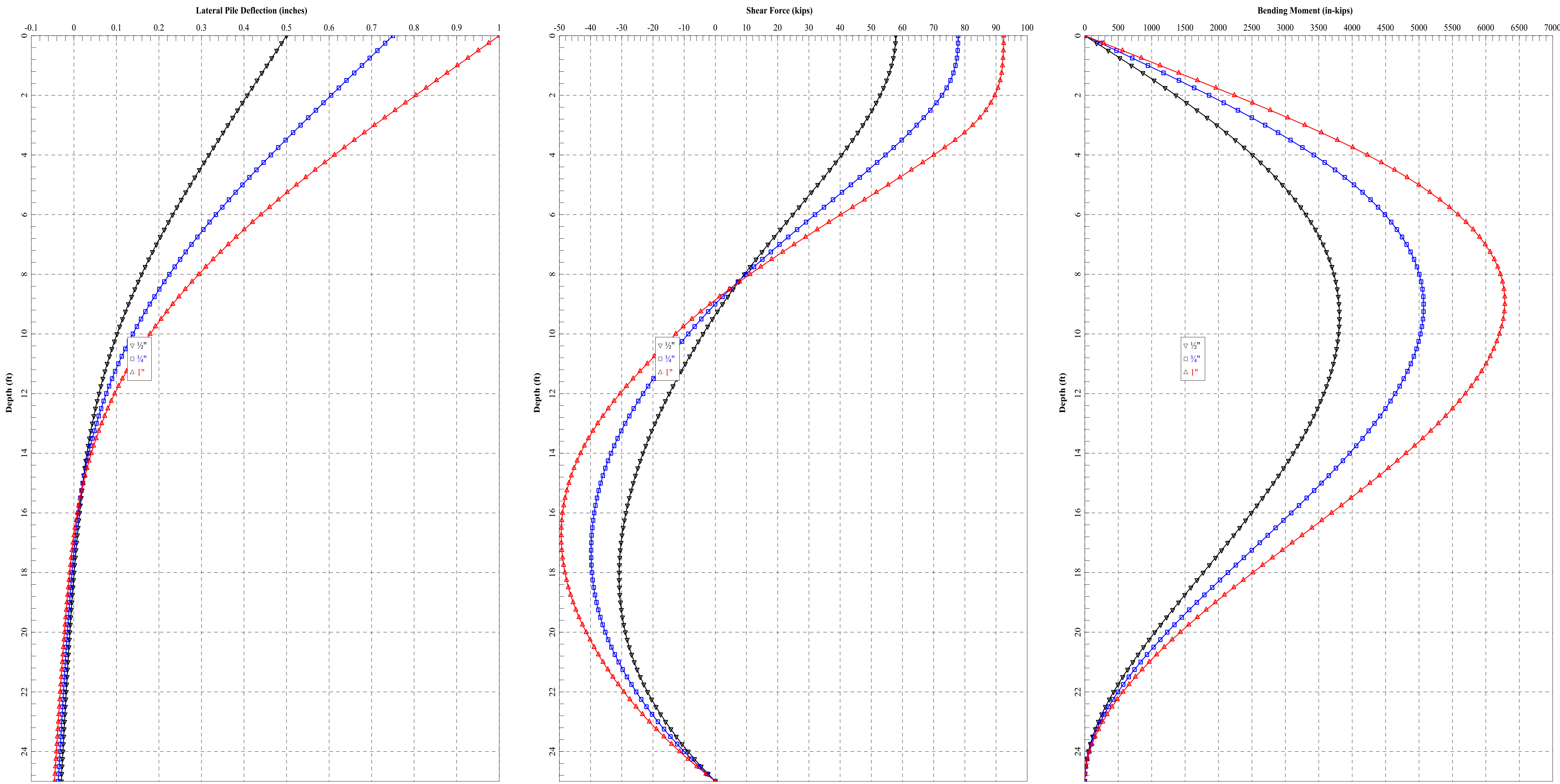
PROJECT NUMBER
SD724

DOCUMENT NUMBER
22-0036

FIGURE NUMBER
8A

LATERAL CAPACITY (2' CIDH)



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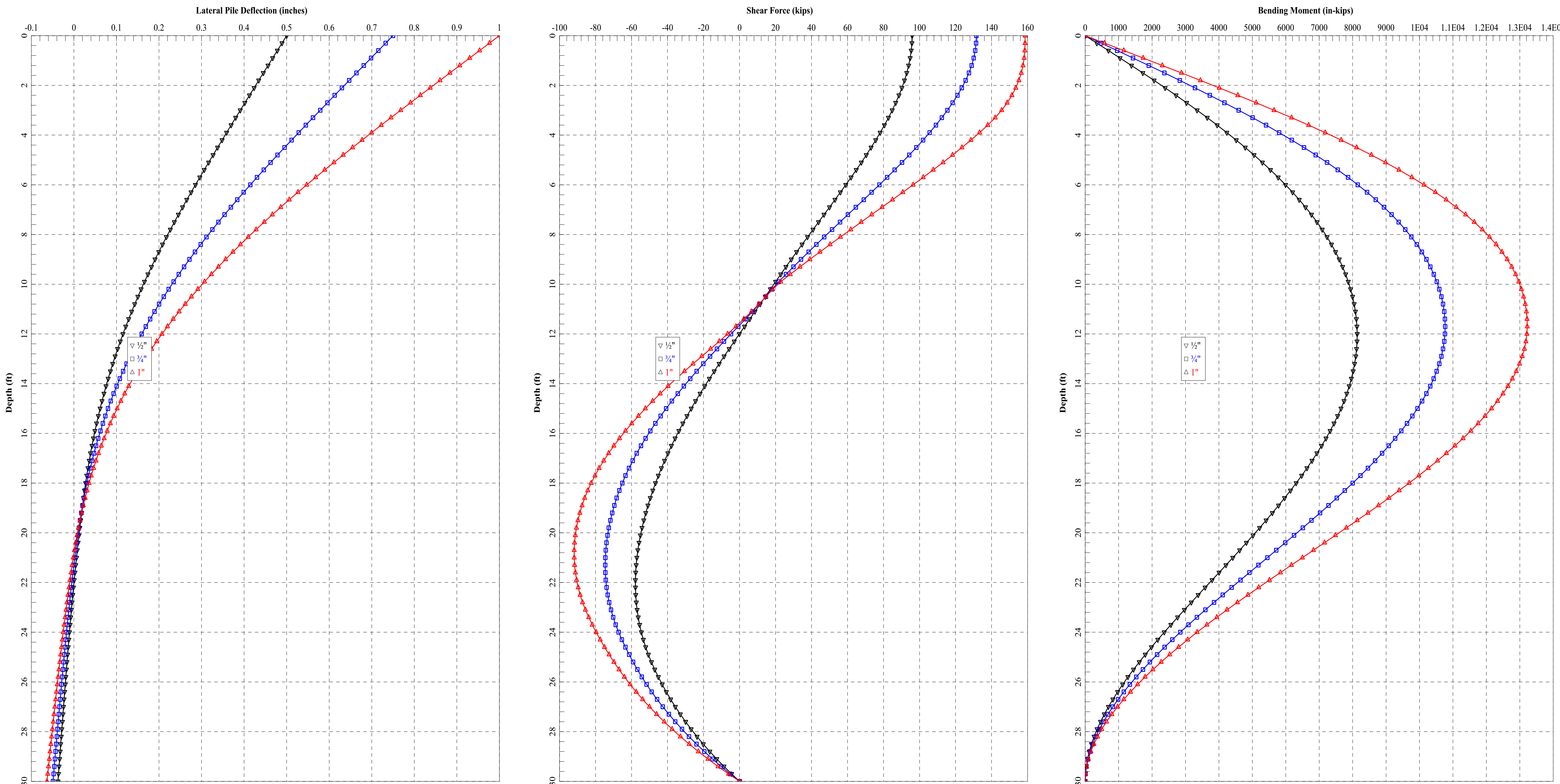
	PROJECT NAME USD Group Biofuels Terminal ECORP Consulting, Inc.	FIGURE NUMBER **8B**
	LATERAL CAPACITY (3' CIDH)	



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PROJECT NUMBER
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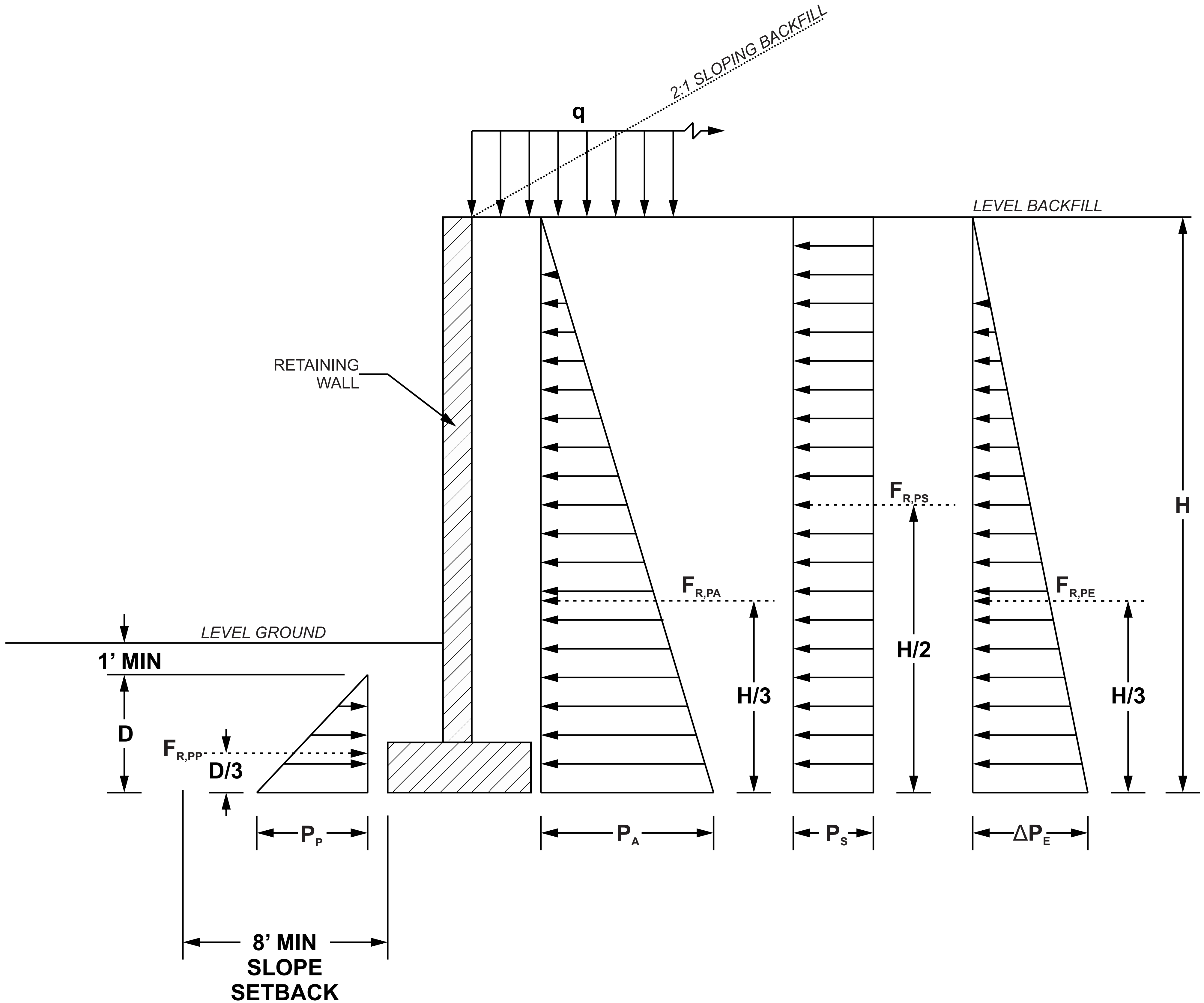
DOCUMENT NUMBER
22-0036

FIGURE NUMBER
8C

LATERAL CAPACITY (4' CIDH)



NOTES:

1. PASSIVE PRESSURES MAY BE INCREASED BY ⅓ DURING SEISMIC LOADING. THE UPPER 12 INCHES OF MATERIAL NOT PROTECTED BY CONCRETE SLABS OR PAVEMENTS SHOULD NOT BE INCLUDED IN THE ESTIMATION OF PASSIVE RESISTANCE.
2. ASSUMES NO HYDROSTATIC PRESSURE. A WALL BACK DRAIN SHOULD BE INSTALLED AS RECOMMENDED IN THE *WALL DRAINAGE DETAIL* FIGURE.
3. SURCHARGES FROM CONSTRUCTION EQUIPMENT, EXCAVATED SOIL, TRAFFIC LOADING OR OTHER UNIFORM LOADING ABOVE THE WALL SHOULD BE CALCULATED USING THE SURCHARGE LATERAL EARTH PRESSURE, P_S. POINT LOADS OR OTHER SURCHARGES CAN BE EVALUATED UPON REQUEST.
4. SEISMIC INCREMENT LATERAL EARTH PRESSURE (ΔP_E) IS BASED ON A DESIGN-LEVEL PEAK GROUND ACCELERATION OF 0.429g. SEISMIC INCREMENT SHOULD BE APPLIED TO WALLS SIX FEET OR GREATER IN HEIGHT.
5. 'H' AND 'D' ARE MEASURED IN FEET.
6. PRESSURES ASSUME GRANULAR AND NON-EXPANSIVE SOIL MATERIALS COMPACTED AS RECOMMENDED IN THE GEOTECHNICAL REPORT.

LATERAL EARTH PRESSURES

LATERAL EARTH PRESSURE TYPE	EQUIVALENT FLUID PRESSURE (PCF)	
ACTIVE, P_A	LEVEL BACKFILL	2:1 SLOPING BACKFILL
COMPACTED FILL	35	55
SEISMIC INCREMENT, ΔP_E*	26	
PASSIVE, P_P**	300	
SURCHARGE, P_S	0.3q	

*SEISMIC PRESSURE, $P_{AE} = P_A + \Delta P_E$
**PASSIVE RESISTANCE VERSUS DISPLACEMENT CURVES CAN BE PROVIDED UPON REQUEST.

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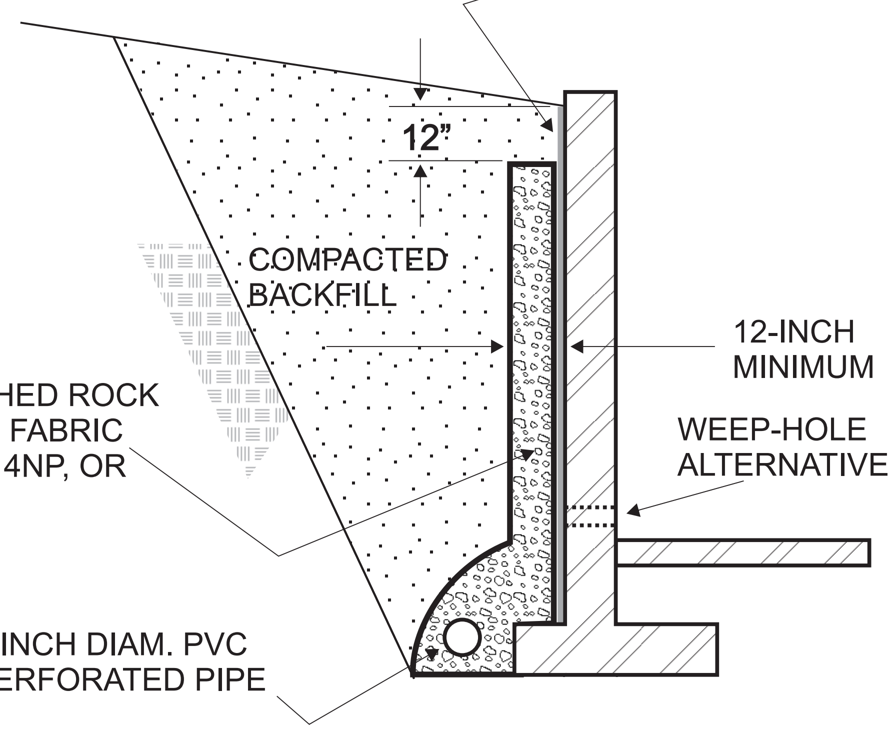
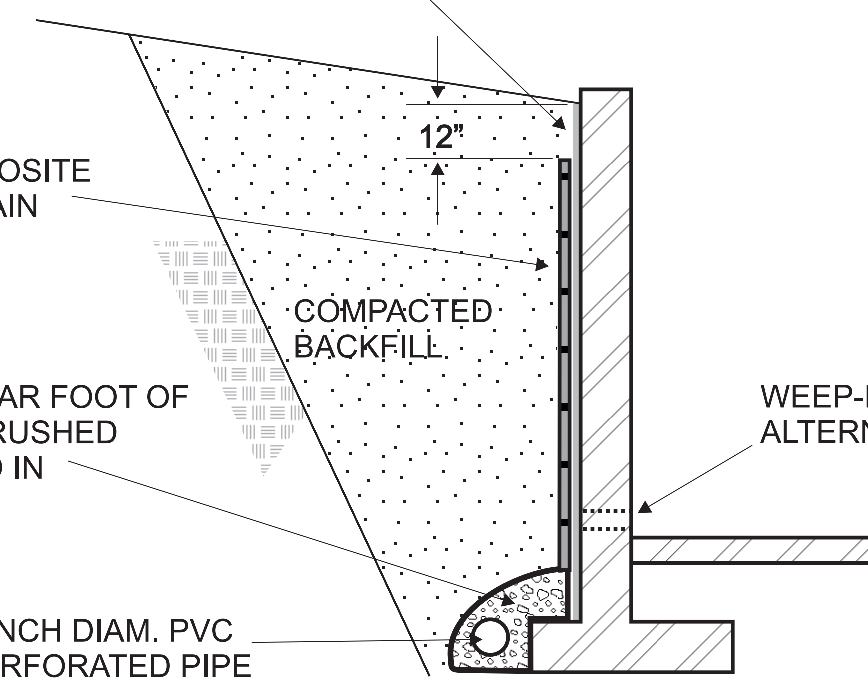
DOCUMENT NUMBER 22-0036
PROJECT NAME USD Group Biofuels Terminal ECORP Consulting, Inc.
FIGURE NUMBER 9A

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

ROCK AND FABRIC ALTERNATIVE



PANEL DRAIN ALTERNATIVE



NOTES

1) Perforated pipe should outlet through a solid pipe to a free gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.

2) As an alternative to the perforated pipe and outlet, weep-holes may be constructed. Weep-holes should be at least 2 inches in diameter, spaced no greater than 8 feet, and be located just above grade at the bottom of wall.

3) Filter fabric should consist of Mirafi 140N, Supac 5NP, Amoco 4599, or similar approved fabric. Filter fabric should be overlapped at least 6-inches.

4) Geocomposite panel drain should consist of Miradrain 6000, J-DRain 400, Supac DS-15, or approved similar product.

GROUP DELTA

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PROJECT NAME USD Group Biofuels Terminal ECORP Consulting, Inc.	DOCUMENT NUMBER 22-0036
	FIGURE NUMBER 9B
WALL DRAINAGE DETAILS	

APPENDIX A
FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Field exploration included a visual reconnaissance of the site, the drilling of 6 exploratory borings and the advancement of 6 cone penetration tests (CPT) between March 22nd and 23rd, 2022. The borings were drilled by Pacific Drilling using their Marl M10 (Yeti) truck mounted drill rig using both a 6 and 8-inch diameter hollow stem flight auger. The maximum depth of exploration was about 31½ feet below grade. The approximate boring and CPT locations are shown on the Exploration Plan, Figure 3. Boring logs are provided in Figures A-1 to A-6, after the Boring Record Legends.

Disturbed samples were collected from the borings using a 2-inch outside diameter Standard Penetration Test (SPT) sampler. Less disturbed samples were collected using a 3-inch diameter ring lined sampler (a modified California sampler). These samples were sealed in plastic bags, labeled, and returned to the laboratory for testing. The drive samples were obtained using an automatic hammer with a calibrated Energy Transfer Ratios (ETR) of about 92 percent. For each sample, the number of blows needed to drive the sampler for each 6-inch depth increment was recorded on the logs. The total number of blows needed to drive each sample 12 inches was then recorded as the equivalent SPT blow count (N). The field blow counts (N) were corrected to reflect a standard 60 percent ETR (N_{60}), as shown on the logs. Bulk soil samples were also collected from the borings.

The CPT soundings were also advanced by Pacific Drilling using a 10 cm^2 cone in general accordance with ASTM D5778. Integrated electronic circuitry was used to measure the tip resistance (Qc) and skin friction (Fs) at 2.5 cm (1 inch) intervals while the CPT was advanced into the soil with hydraulic down pressure. A piezometer located behind the cone tip measured transient pore pressure (u). A color-coded log showing the interpreted soil profile is provided for each CPT sounding, based on the normalized cone resistance and friction ratio (Robertson, 2010). The raw CPT data and estimated undrained shear strengths for the clay layers are also shown after each interpreted soil profile. The CPT data and interpretations are presented after the logs in Figures A-7 to A-12.

Boring No.	Drill Date	Surface Elevation	Total Depth	Bottom Elevation	Approximate Latitude	Approximate Longitude	Figure No.
B-1	03/23/22	17′	21½′	-4½′	32.664124°	-117.112183°	A-1
B-2	03/23/22	16′	31½′	-15½′	32.664274°	-117.112647°	A-2
B-3	03/23/22	17′	21′	-4′	32.663949°	-117.112666°	A-3
B-4	03/23/22	16′	21½′	-5½′	32.664785°	-117.112909°	A-4
B-5	03/23/22	16′	6′	10′	32.664521°	-117.112342°	A-5
B-6	03/23/22	15′	21½′	-6½′	32.664751°	-117.113163°	A-6
CPT-1	03/22/22	17′	6′	11′	32.664313°	-117.112478°	A-7
CPT-2	03/22/22	17′	24′	-7′	32.664452°	-117.112863°	A-8
CPT-3	03/22/22	16′	18½′	-2½′	32.664943°	-117.113055°	A-9
CPT-4	03/22/22	18′	23½′	-5½′	32.663881°	-117.112910°	A-10
CPT-5	03/22/22	17′	26′	-9′	32.664440°	-117.113124°	A-11
CPT-6	03/22/22	15′	26½′	-11½′	32.665155°	-117.113391°	A-12

APPENDIX A

FIELD EXPLORATION (Continued)

The boring and CPT locations were determined by visually estimating, pacing and taping distances from landmarks shown on the Exploration Plan, Figure 3. The locations shown should not be considered more accurate than is implied by the method of measurement used and the scale of the map. The lines designating the interface between differing soil materials on the logs may be abrupt or gradational. Further, soil conditions at locations between the excavations may be substantially different from those at the specific locations we explored. It should be noted that the passage of time may also result in changes in the soil conditions reported in the logs.

SOIL IDENTIFICATION AND DESCRIPTION SEQUENCE

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

Describe the soil using descriptive terms in the order shown

Minimum Required Sequence:

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

⬭ = optional for non-Caltrans projects

Where applicable:

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

REFERENCE: 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 and Description

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

Project No. SD724

USD Group Biofuels Terminal
ECORP Consulting, Inc.

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 TESTING

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

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler (2.4" ID, 3" OD)
- 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)

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 located.	
Estimated Material Change	Change in material cannot be accurately located either because the change is gradational or because of limitations of the drilling and sampling methods.	
Soil / Rock Boundary	Material changes from soil characteristics to rock characteristics.	

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

GROUP DELTA

Project No. SD724

USD Group Biofuels Terminal
ECORP Consulting, Inc.

BORING RECORD LEGEND #2

CONSISTENCY OF COHESIVE SOILS				
Description	Shear Strength (tsf)	Pocket Penetrometer, PP, Measurement (tsf)	Torvane, TV, Measurement (tsf)	Vane Shear, VS, Measurement (tsf)
Very Soft	Less than 0.12	Less than 0.25	Less than 0.12	Less than 0.12
Soft	0.12 - 0.25	0.25 - 0.5	0.12 - 0.25	0.12 - 0.25
Medium Stiff	0.25 - 0.5	0.5 - 1	0.25 - 0.5	0.25 - 0.5
Stiff	0.5 - 1	1 - 2	0.5 - 1	0.5 - 1
Very Stiff	1 - 2	2 - 4	1 - 2	1 - 2
Hard	Greater than 2	Greater than 4	Greater than 2	Greater than 2

APPARENT DENSITY OF COHESIONLESS SOILS	
Description	SPT N_{60} (blows / 12 inches)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Greater than 50

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

PERCENT OR PROPORTION OF SOILS	
Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 - 10%
Little	15 - 25%
Some	30 - 45%
Mostly	50 - 100%

PARTICLE SIZE		
Description		Size (in)
Boulder		Greater than 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		Less than 1/300

CEMENTATION	
Description	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.

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

Plasticity

Description	Criteria
Nonplastic	A 1/8-in. 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. The thread 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	
Description	SPT N_{60} (blows/12 inches)
Very Soft	0 - 2
Soft	2 - 4
Medium Stiff	4 - 8
Stiff	8 - 15
Very Stiff	15 - 30
Hard	Greater than 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 Classification Manual, 2010.

GROUP DELTA

Project No. SD724

USD Group Biofuels Terminal
ECORP Consulting, Inc.

BORING RECORD LEGEND #3

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
ECORP USD Group Biofuels Terminal	SD724	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
837 19th Street, National City, California	3/23/2022	3/23/2022	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling Company	Hollow Stem Auger	S. Narveson	M. Fagan

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl M10	6	21.5	17	▼ 16.5 / *0.5*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DESCRIPTION AND CLASSIFICATION
0	17									**PAVEMENT:** Asphalt concrete (3"), no base.
0.5–4	15	Bulk	B-1						PA PI CR EI~0	**FILL:** CLAYEY SAND WITH GRAVEL (SC); dense; dark brown (75YR 3/2); moist; mostly fines; some fine SAND; little GRAVEL; low plasticity. (17% Gravel; 40% Sand; 43% Fines) (LL~22; PL~11; PI~11)
4–10										LEAN CLAY WITH SAND (CL); hard; brown (7.5Y 4/3); moist; mostly fines; little SAND; trace GRAVEL; low to medium plasticity. Dark stains observed on GRAVEL.
5		Ring	R-2	8 17 21	38	39	20.6	105	DS	
10	10									
10–16.5										**ALLUVIUM (Qya):** Poorly-graded SAND with SILT (SP-SM); medium dense; brown (7.5YR 4/3); moist; mostly fine to medium SAND; few fines; nonplastic; slightly micaceous.
10		SPT	S-3	3 7 9	16	24				
15	5	Ring	R-4	6 9 11	20	20	7.1	99		
16.5–21.5	0									Well-graded SAND (SW); medium dense; light brown (7.5YR 6/3); saturated; mostly fine to coarse SAND; trace fines; trace GRAVEL; nonplastic.
20		SPT	S-5	6 6 12	18	28				
21.5	-5									Boring terminated at 21½ feet. Groundwater measured at 16½ feet. Boring backfilled using bentonite grout.

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FIGURE A-1

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
ECORP USD Group Biofuels Terminal	SD724	**B-2**

SITE LOCATION	START	FINISH	SHEET NO.
837 19th Street, National City, California	3/23/2022	3/23/2022	1 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling Company	Hollow Stem Auger	S. Narveson	M. Fagan

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl M10	8	31.5	16	▼ 14.5 / *1.5*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0–5	15	Bulk	B-1						PA			**FILL:** SANDY LEAN CLAY (CL); stiff; brown (75YR 4/3); moist; mostly fines; some fine to coarse SAND; few GRAVEL; low to medium plasticity. (5% Gravel; 43% Sand; 52% Fines)
5	10	Ring	R-2	2 8 14	22	22	14.4	117	DS	5		SANDY SILT (ML); very stiff; brown (75YR 4/3); moist; mostly fines; little SAND; trace GRAVEL; low plasticity.
10	5	SPT	S-3	7 9 10	19	29				10		**ALLUVIUM (Qya):** Poorly-graded SAND with SILT (SP-SM); medium dense; brown (7.5YR 4/3); moist; mostly fine to medium SAND; few fines; nonplastic; slightly micaceous.
15	0	Ring	R-4	10 14 20	34	35	10.5	109	DS	15		Well-graded SAND (SW); dense; brown (7.5Y 5/3); saturated; mostly fine to coarse SAND; trace fines; trace fine GRAVEL; nonplastic.
20	-5	SPT	S-5	6 10 13	23	35				20		**VERY OLD PARALIC DEPOSITS (Qvop6):** SILTY SANDTONE (SM); dense; grayish brown (10YR 5/2); saturated; mostly fine SAND; some fines; low plasticity.

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FIGURE A-2 a

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
ECORP USD Group Biofuels Terminal	SD724	B-2

SITE LOCATION	START	FINISH	SHEET NO.
837 19th Street, National City, California	3/23/2022	3/23/2022	2 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling Company	Hollow Stem Auger	S. Narveson	M. Fagan

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl M10	8	31.5	16	▼ 14.5 / *1.5*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	-10		R-6	4 15 21	36	37	---	---				**<u>VERY OLD PARALIC DEPOSITS (Qvop6)</u>:** SILTY SANDTONE (SM); dense; grayish brown (10YR 5/2); saturated; mostly fine to medium SAND; some fines; low plasticity; slightly micaceous.
			S-7	2 6 14	20	30						
30										30		
	-15		S-8	13 14 14	28	43						
												Boring terminated at 31½ feet. Groundwater measured at 14½ feet. Boring backfilled using bentonite grout.
35	-20									35		
40	-25									40		
45	-30									45		

GDC_LOG_BORING_MMX_SOIL_SD SD724 LOGS.GPJ GDCLOG.GDT 4/14/22

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
ECORP USD Group Biofuels Terminal	SD724	B-3

SITE LOCATION	START	FINISH	SHEET NO.
837 19th Street, National City, California	3/23/2022	3/23/2022	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling Company	Hollow Stem Auger	S. Narveson	M. Fagan

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl M10	6	21	17	▼ 15.5 / *1.5*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	15		B-1						PA EI~25			**FILL:** SANDY LEAN CLAY (CL); stiff; reddish brown (5YR 5/3); moist; mostly fines; some fine to medium SAND; trace GRAVEL; low to medium plasticity. (4% Gravel; 45% Sand; 51% Fines)
5	10		S-2	4 5 5	10	15				5		SANDY SILT (ML); very stiff; brown (5YR 5/3); moist; mostly fines; little SAND; trace GRAVEL; low plasticity.
10	5		R-3	9 9 11	20	20	4.3	105		10		**ALLUVIUM (Qya):** Poorly-graded SAND (SP); medium dense; brown (7.5YR 5/3); moist; mostly fine to medium SAND; trace fines; nonplastic.
15	0		S-4	4 5 6	11	17				15		Well-graded SAND (SW); medium dense; gray (7.5Y 5/1); saturated; mostly fine to coarse SAND; trace fines; trace fine GRAVEL; nonplastic.
20	-5		S-5	8 36	44	67				20		Dense.
												Boring terminated at 21 feet. Groundwater measured at 15 feet. Boring backfilled using bentonite grout.

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FIGURE A-3

BORING RECORD

PROJECT NAME	ECORP USD Group Biofuels Terminal
PROJECT NUMBER	SD724
BORING	B-4
SITE LOCATION	837 19th Street, National City, California
START	3/23/2022
FINISH	3/23/2022
SHEET NO.	1 of 1
DRILLING COMPANY	Pacific Drilling Company
DRILLING METHOD	Hollow Stem Auger
LOGGED BY	S. Narveson
CHECKED BY	M. Fagan
DRILLING EQUIPMENT	Marl M10
BORING DIA. (in)	8
TOTAL DEPTH (ft)	21.5
GROUND ELEV (ft)	16
DEPTH/*ELEV.* GROUND WATER (ft)	▼ 15.0 / *1.0*
SAMPLING METHOD	Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	15	Bulk	B-1						PA PI EI~0	SC	**FILL:** CLAYEY SAND WITH GRAVEL (SC); dense; brown (10YR 4/2); moist; mostly fine SAND; some fines; little GRAVEL; low plasticity. Contains some glass and plastic debris. (17% Gravel; 45% Sand; 38% Fines) (LL~21; PL~13; PI~8)
5	10	Ring	R-2	9 16 21	37	38	11.5	118		CL	LEAN CLAY WITH SAND (CL); hard; reddish brown (5Y 4/3); moist; mostly fines; little SAND; low to medium plasticity. Contains metal and fabric debris.
10	5	SPT	S-3	5 11 12	23	35				ML	**ALLUVIUM (Qya):** SANDY SILT (ML); dense; brown (7.5YR 5/3); moist; mostly fines; some fine SAND; nonplastic; slightly micaceous.
12										SM	SILTY SAND (SM); dense; grayish brown (10YR 5/2); moist; mostly fines; some fine SAND; nonplastic; slightly micaceous.
15	0	Ring	R-4	7 13 15	28	29	18.8	108		SP-SM	Poorly-graded SAND with SILT (SP-SM); medium dense; gray (7.5Y 5/1); saturated; mostly fine to medium SAND; trace fines; nonplastic; slightly micaceous.
20	-5	SPT	S-5	5 7 11	18	28					
21.5											Boring terminated at 21½ feet. Groundwater measured at 16 feet. Boring backfilled using bentonite grout.

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FIGURE A-4

BORING RECORD

PROJECT NAME: ECORP USD Group Biofuels Terminal	PROJECT NUMBER: SD724	BORING: **B-5**
SITE LOCATION: 837 19th Street, National City, California	START: 3/23/2022	FINISH: 3/23/2022
SHEET NO.: 1 of 1		
DRILLING COMPANY: Pacific Drilling Company	DRILLING METHOD: Hollow Stem Auger	LOGGED BY: S. Narveson
CHECKED BY: M. Fagan		
DRILLING EQUIPMENT: Marl M10	BORING DIA. (in): 8	TOTAL DEPTH (ft): 6
GROUND ELEV (ft): 16	DEPTH/*ELEV.* GROUND WATER (ft): N/A / *na*	
SAMPLING METHOD: Hammer: 140 lbs., Drop: 30 in. (Automatic)	NOTES: ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	16											**PAVEMENT:** Asphalt concrete (8"), no base.
	15											**FILL:** SANDY LEAN CLAY (CL); stiff to hard; reddish brown (5Y 4/3); moist; mostly fines; some SAND; low to medium plasticity.
			B-1						PA R~21			(5% Gravel; 44% Sand; 51% Fines)
5			S-2	7 11 12	23	35				5		
	10											Boring terminated at 6 feet. No groundwater encountered. Boring backfilled using bentonite grout.
10	5									10		
15	0									15		
20	-5									20		

GDC_LOG_BORING_MMX_SOIL_SD SD724 LOGS.GPJ GDCLOG.GDT 4/14/22

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BORING RECORD

PROJECT NAME	ECORP USD Group Biofuels Terminal
PROJECT NUMBER	SD724
BORING	B-6
SITE LOCATION	837 19th Street, National City, California
START	3/23/2022
FINISH	3/23/2022
SHEET NO.	1 of 1
DRILLING COMPANY	Pacific Drilling Company
DRILLING METHOD	Hollow Stem Auger
LOGGED BY	S. Narveson
CHECKED BY	M. Fagan
DRILLING EQUIPMENT	Marl M10
BORING DIA. (in)	8
TOTAL DEPTH (ft)	21.5
GROUND ELEV (ft)	15
DEPTH/*ELEV.* GROUND WATER (ft)	▼ 14.5 / *0.5*
SAMPLING METHOD	Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			B-1						PA CR R~23			**FILL:** SANDY LEAN CLAY (CL); stiff; brown (75YR 5/2); moist; mostly fines; some fine SAND; few GRAVEL; low to medium plasticity. Contains some wood and vegetative debris. About 3" of coarse railroad ballast covers the ground surface. (5% Gravel; 44% Sand; 51% Fines)
5	10		R-2	4 8 14	22	22	17.8	111		5		LEAN CLAY (CL); hard; brown (7.5Y 4/3); moist; mostly fines; few fine SAND; medium to high plasticity.
10	5		S-3	7 10 13	23	35				10		**ALLUVIUM (Qya):** SANDY SILT (ML); medium dense to dense; brown (7.5YR 4/2); moist; mostly fines; some fine SAND; low plasticity; slightly micaceous.
												SILTY SAND (SM); dense; grayish brown (10YR 5/2); moist; mostly fines; some fine SAND; nonplastic; slightly micaceous.
15	0		R-4	5 7 15	22	22	---	---		15		Well-graded SAND (SW); medium dense to dense; brown (7.5Y 5/1); saturated; mostly fine to coarse SAND; trace fines; nonplastic.
20	-5		S-5	7 10 15	25	38				20		Dense.
												Boring terminated at 21.5 feet. Groundwater measured at 15 feet. Boring backfilled using bentonite grout.

GDC_LOG_BORING_MMX_SOIL_SD SD724 LOGS.GPJ GDCLOG.GDT 4/14/22

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FIGURE A-6

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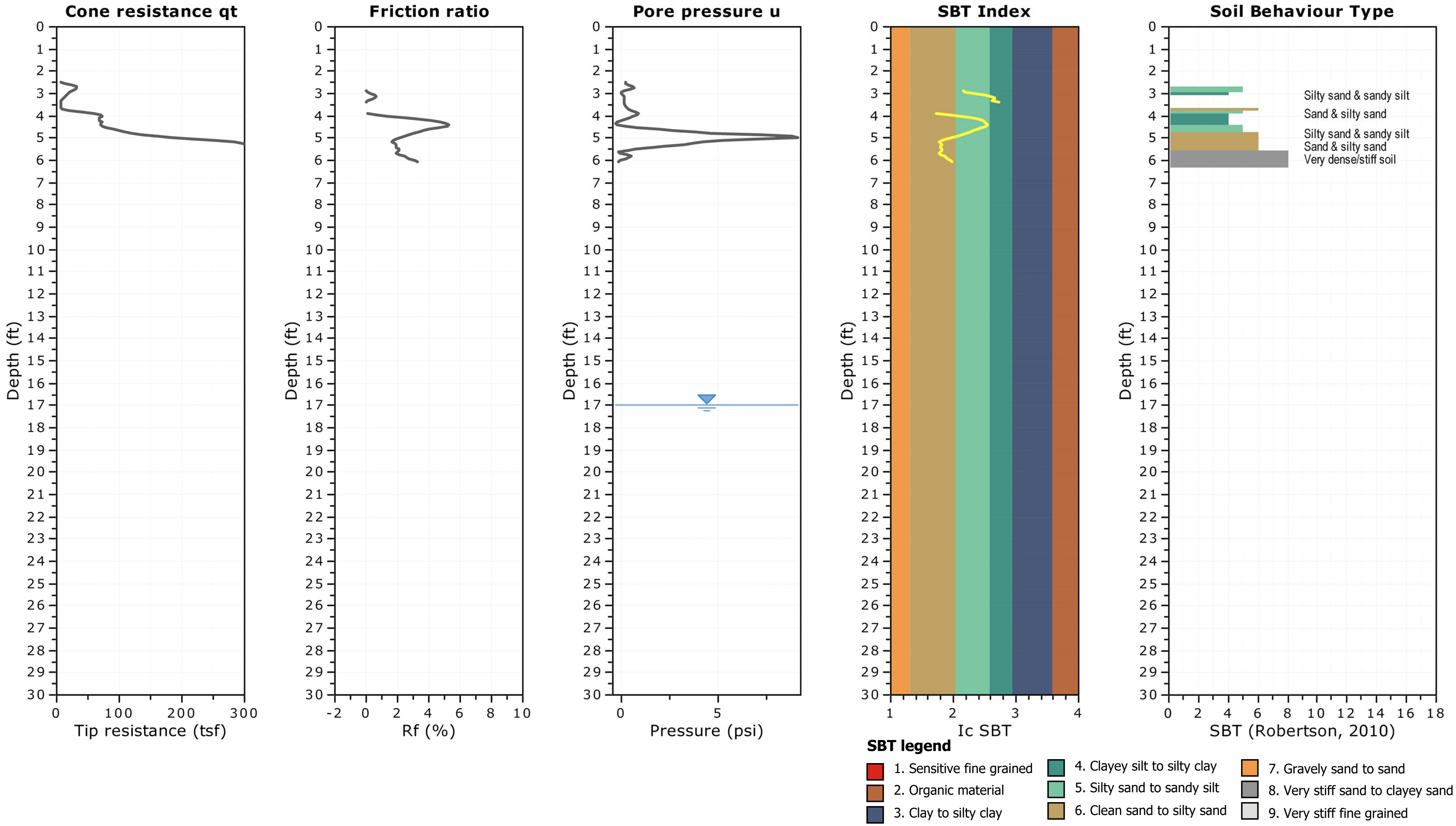
CPT-1

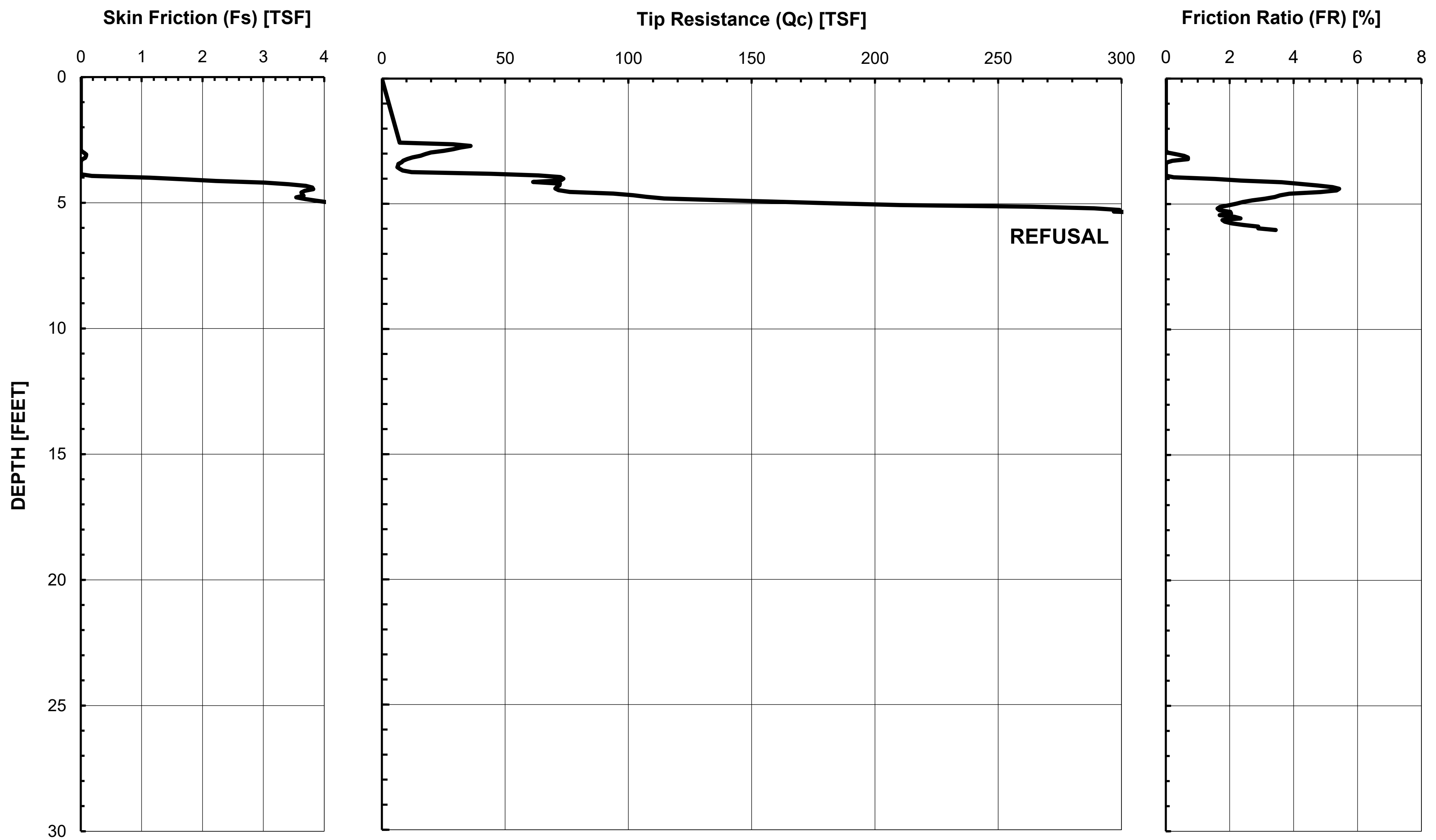
Project: USD Group Biofuels Terminal
Location: 837 19th Street, National City, CA

Total depth: 6.04 ft, Date: 3/23/2022
Surface Elevation: 17.00 ft



CPeT-IT v.3.5.3.3 - CPTU data presentation & interpretation software - Report created on: 4/1/2022, 2:42:54 PM
Project file: N:\Projects\SD\SD700\SD724 ECORP USD Clean Fuels National City Rail Terminal\9. Reports\22-0036\Appendix A\CPeT.cpt

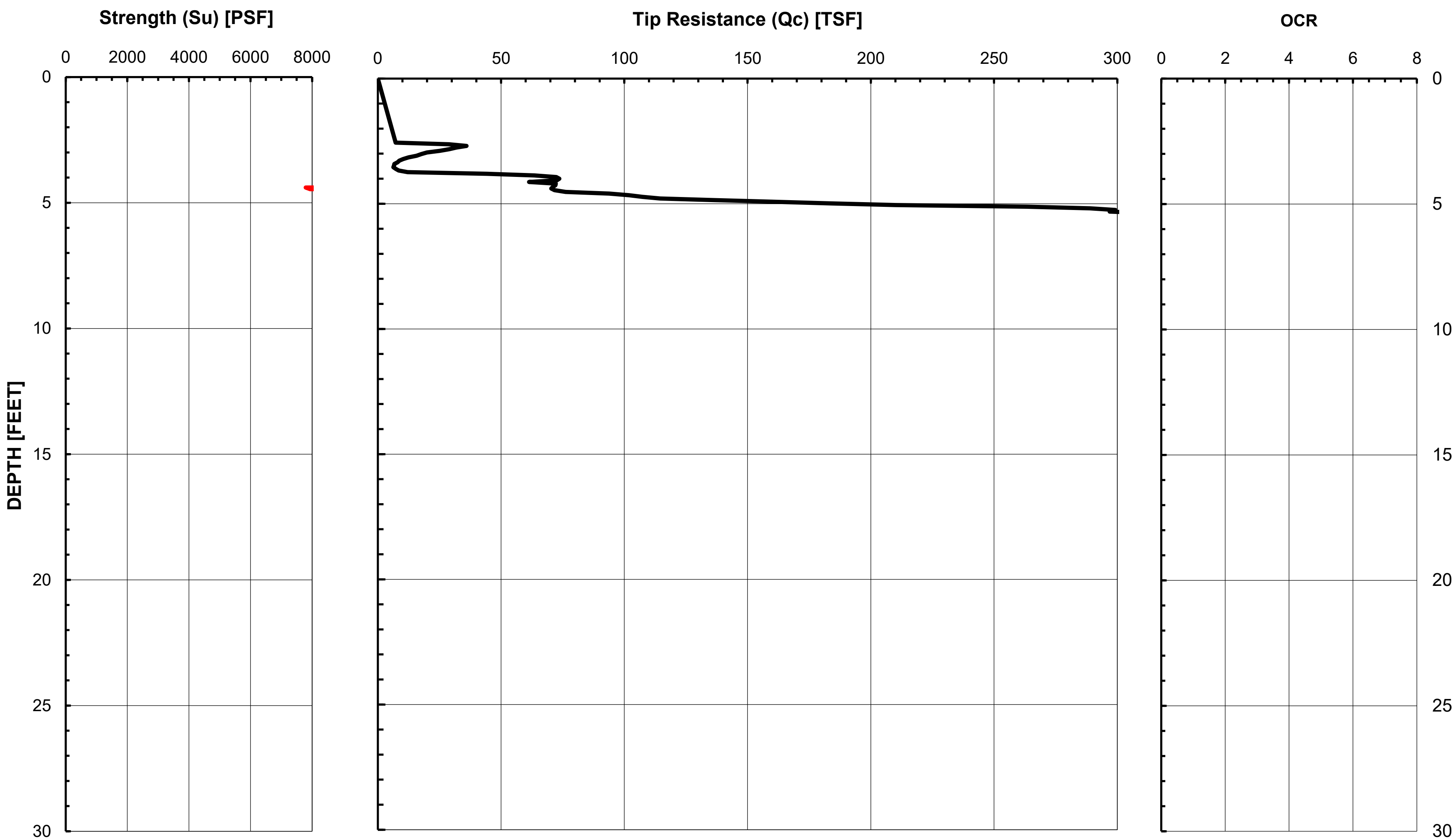
FIGURE A-7



GROUP DELTA

CONE PENETOMETER DATA (CPT-1)

Document No. 22-0036
Project No. SD724
FIGURE A-7a



GROUP DELTA

ESTIMATED STRENGTH AND OCR (CPT-1)

Document No. 22-0036
Project No. SD724
FIGURE A-7b

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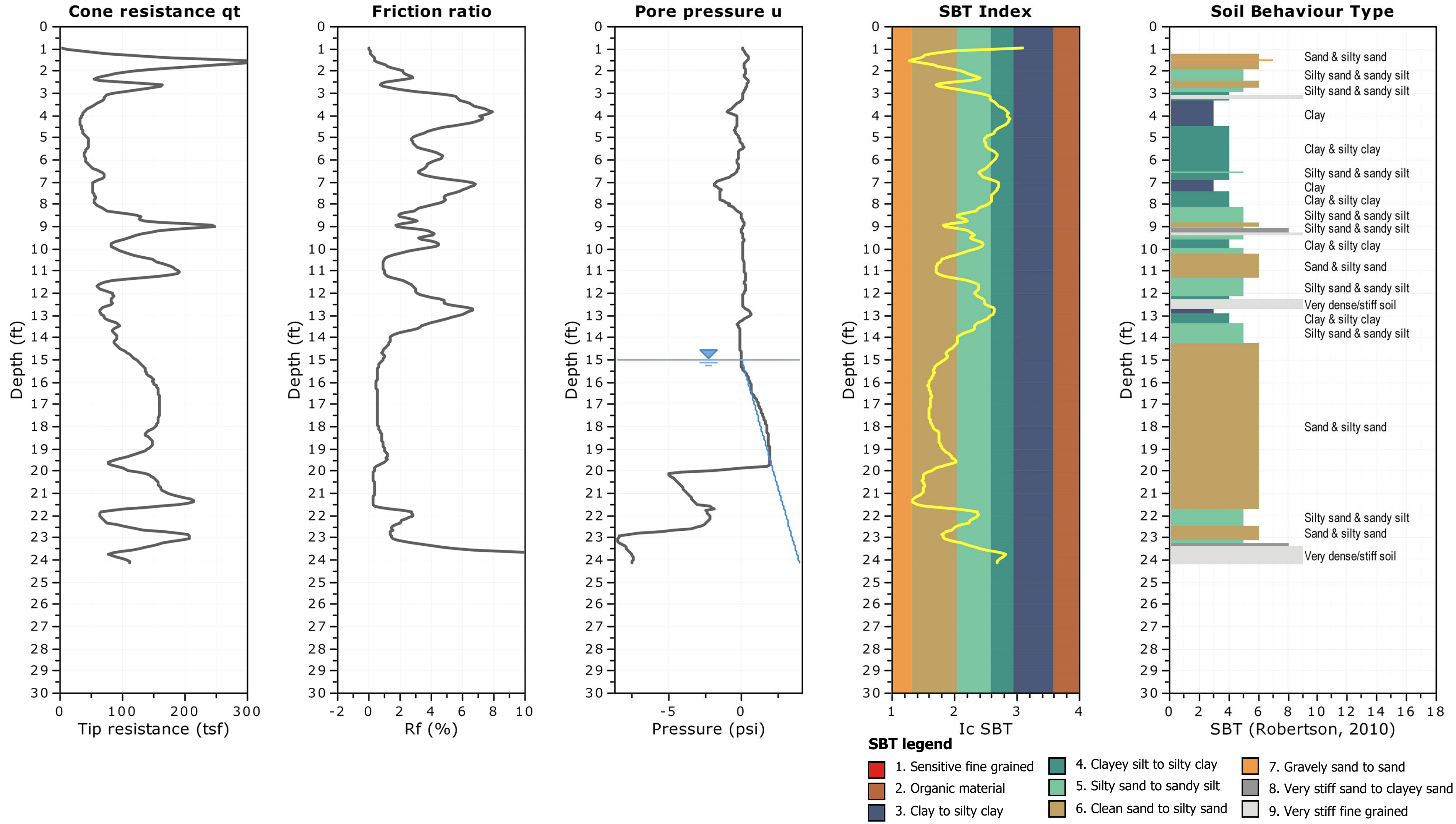
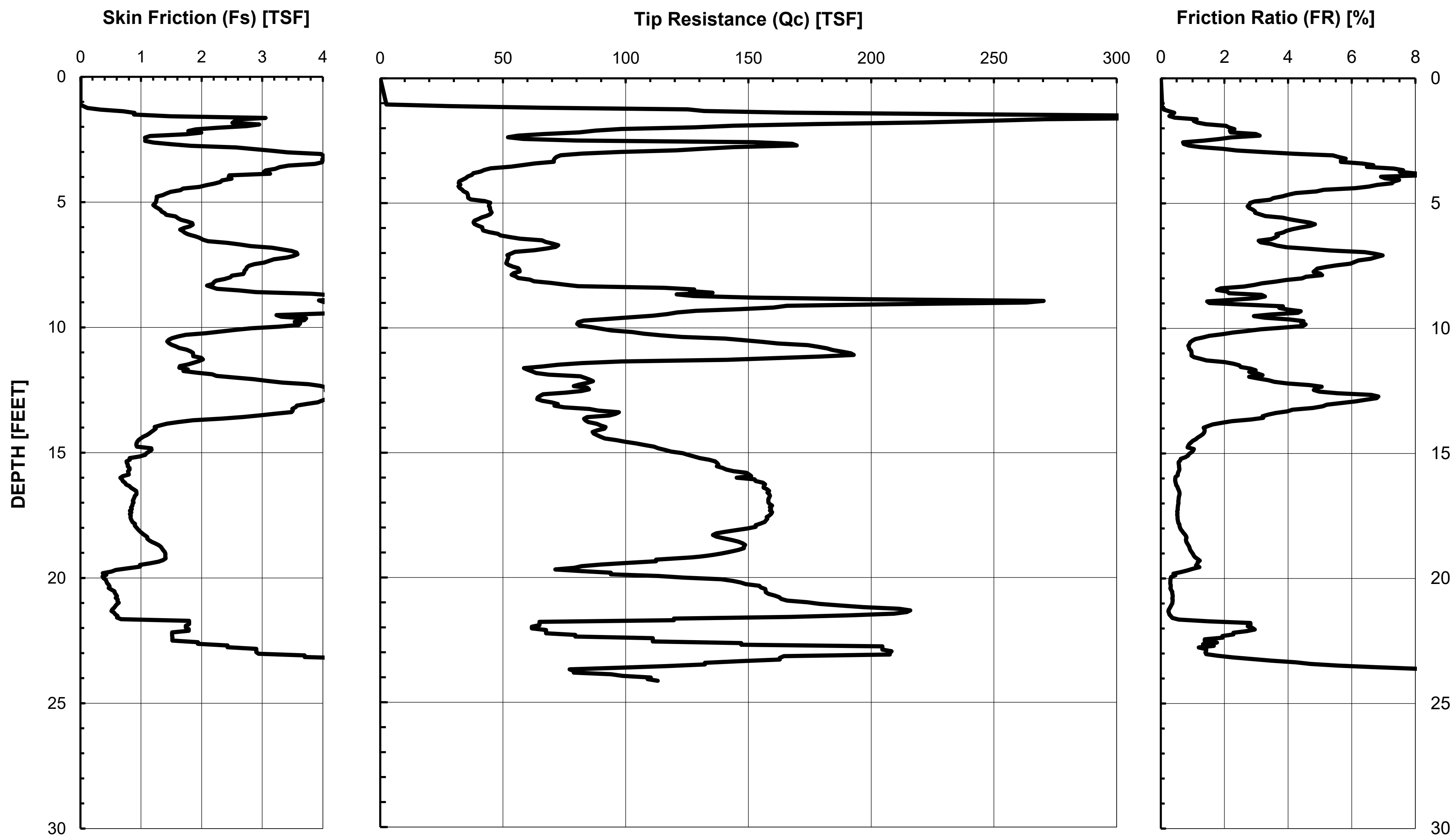
CPT-2

Project: USD Group Biofuels Terminal
Location: 837 19th Street, National City, CA

Total depth: 24.15 ft, Date: 3/23/2022
Surface Elevation: 17.00 ft

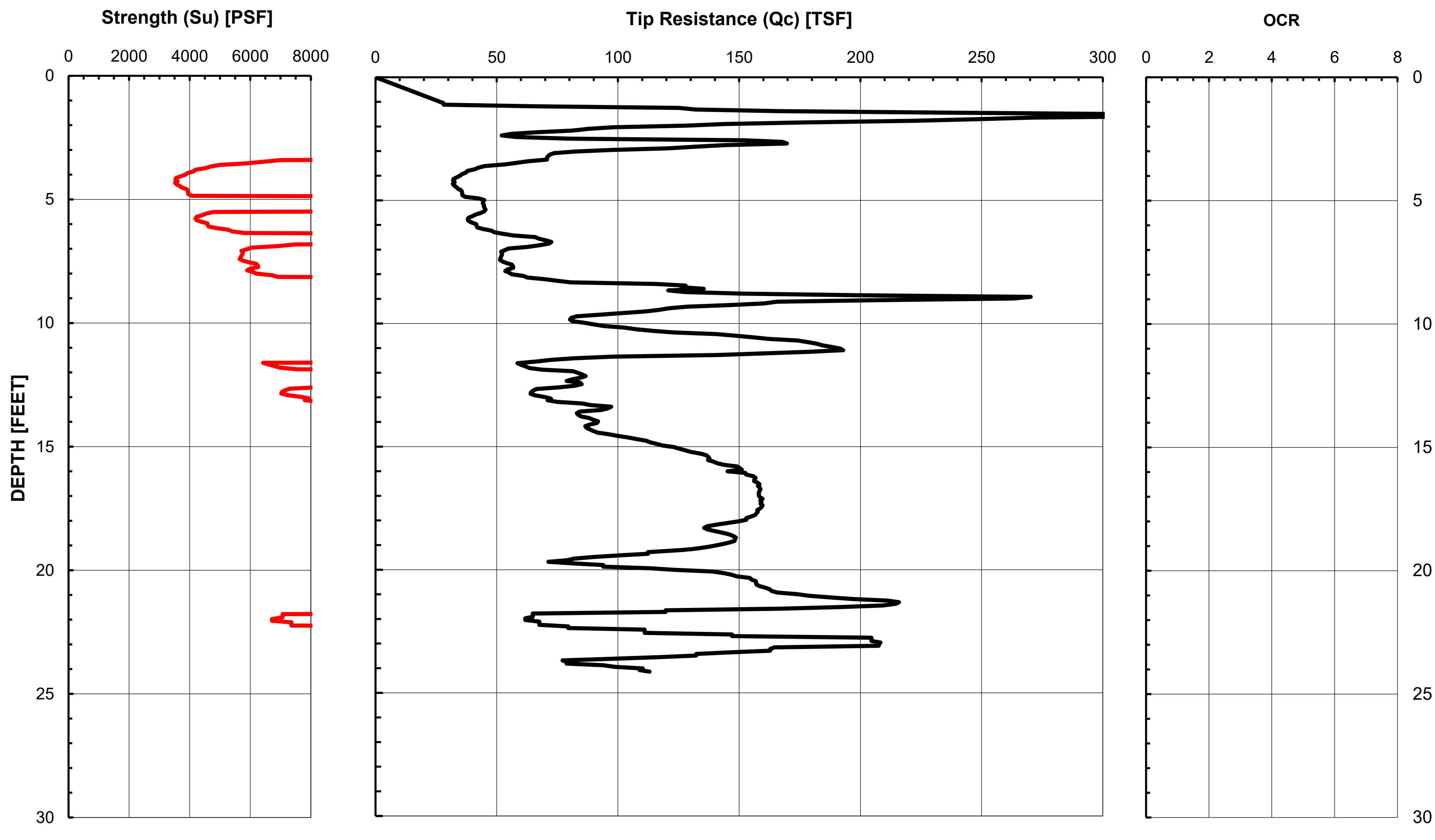




GROUP DELTA

CONE PENETOMETER DATA (CPT-2)

Document No. 22-0036
Project No. SD724
FIGURE A-8a



GROUP DELTA

ESTIMATED STRENGTH AND OCR (CPT-2)

Document No. 22-0036
Project No. SD724
FIGURE A-8b

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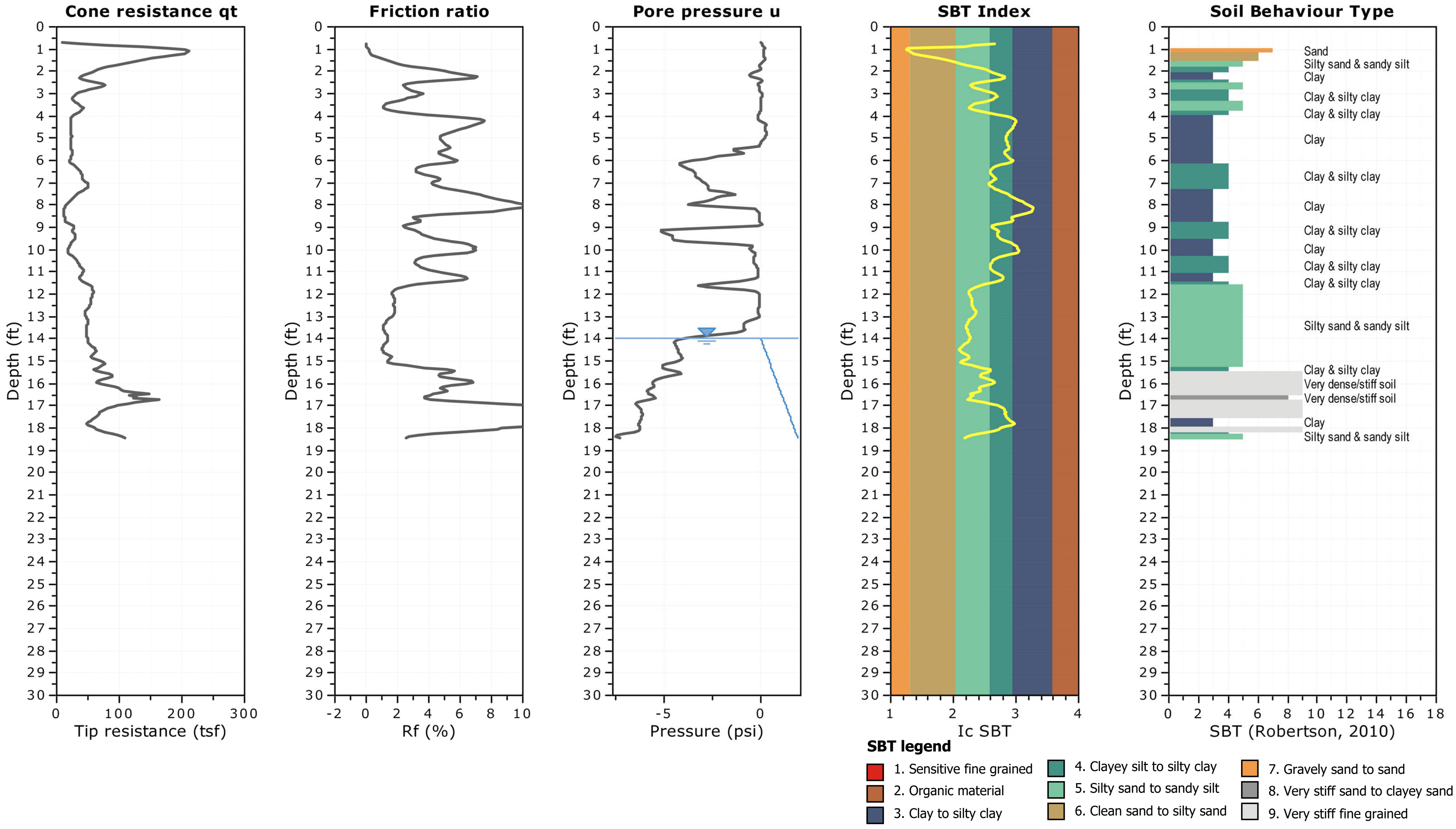
CPT-3

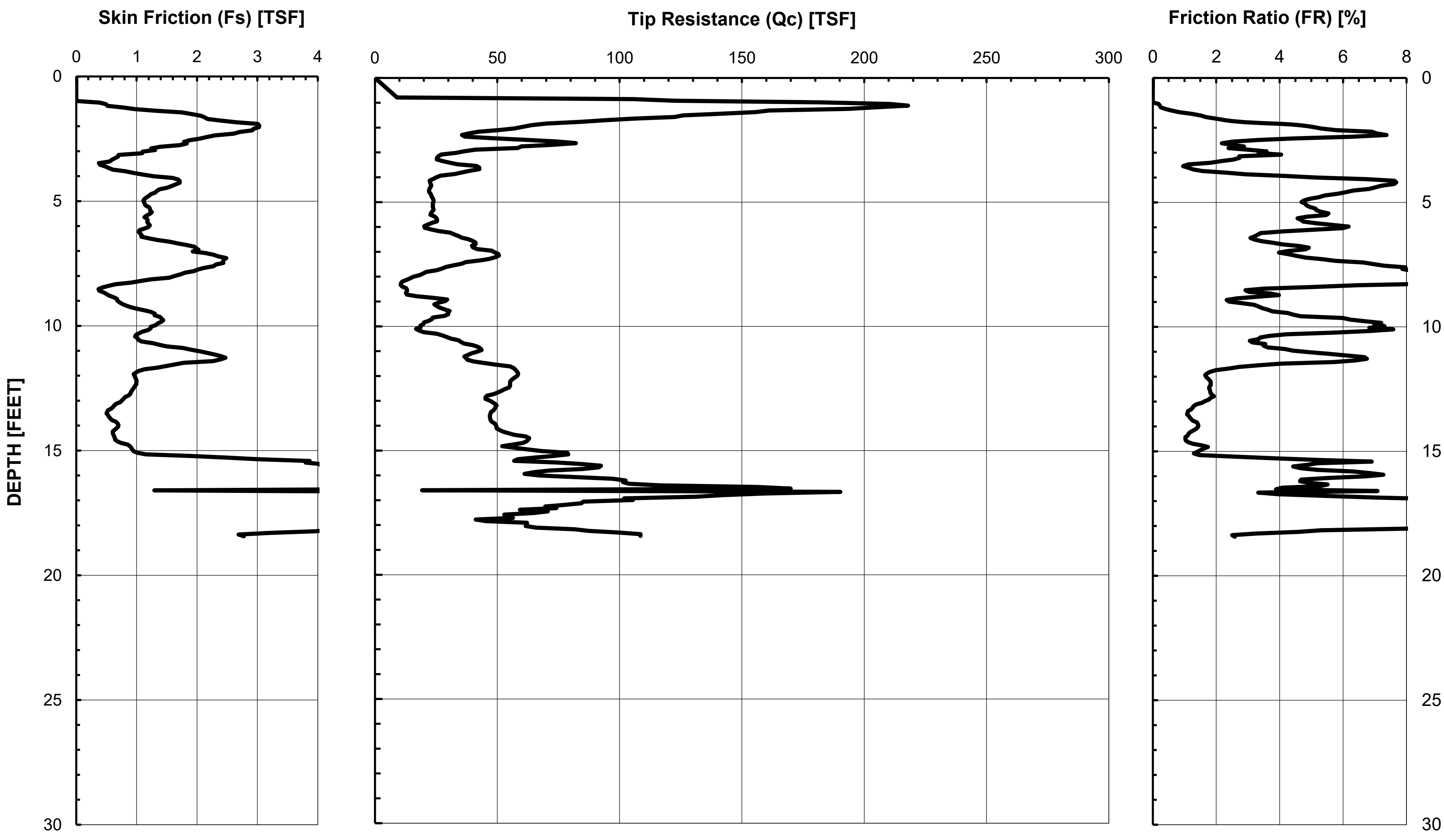
Project: USD Group Biofuels Terminal
Location: 837 19th Street, National City, CA

Total depth: 18.44 ft, Date: 3/23/2022
Surface Elevation: 16.00 ft



CPeT-IT v.3.5.3.3 - CPTU data presentation & interpretation software - Report created on: 4/1/2022, 2:42:54 PM
Project file: N:\Projects\SD\SD700\SD724 ECORP USD Clean Fuels National City Rail Terminal\9. Reports\22-0036\Appendix A\CPeT.cpt

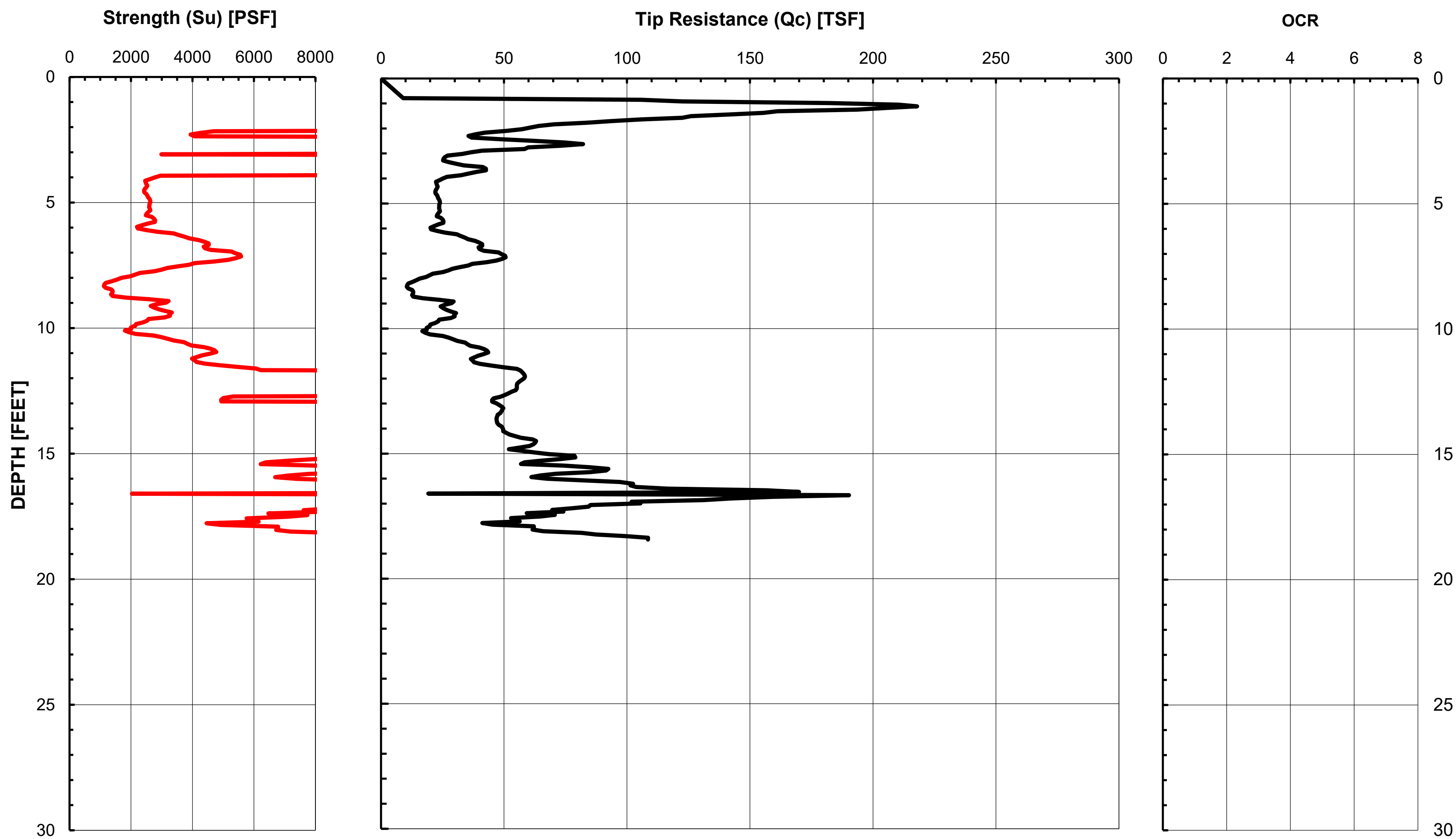
FIGURE A-9



GROUP DELTA

CONE PENETOMETER DATA (CPT-3)

Document No. 22-0036
Project No. SD724
FIGURE A-9a



GROUP DELTA

ESTIMATED STRENGTH AND OCR (CPT-3)

Document No. 22-0036
Project No. SD724
FIGURE A-9b

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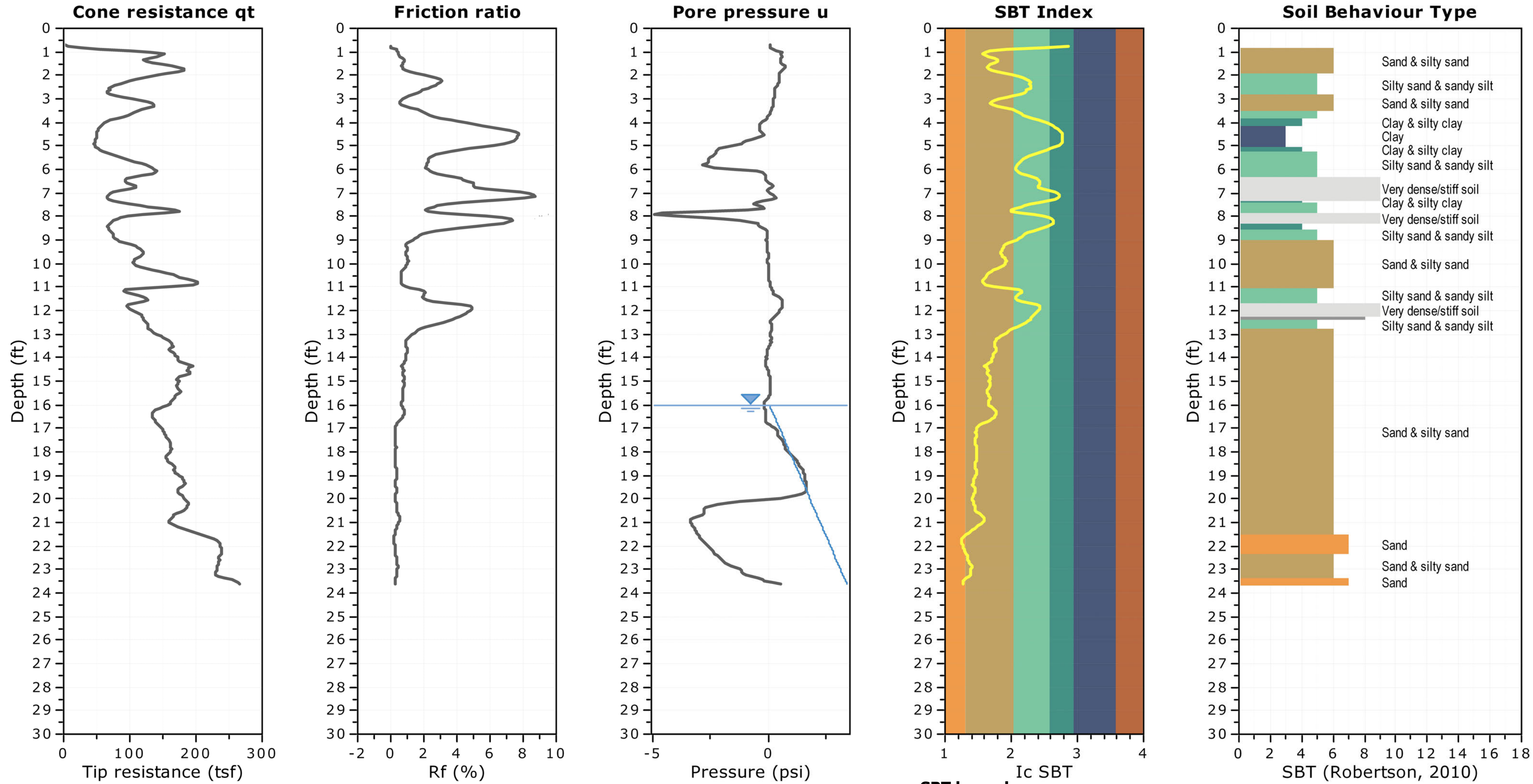
CPT-4

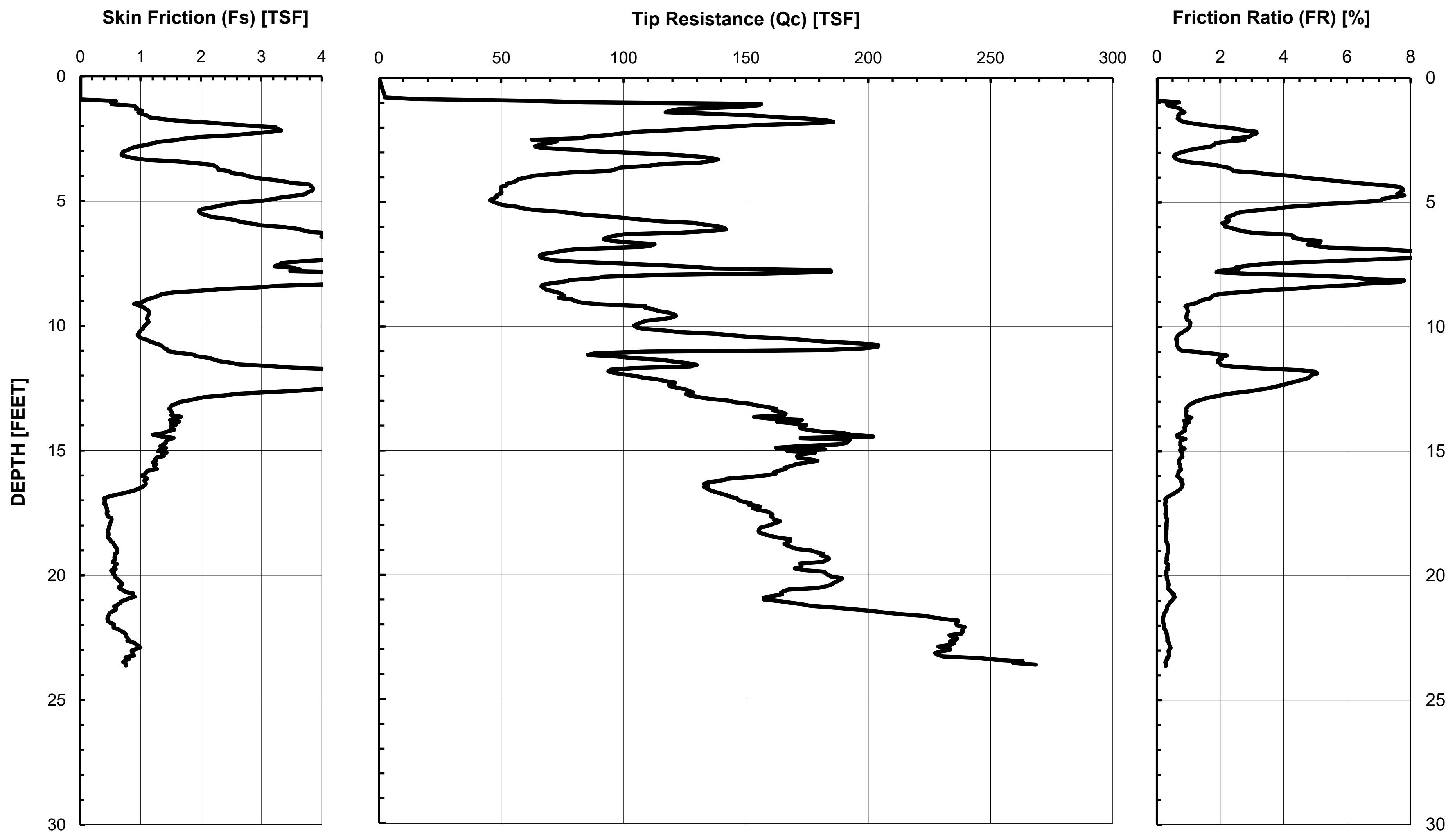
Project: **USD Group Biofuels Terminal**
Location: **837 19th Street, National City, CA**

Total depth: 23.62 ft, Date: 3/23/2022
Surface Elevation: 18.00 ft



SBT legend

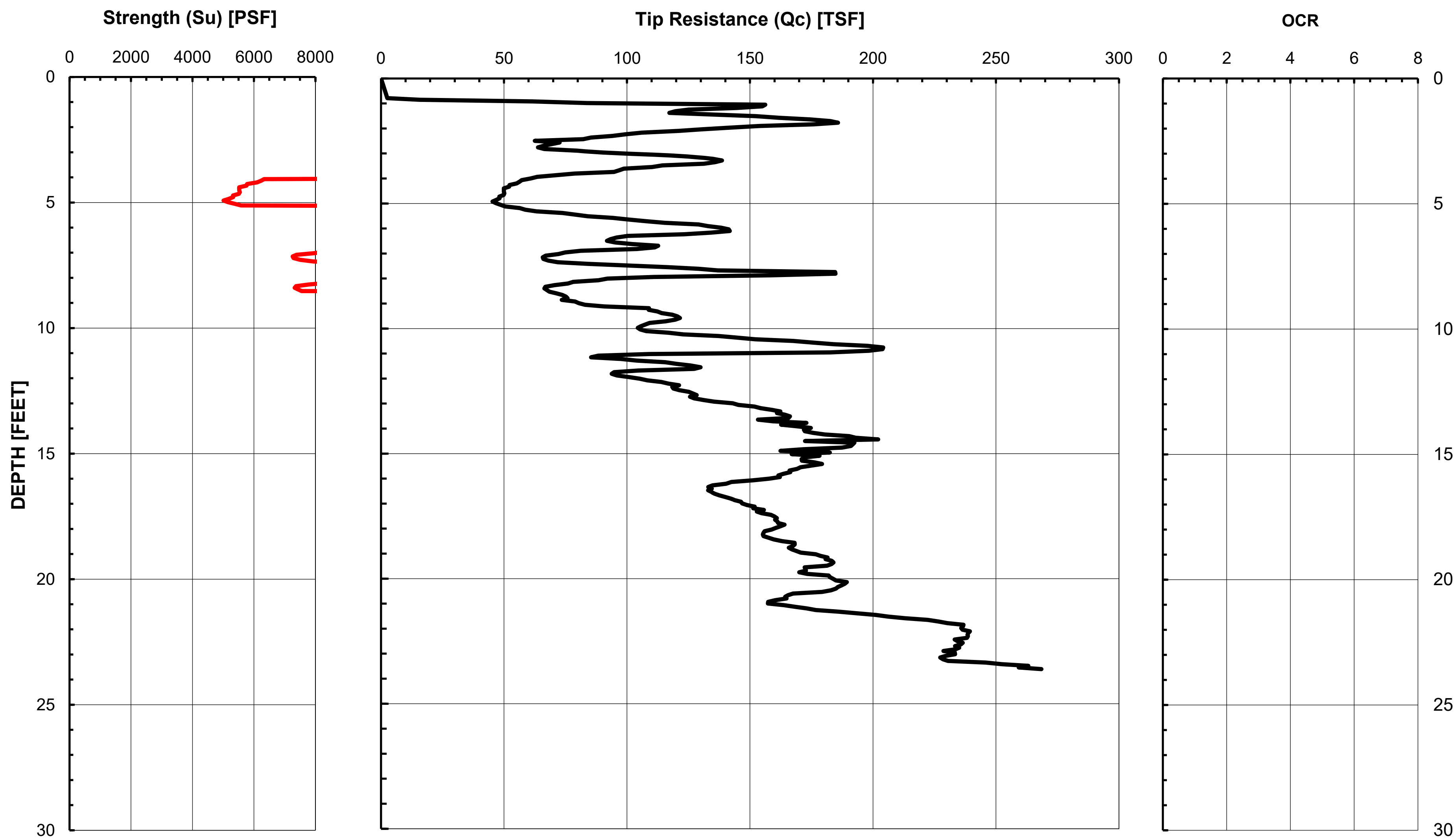
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained



GROUP DELTA

CONE PENETOMETER DATA (CPT-4)

Document No. 22-0036
Project No. SD724
FIGURE A-10a



GROUP DELTA

ESTIMATED STRENGTH AND OCR (CPT-4)

Document No. 22-0036
Project No. SD724
FIGURE A-10b

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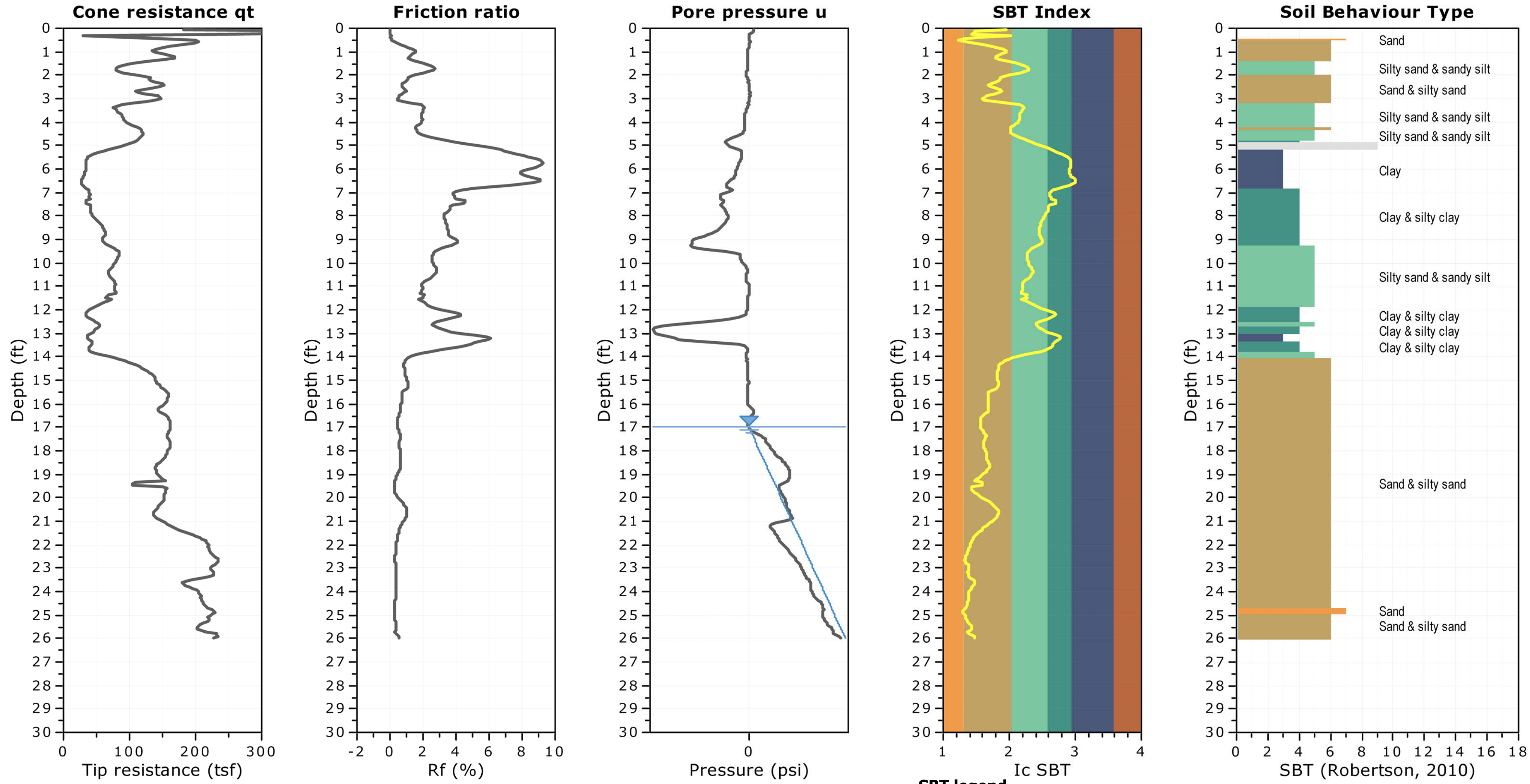
CPT-5

Project: USD Group Biofuels Terminal

Location: 837 19th Street, National City, CA

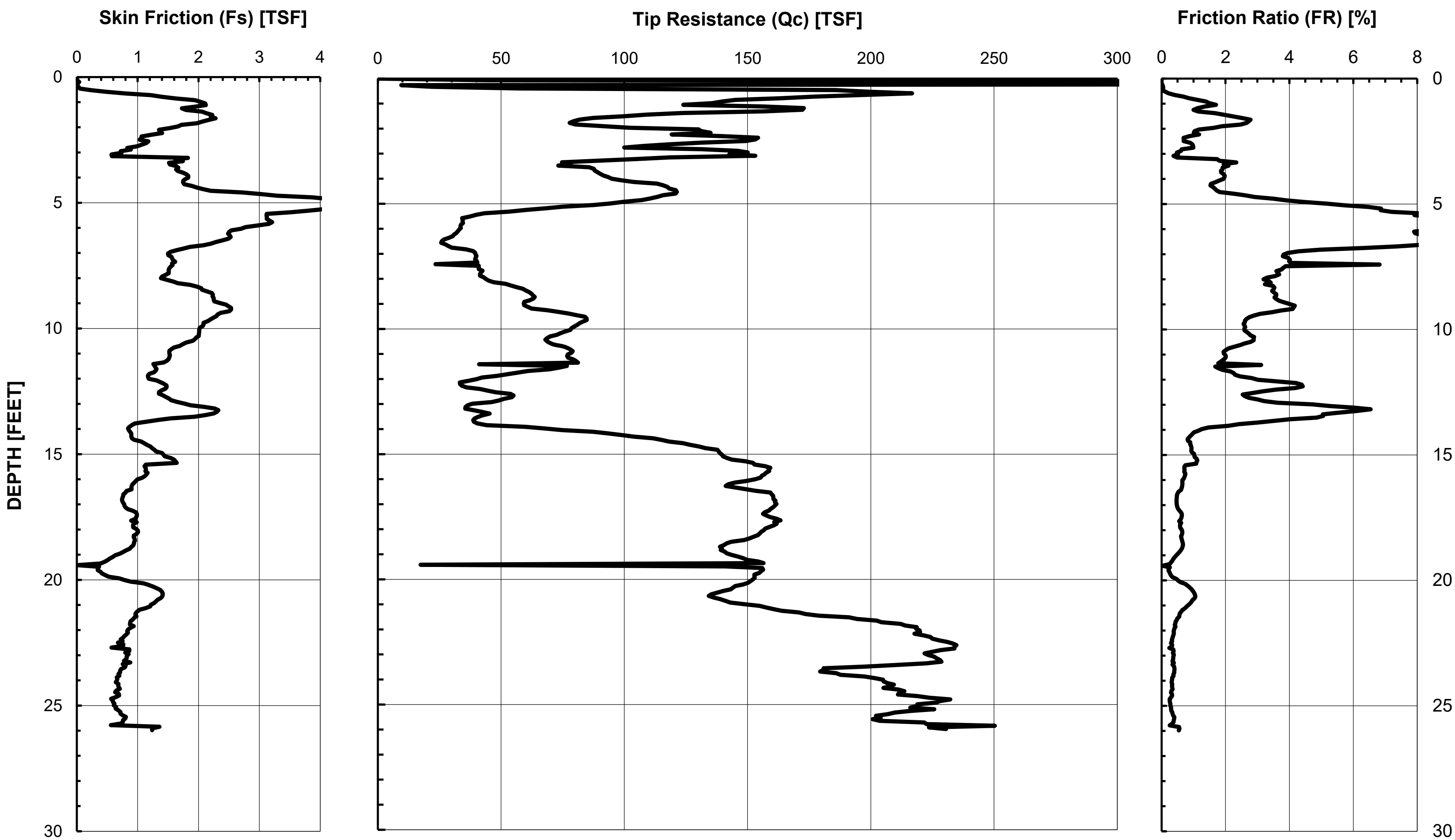
Total depth: 25.98 ft, Date: 3/23/2022

Surface Elevation: 17.00 ft



SBT legend

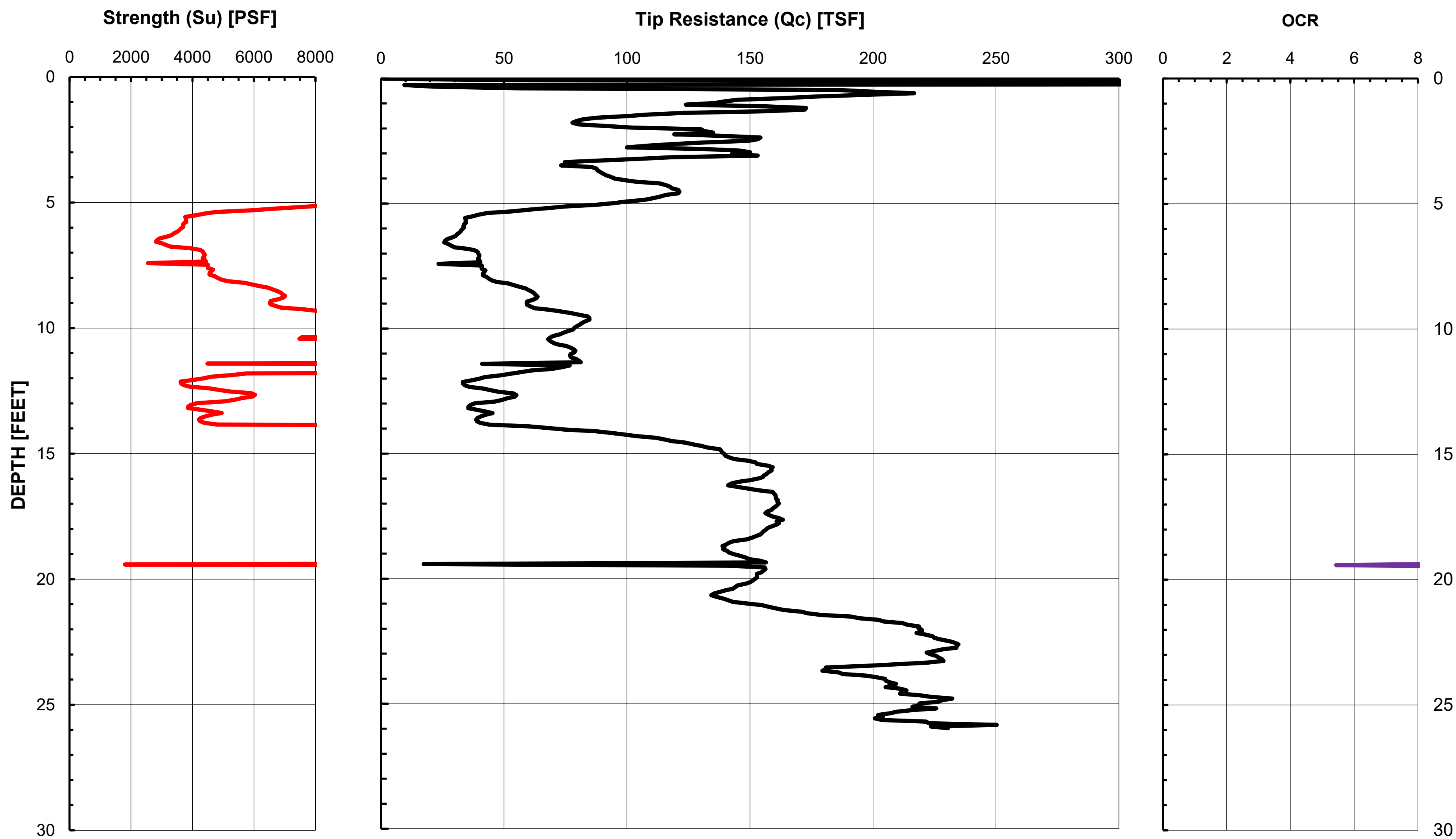
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained



GROUP DELTA

CONE PENETOMETER DATA (CPT-5)

Document No. 22-0036
Project No. SD724
FIGURE A-11a



GROUP DELTA

ESTIMATED STRENGTH AND OCR (CPT-5)

Document No. 22-0036
Project No. SD724
FIGURE A-11b

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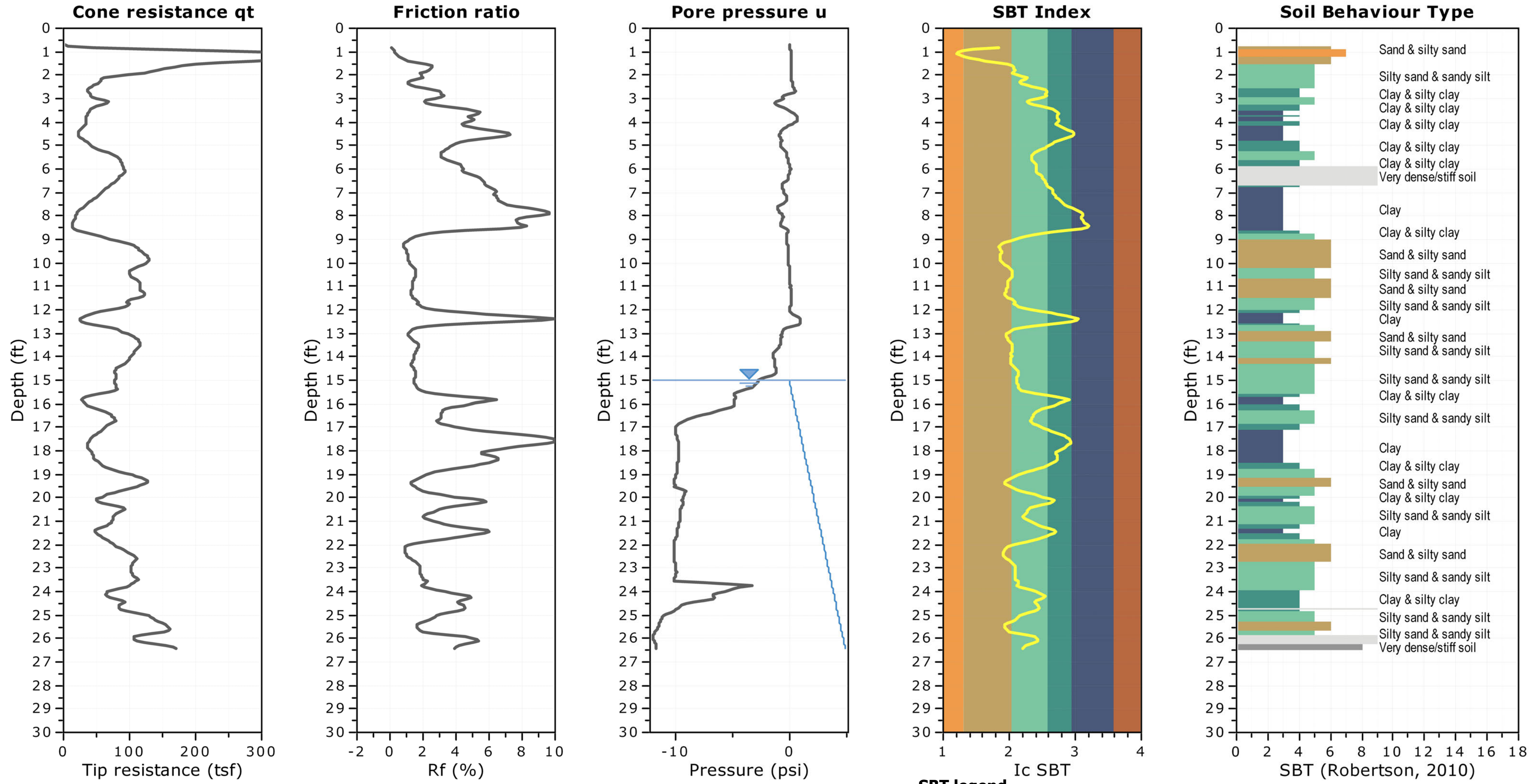
CPT-6

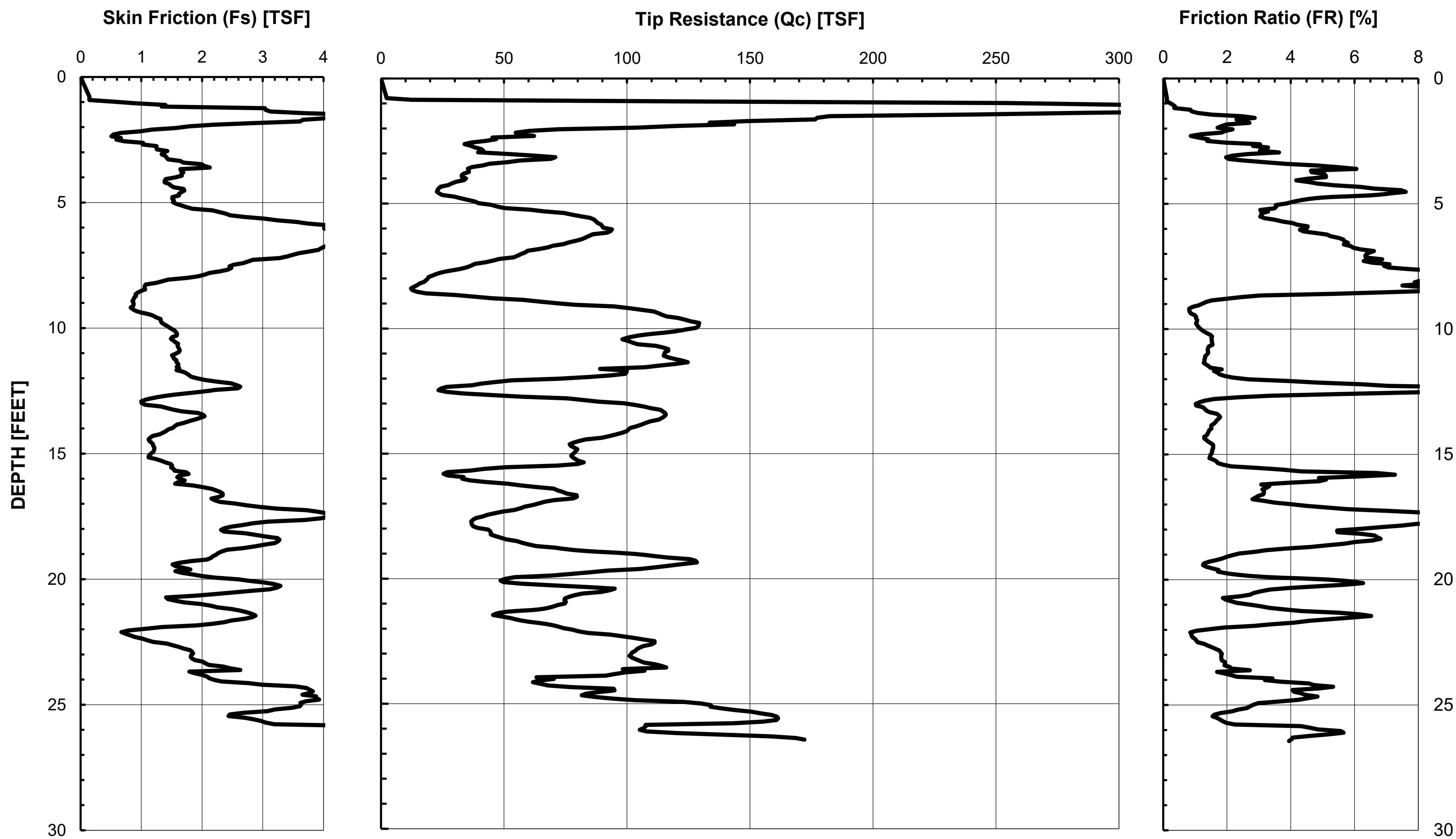
Project: USD Group Biofuels Terminal
Location: 837 19th Street, National City, CA

Total depth: 26.44 ft, Date: 3/23/2022
Surface Elevation: 15.00 ft



SBT legend

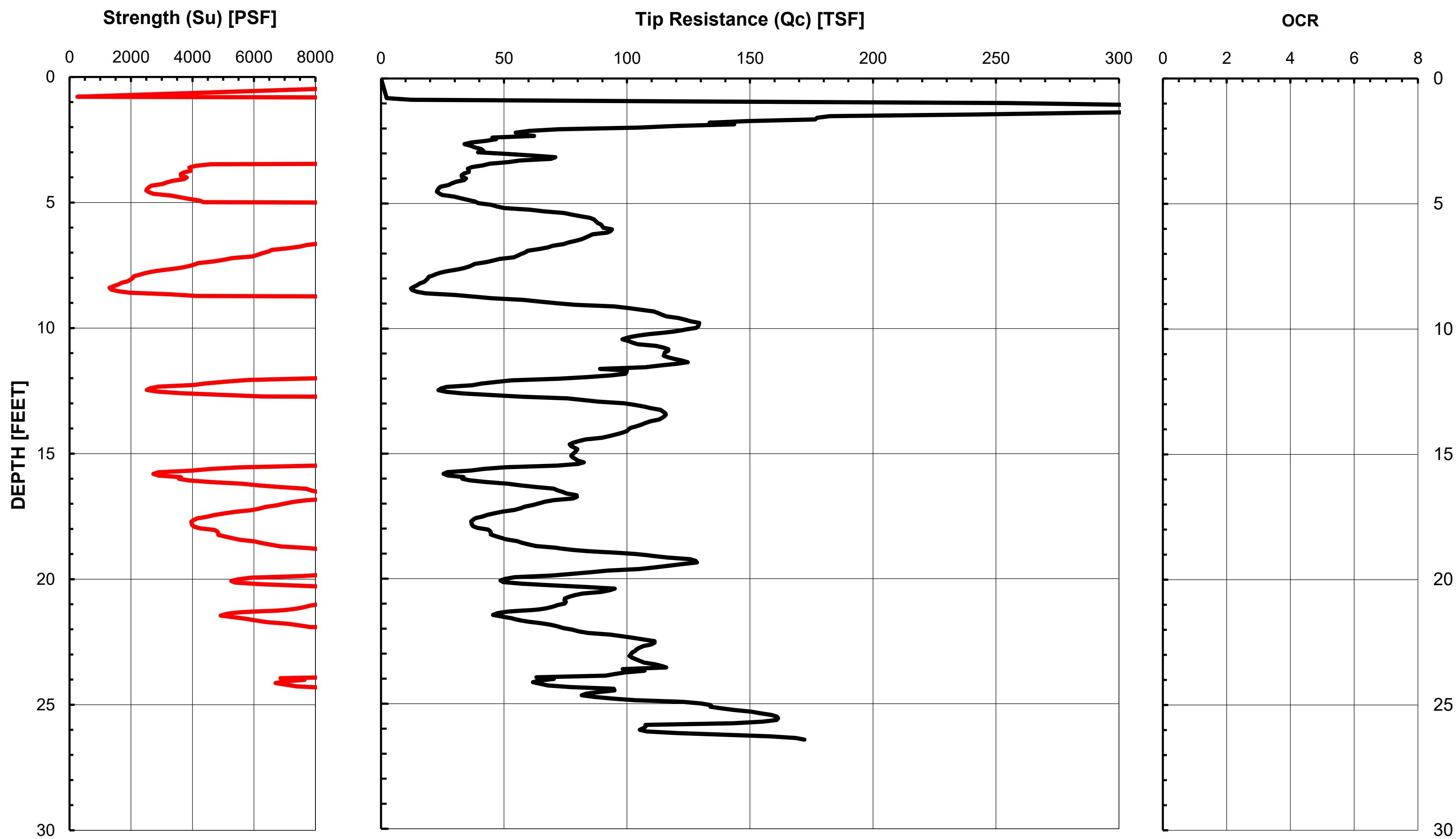
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained



GROUP DELTA

CONE PENETOMETER DATA (CPT-6)

Document No. 22-0036
Project No. SD724
FIGURE A-12a



GROUP DELTA

ESTIMATED STRENGTH AND OCR (CPT-6)

Document No. 22-0036
Project No. SD724
FIGURE A-12b

APPENDIX B
LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Laboratory testing was conducted in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions and in the same locality. No warranty, express or implied, is made as to the correctness or serviceability of the test results, or the conclusions derived from these tests. Where a specific laboratory test method has been referenced, such as ASTM or Caltrans, the reference only applies to the specified laboratory test method, which has been used only as a guidance document for the general performance of the test and not as a "Test Standard". A brief description of the various tests performed for this project follows.

Classification: Soils were visually classified according to the Unified Soil Classification System as established by the American Society of Civil Engineers per ASTM D2487. The soil classifications are shown on the boring logs in Appendix A.

Particle Size Analysis: Particle size analyses were performed in general accordance with ASTM D422, and were used to supplement visual soil classifications. The test results are summarized in Figures B-1.1 through B-1.6.

Atterberg Limits: ASTM D4318 was used to determine the liquid and plastic limits, and plasticity index of a selected sample. The test results are shown in Figures B-1.1 and B-1.4.

Expansion Index: The expansion potentials of selected soil samples were estimated in general accordance with the laboratory procedures outlined in ASTM test method D4829. The test results are summarized in Figure B-2. Figure B-2 also presents common criteria for evaluating the expansion potential based on the expansion index.

pH and Resistivity: To assess the potential for reactivity with buried metals, selected soil samples were tested for pH and minimum resistivity using Caltrans test method 643. The corrosivity test results are summarized in Figure B-3.

Sulfate Content: To assess the potential for reactivity with concrete, selected soil samples were tested for water soluble sulfate. The sulfate was extracted from the soil under vacuum using a 10:1 (water to dry soil) dilution ratio. The extracted solution was tested for water soluble sulfate in general accordance with ASTM D516. The test results are also presented in Figure B-3, along with common criteria for evaluating soluble sulfate content.

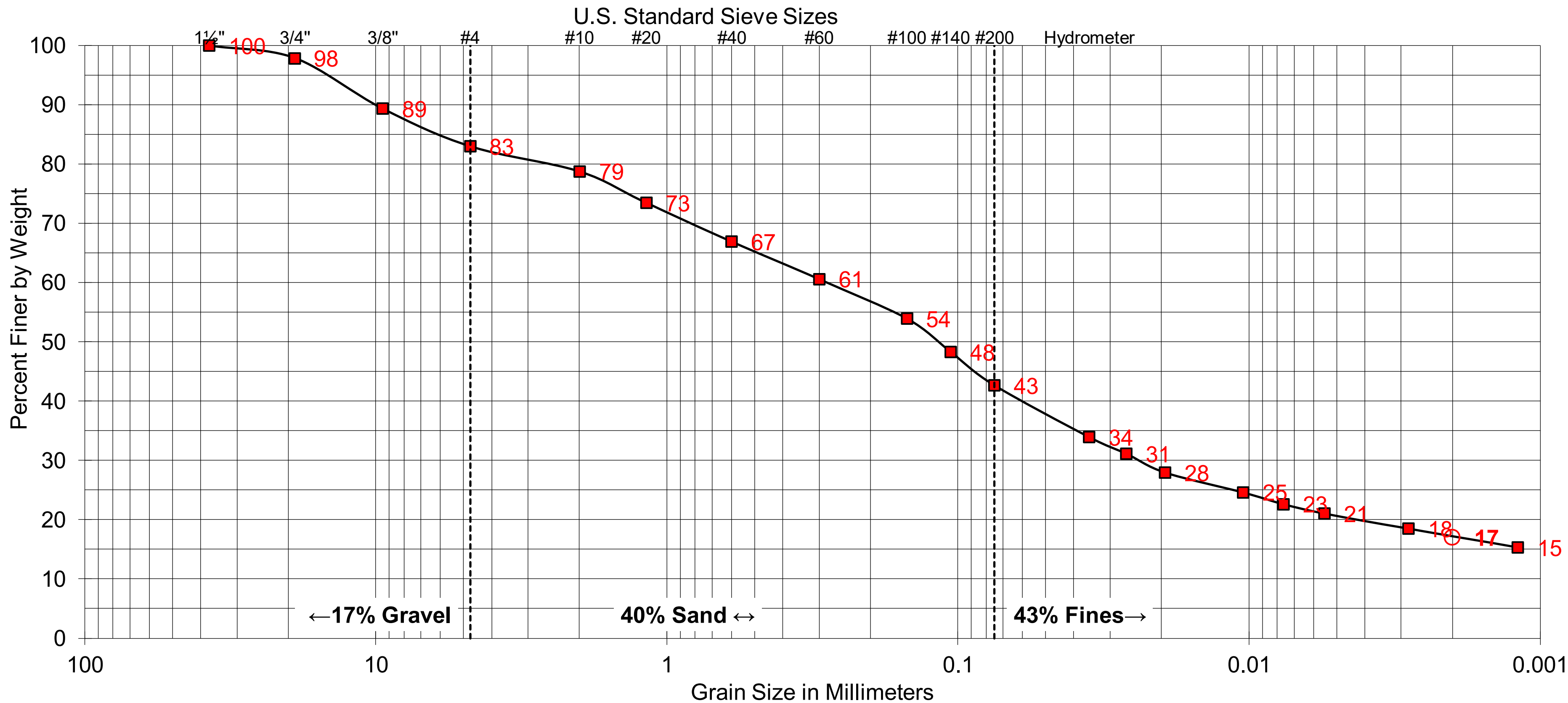
Chloride Content: Soil samples were also tested for water soluble chloride. The chloride was extracted from the soil under vacuum using a 10:1 (water to dry soil) dilution ratio as described above. The extracted solutions were then tested for water soluble chloride using a calibrated ion specific electronic probe. The test results are also shown in Figure B-3.

APPENDIX B

LABORATORY TESTING (Continued)

Direct Shear: The shear strengths of selected samples of the on-site soils were assessed using direct shear testing performed in general accordance with ASTM D3080. The direct shear test results are summarized in Figures B-4.1 through B-4.3.

R-Value: R-Value tests were performed on selected samples of the on-site soils in general accordance with CTM 301. The test results are shown in Figures B-5.1 and B-5.2.



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	B-1
SAMPLE DEPTH:	½' - 5'

UNIFIED SOIL CLASSIFICATION: SC

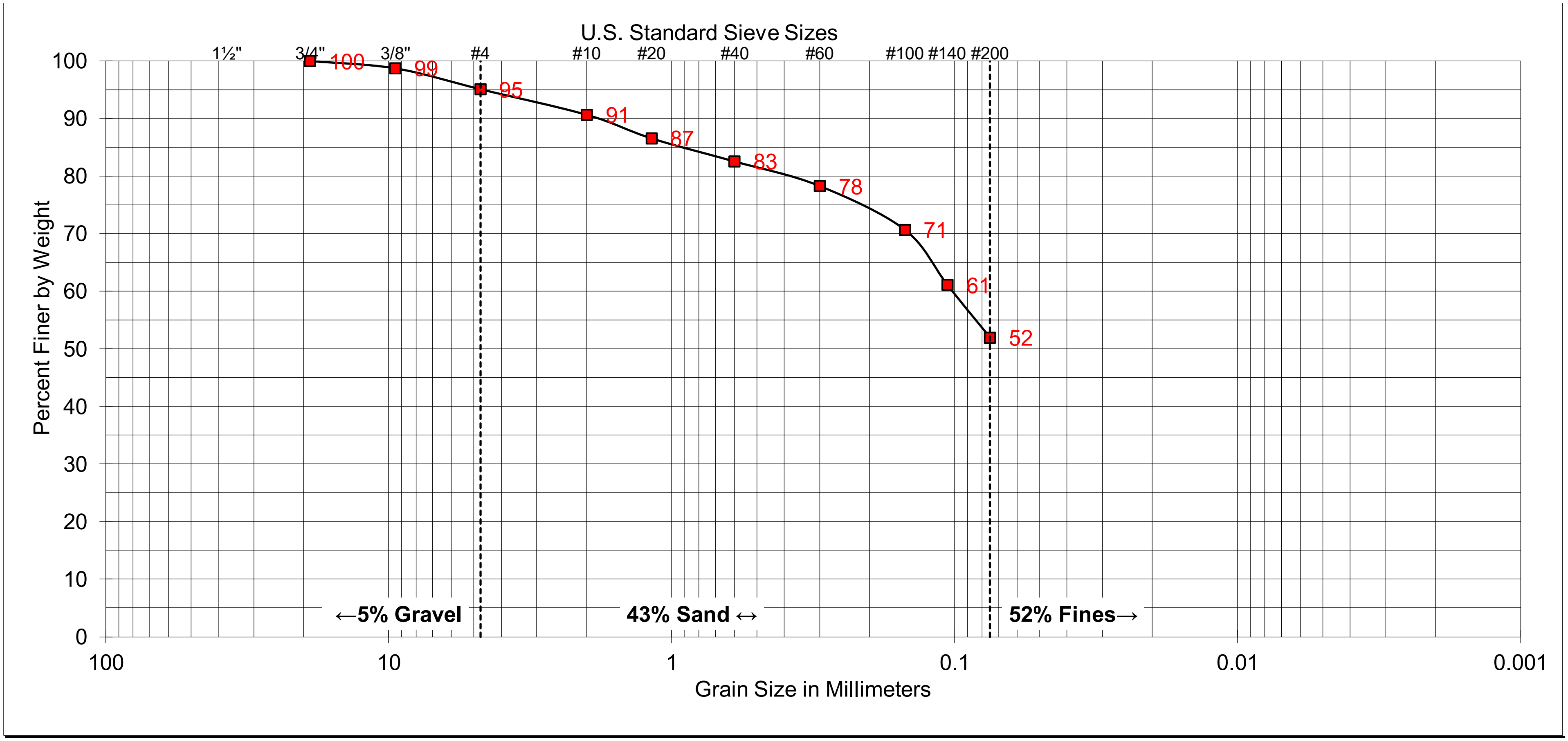
DESCRIPTION: CLAYEY SAND WITH GRAVEL

ATTERBERG LIMITS	
LIQUID LIMIT:	22
PLASTIC LIMIT:	11
PLASTICITY INDEX:	11

GROUP DELTA

SOIL CLASSIFICATION

Document No. 22-0036
Project No. SD724
FIGURE B-1.1



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	B-2
SAMPLE DEPTH:	0' - 5'

UNIFIED SOIL CLASSIFICATION: CL

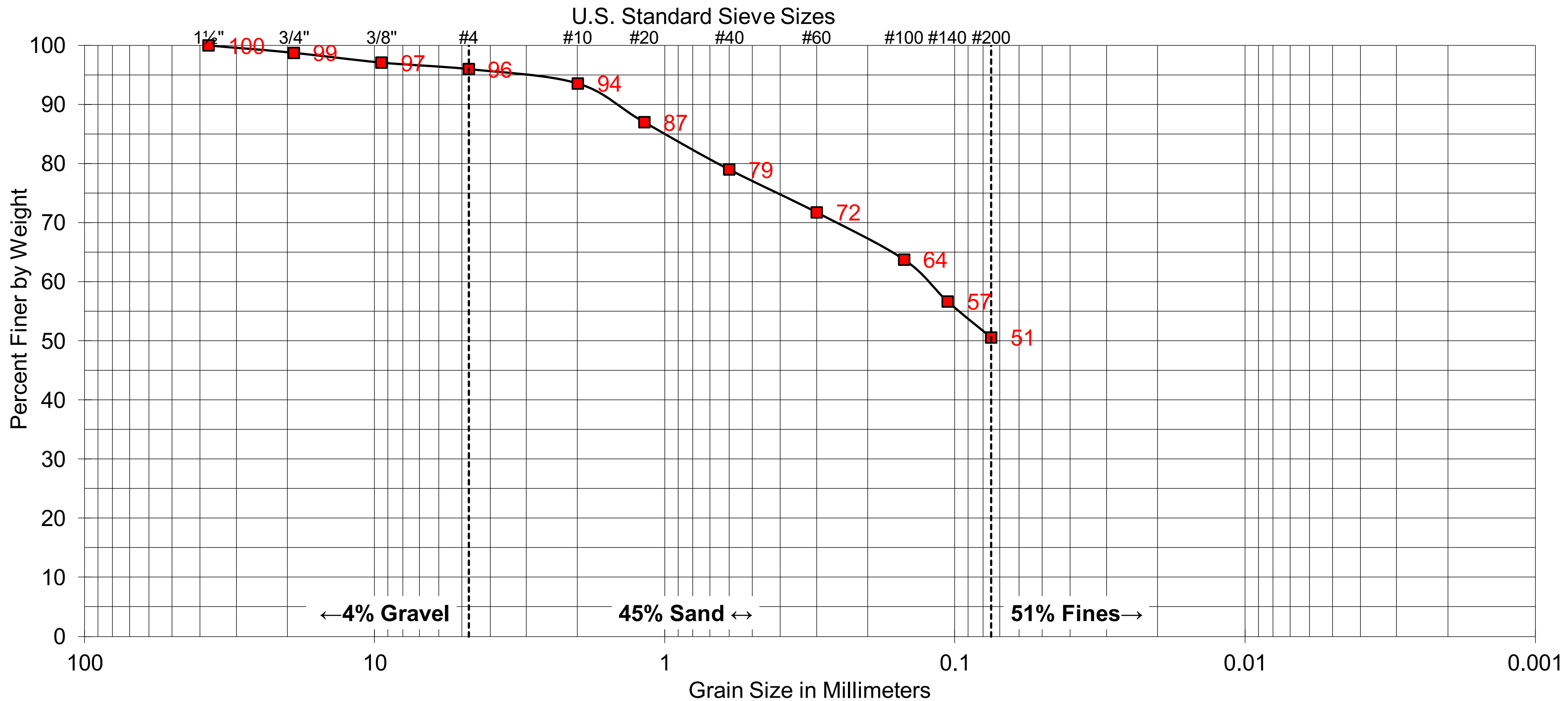
DESCRIPTION: SANDY LEAN CLAY

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

GROUP DELTA

SOIL CLASSIFICATION

Document No. 22-0036
Project No. SD724
FIGURE B-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	B-3
SAMPLE DEPTH:	1' - 5'

UNIFIED SOIL CLASSIFICATION: CL

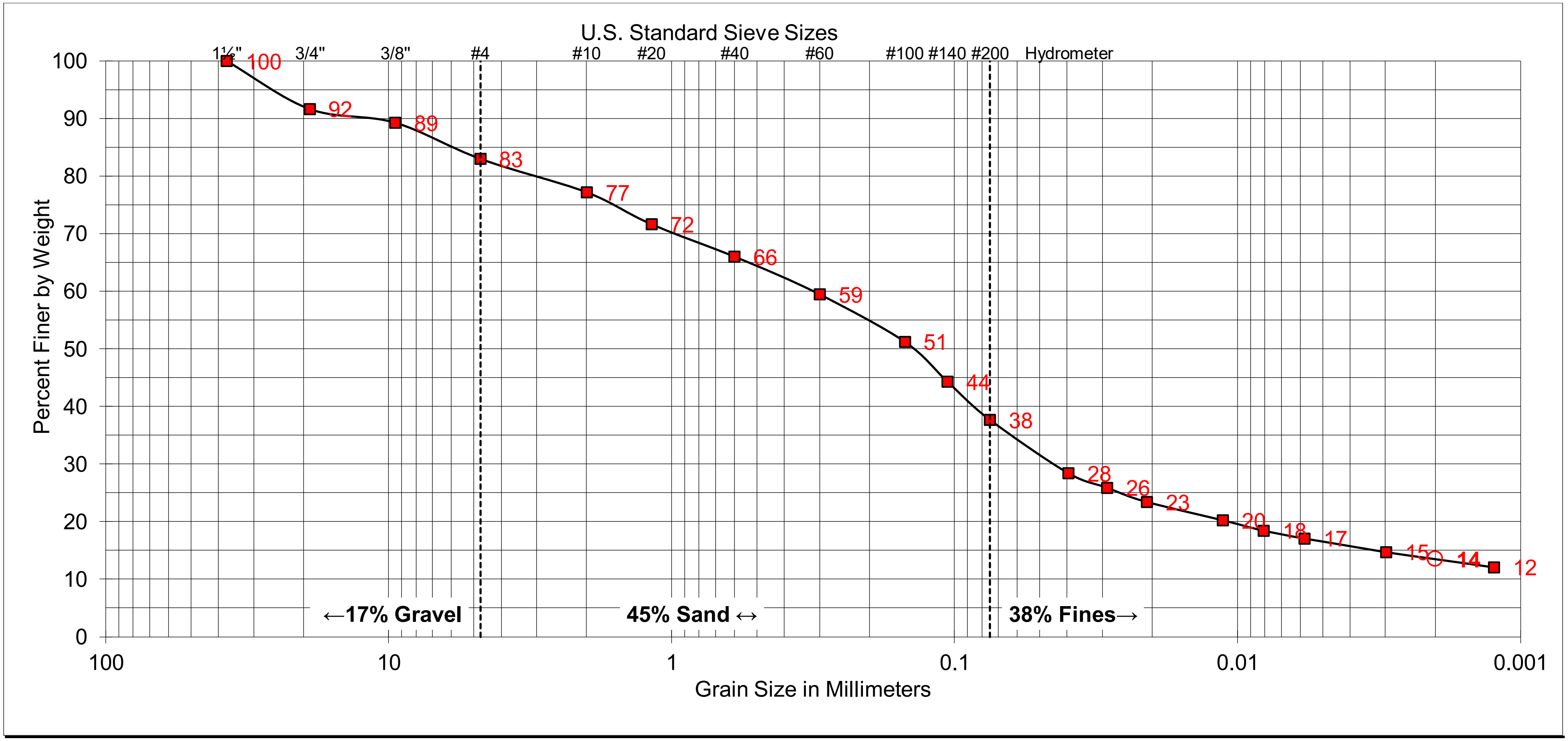
DESCRIPTION: SANDY LEAN CLAY

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

GROUP DELTA

SOIL CLASSIFICATION

Document No. 22-0036
Project No. SD724
FIGURE B-1.3



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	B-4
SAMPLE DEPTH:	0' - 5'

UNIFIED SOIL CLASSIFICATION: SC

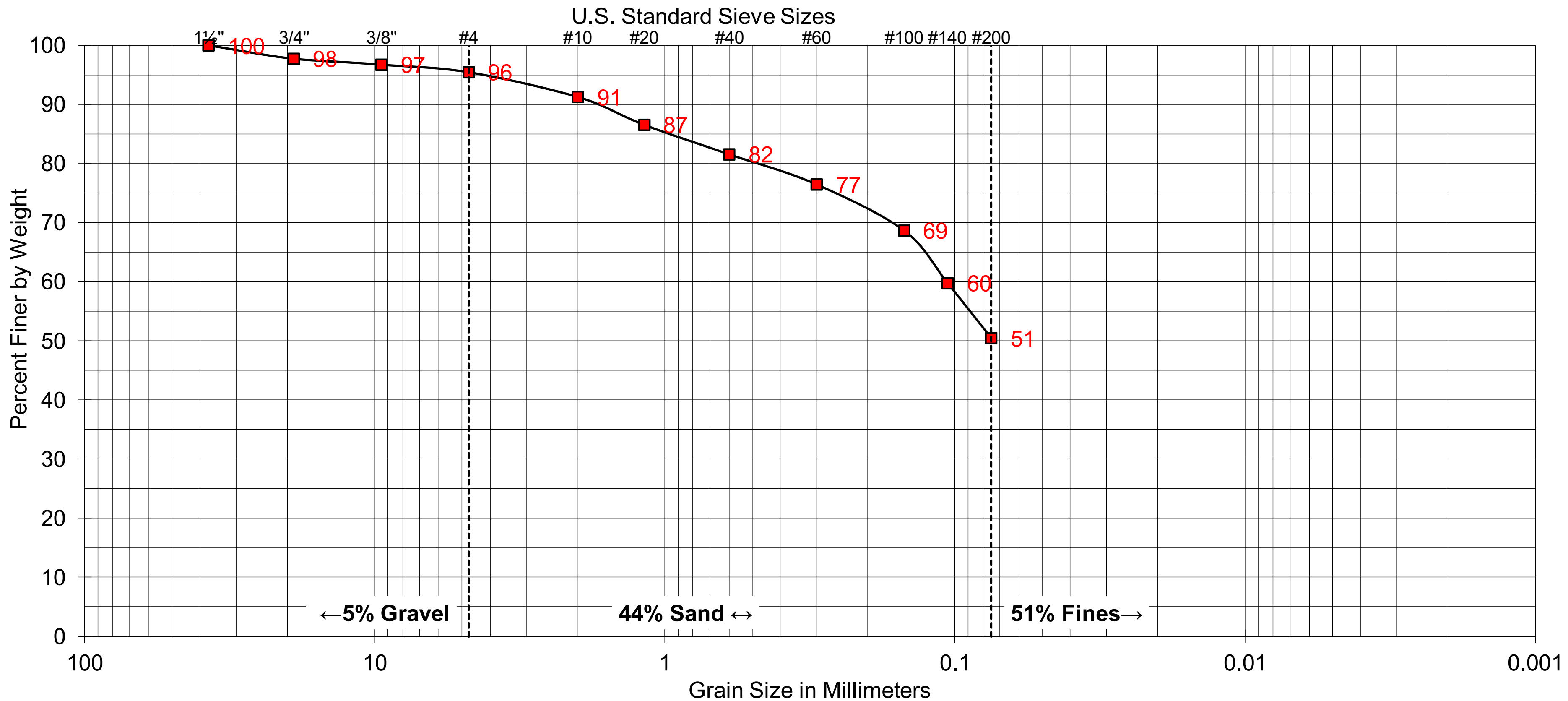
DESCRIPTION: CLAYEY SAND WITH GRAVEL

ATTERBERG LIMITS	
LIQUID LIMIT:	21
PLASTIC LIMIT:	13
PLASTICITY INDEX:	8

GROUP DELTA

SOIL CLASSIFICATION

Document No. 22-0036
Project No. SD724
FIGURE B-1.4



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	B-5
SAMPLE DEPTH:	1' - 5'

UNIFIED SOIL CLASSIFICATION: CL

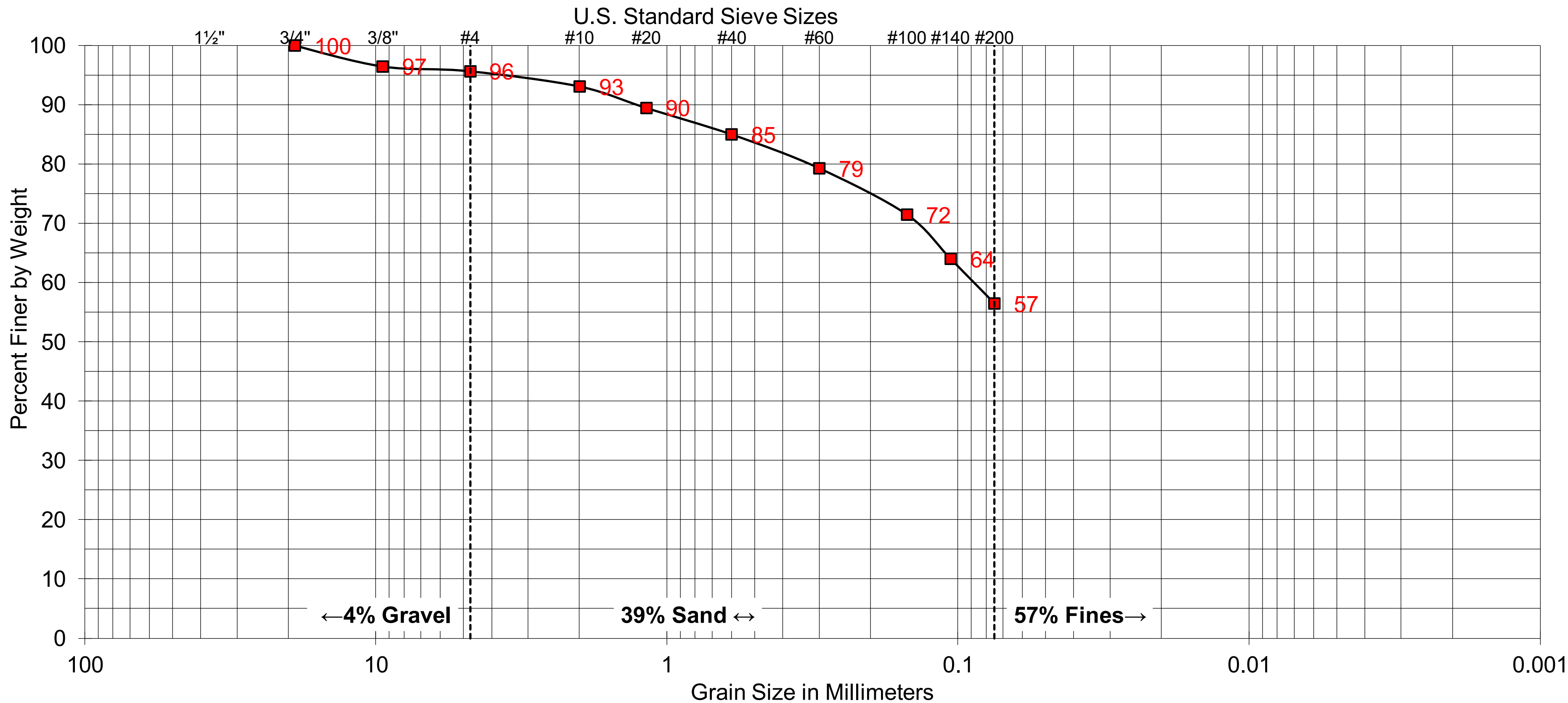
DESCRIPTION: SANDY LEAN CLAY

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

GROUP DELTA

SOIL CLASSIFICATION

Document No. 22-0036
Project No. SD724
FIGURE B-1.5



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	B-6
SAMPLE DEPTH:	0' - 5'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: SANDY LEAN CLAY

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

GROUP DELTA

SOIL CLASSIFICATION

Document No. 22-0036
Project No. SD724
FIGURE B-1.6

EXPANSION TEST RESULTS
(ASTM D4829)

SAMPLE	DESCRIPTION	EXPANSION INDEX
B-1 @ ½' – 5'	**FILL**: Dark brown clayey sand with gravel (SC).	0
B-3 @ 1' – 5'	**FILL**: Reddish brown sandy lean clay (CL).	25
B-4 @ 0' – 5'	**FILL**: Yellow brown clayey sand with gravel (SC).	0

EXPANSION INDEX	POTENTIAL EXPANSION
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very High

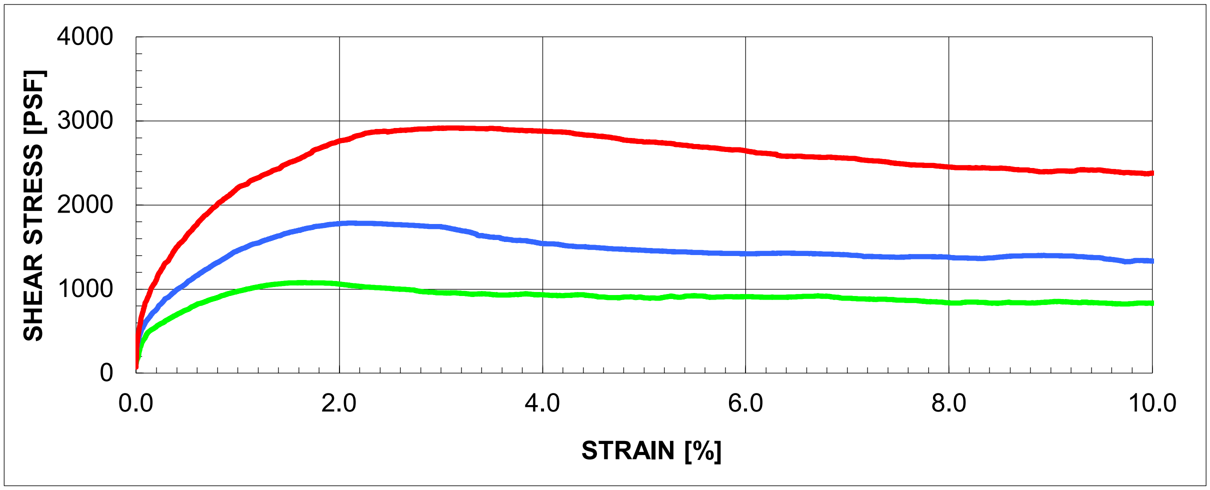
CORROSIVITY TEST RESULTS
(ASTM D516, CTM 643)

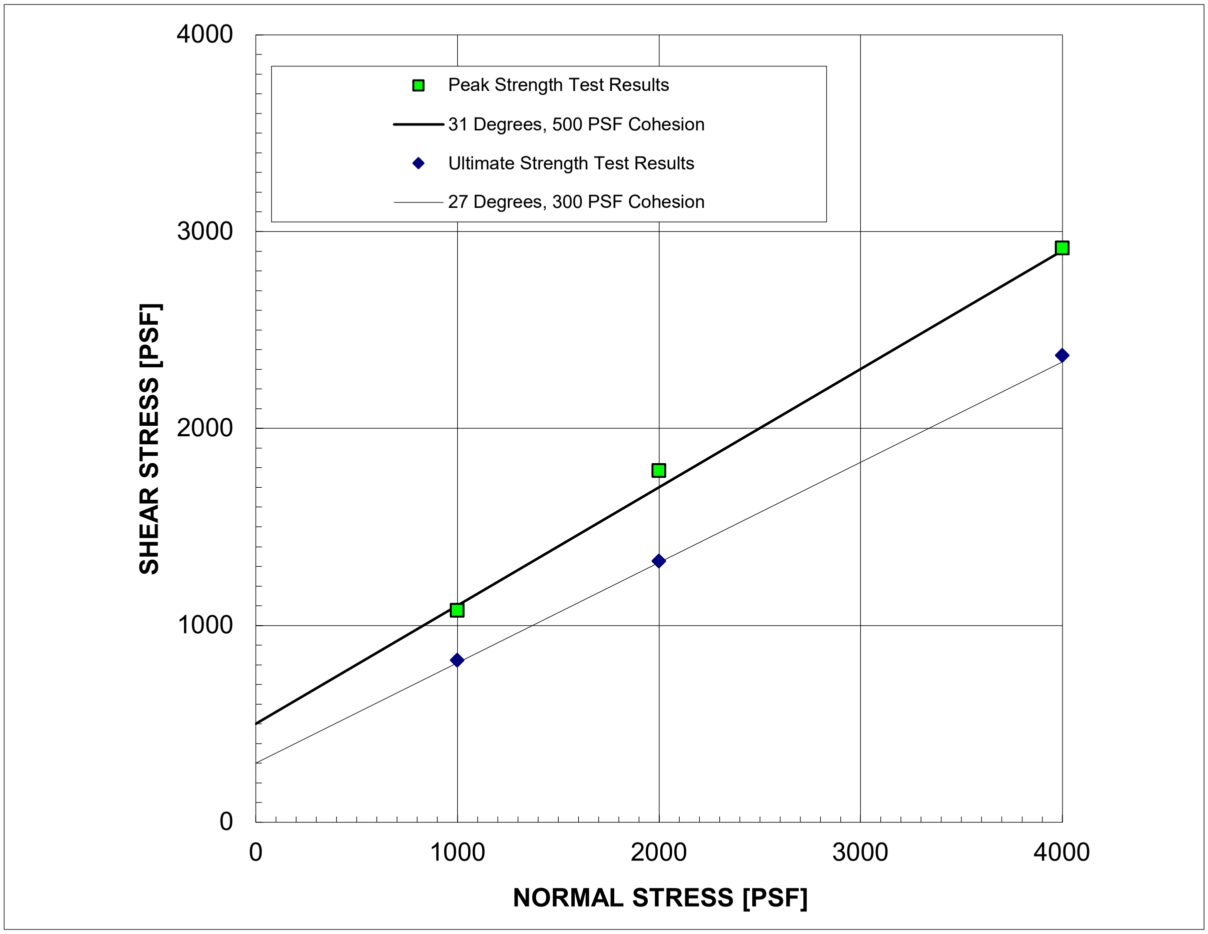
SAMPLE	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
B-1 @ ½' – 5'	7.9	1,390	0.02	0.01
B-6 @ 0' – 5'	8.6	960	0.03	0.01

SULFATE CONTENT [%]	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	-
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

SOIL RESISTIVITY [OHM-CM]	GENERAL DEGREE OF CORROSIVITY TO FERROUS METALS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

CHLORIDE (Cl) CONTENT [%]	GENERAL DEGREE OF CORROSIVITY TO METALS
0.00 to 0.03	Negligible
0.03 to 0.15	Corrosive
Above 0.15	Severely Corrosive





SAMPLE: B-1 @ 5'

UNDOCUMENTED FILL:
Brown lean clay with sand (CL)

	PEAK	ULTIMATE
ϕ'	31 °	27 °
C'	500 PSF	300 PSF

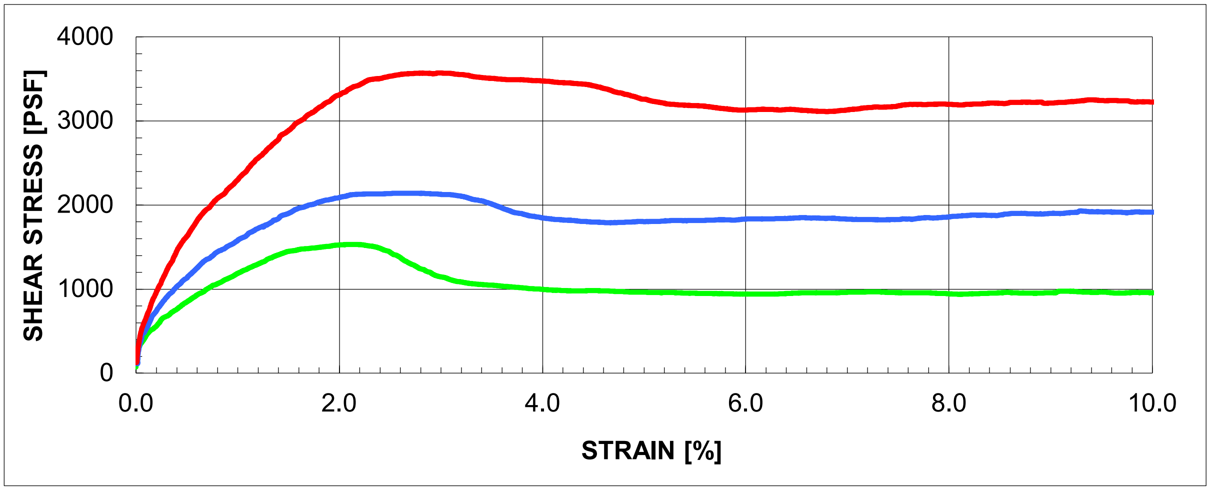
STRAIN RATE: 0.0002 IN/MIN
(Sample was consolidated and drained)

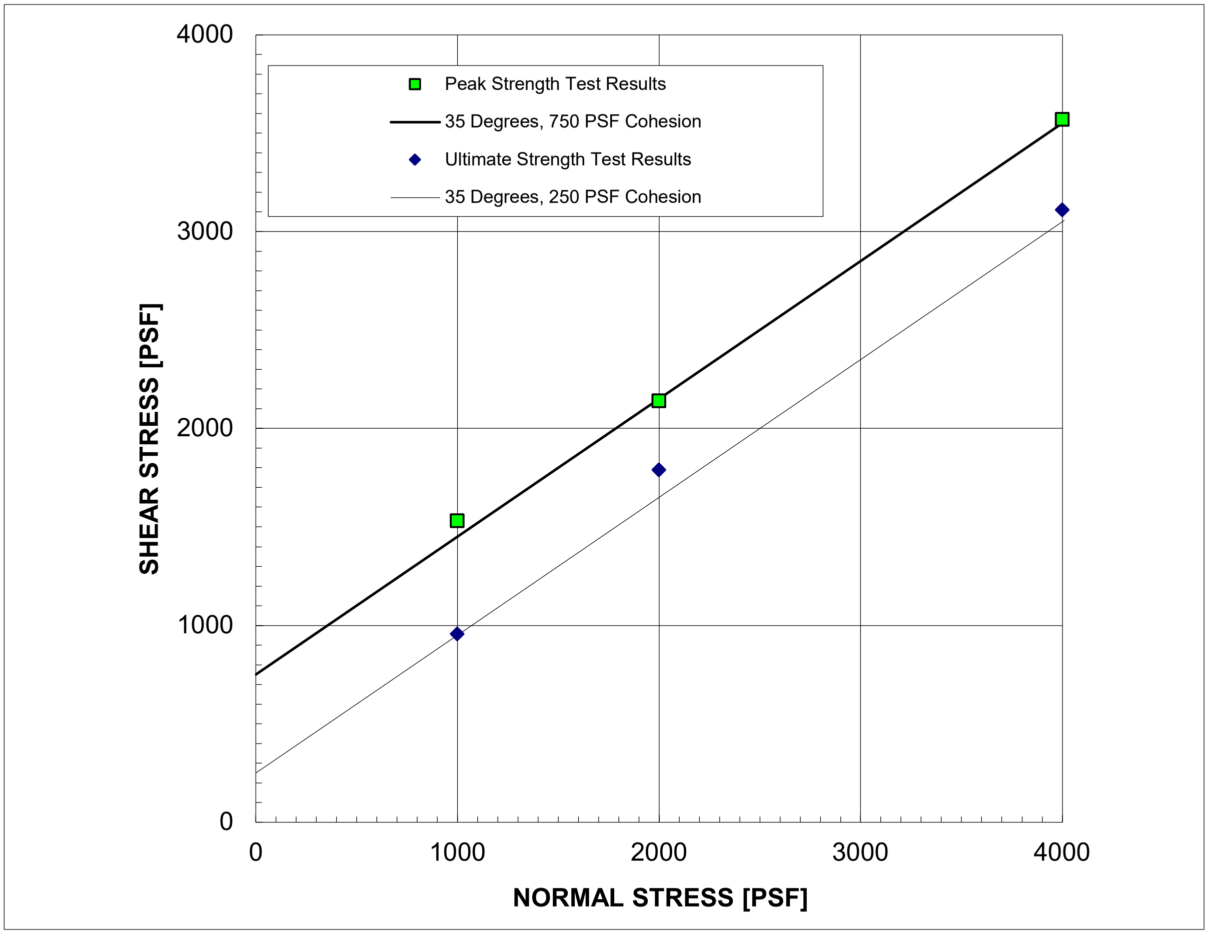
	IN-SITU	AS-TESTED
γ_d	104.9 PCF	104.9 PCF
w_c	20.6 %	23.8 %

GROUP DELTA

DIRECT SHEAR TEST RESULTS

Doucment No. 22-0036
Project No. SD724
FIGURE B-4.1





SAMPLE: B-2 @ 5'

UNDOCUMENTED FILL:
Yellowish brown sandy silt (ML)

	PEAK	ULTIMATE
ϕ'	35 °	35 °
C'	750 PSF	250 PSF

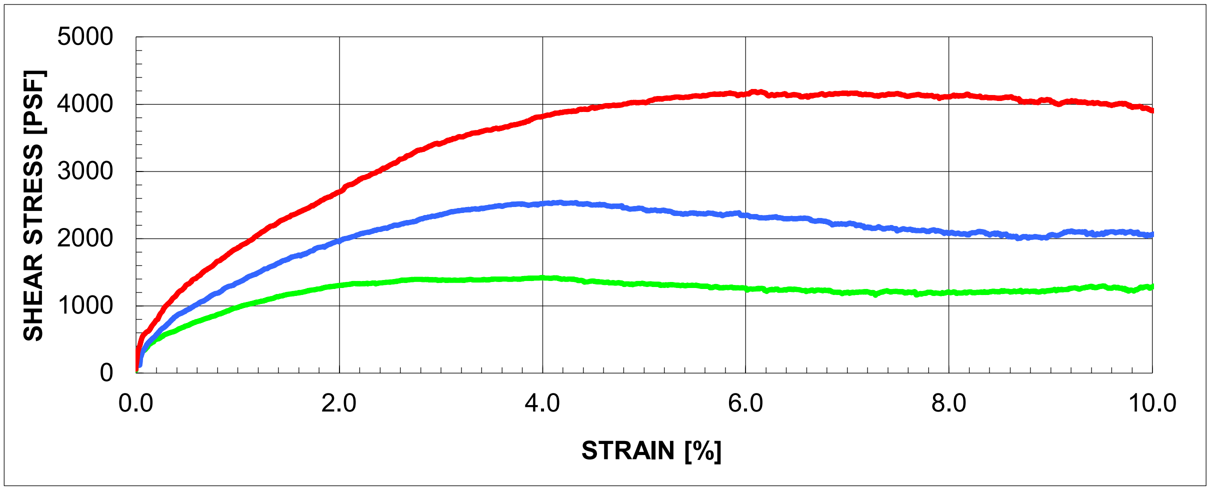
STRAIN RATE: 0.0007 IN/MIN
(Sample was consolidated and drained)

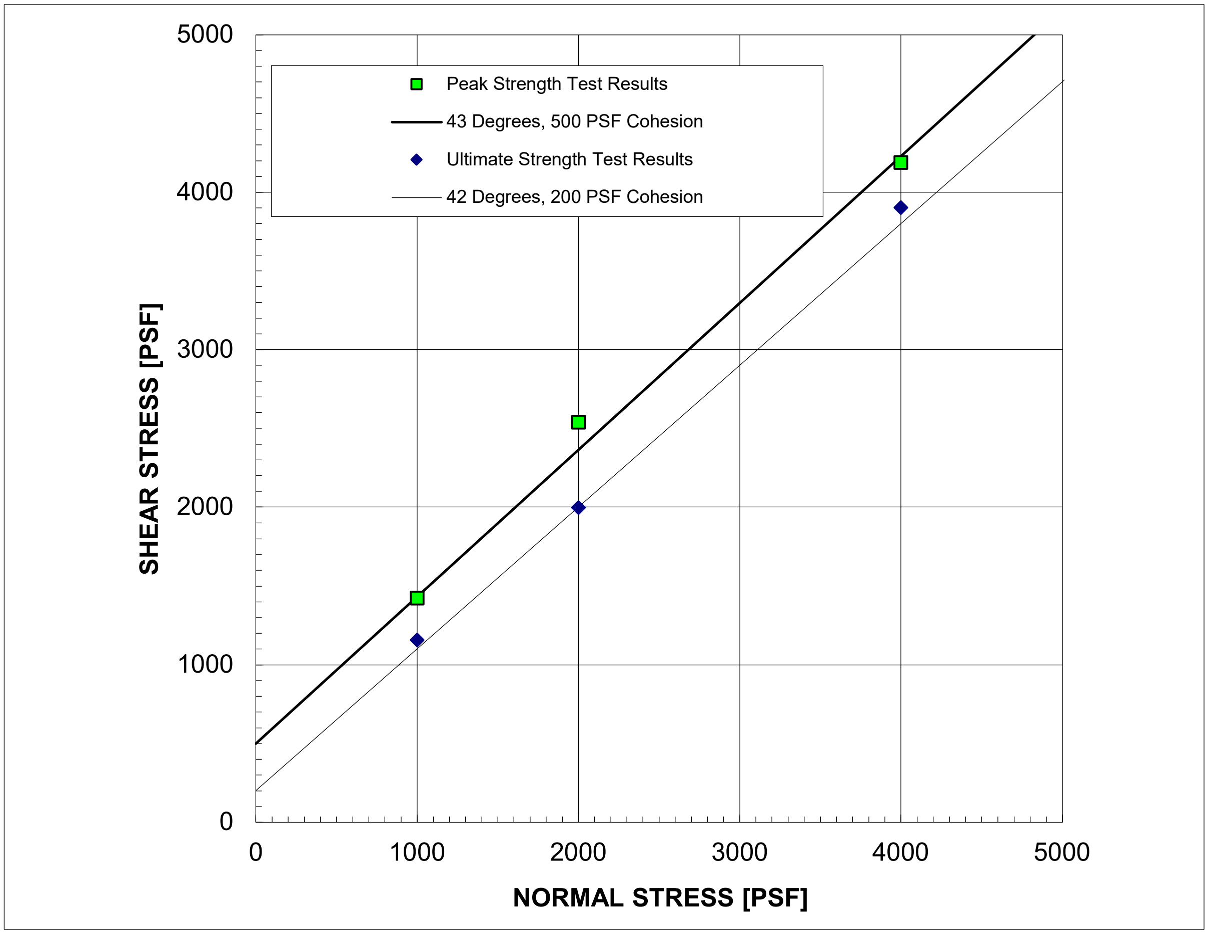
	IN-SITU	AS-TESTED
γ_d	117.0 PCF	117.0 PCF
w_c	14.4 %	16.3 %

GROUP DELTA

DIRECT SHEAR TEST RESULTS

Doucment No. 22-0036
Project No. SD724
FIGURE B-4.2





SAMPLE: B-2 @ 15'

ALLUVIUM (Qa):
Yellowish brown well graded sand (SW)

	PEAK	ULTIMATE
ϕ'	43 °	42 °
C'	500 PSF	200 PSF

STRAIN RATE: 0.0030 IN/MIN

(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	109.3 PCF	109.3 PCF
w_c	10.5 %	19.7 %

GROUP DELTA

DIRECT SHEAR TEST RESULTS

Doucment No. 22-0036
Project No. SD724
FIGURE B-4.3

SAMPLE NO.: B-5 **SAMPLE DATE:** 3/23/22

SAMPLE LOCATION: 1' - 5' **TEST DATE:** 4/11/22

SAMPLE DESCRIPTION: Dark reddish brown sandy lean clay (CL)

LABORATORY TEST DATA

	TEST SPECIMEN	**1**	**2**	**3**	**4**	**5**	
A	COMPACTOR PRESSURE	120	190	100			[PSI]
B	INITIAL MOISTURE	2.3	2.3	2.3			[%]
C	BATCH SOIL WEIGHT	1200	1200	1200			[G]
D	WATER ADDED	115	102	125			[ML]
E	WATER ADDED (D*(100+B)/C)	9.8	8.7	10.7			[%]
F	COMPACTION MOISTURE (B+E)	12.1	11.0	13.0			[%]
G	MOLD WEIGHT	2010.3	2012.6	2018.1			[G]
H	TOTAL BRIQUETTE WEIGHT	3181.0	3143.4	3144.7			[G]
I	NET BRIQUETTE WEIGHT (H-G)	1170.7	1130.8	1126.6			[G]
J	BRIQUETTE HEIGHT	2.56	2.42	2.50			[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))	123.6	127.6	120.9			[PCF]
L	EXUDATION LOAD	3052	5360	2281			[LB]
M	EXUDATION PRESSURE (L/12.54)	243	427	182			[PSI]
N	STABILOMETER AT 1000 LBS	47	37	53			[PSI]
O	STABILOMETER AT 2000 LBS	116	91	124			[PSI]
P	DISPLACEMENT FOR 100 PSI	5.08	4.71	5.85			[Turns]
Q	R VALUE BY STABILOMETER	16	29	11			
R	CORRECTED R-VALUE (See Fig. 14)	17	28	11			
S	EXPANSION DIAL READING	0.0005	0.0013	0.0003			[IN]
T	EXPANSION PRESSURE (S*43,300)	22	56	13			[PSF]
U	COVER BY STABILOMETER	0.87	0.75	0.93			[FT]
V	COVER BY EXPANSION	0.17	0.43	0.10			[FT]

TRAFFIC INDEX:	5.0
GRAVEL FACTOR:	1.53
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	21
R-VALUE BY EXPANSION:	28
R-VALUE AT EQUILIBRIUM:	21

*Note: Gravel factor estimated from pavement section using CTM 301, Section C, Part b.

REV. 2, DATED 1/31/15

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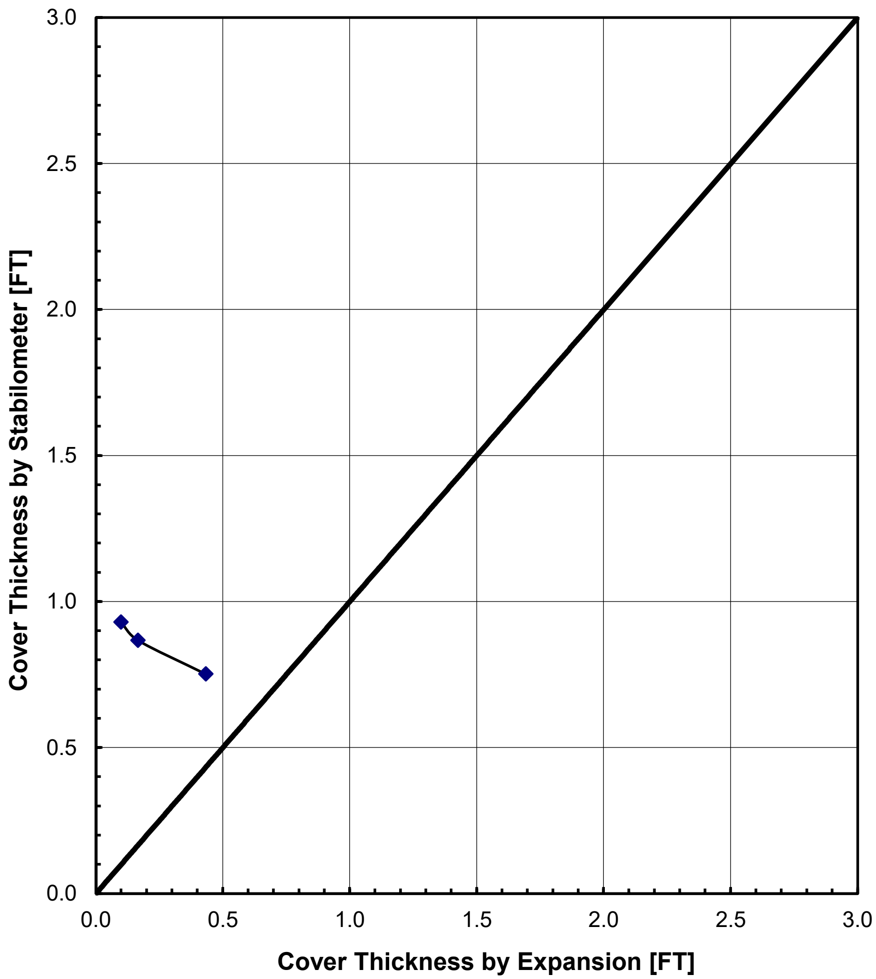
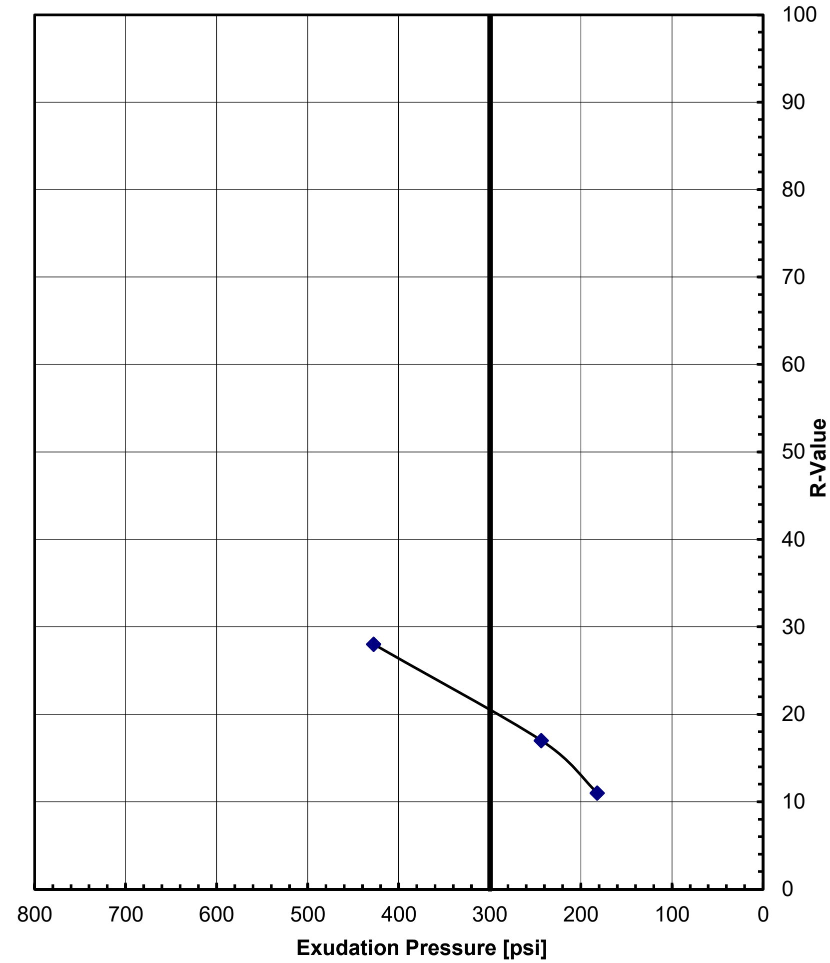
R-VALUE TEST RESULTS
CT301

Document No. 22-0036
Project No. SD713
FIGURE B-5.1.1

Sample: B-5 @ 1' - 5'



R-Value at Equilibrium: 21



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COVER AND EXUDATION CHARTS

Document No. 22-0036
Project No. SD713
FIGURE B-5.1.2

SAMPLE NO.: B-6

SAMPLE LOCATION: 0' - 5'

SAMPLE DESCRIPTION: Dark brown sandy lean clay (CL)

SAMPLE DATE: 3/23/22

TEST DATE: 4/11/22

LABORATORY TEST DATA

	TEST SPECIMEN	1	2	3	4	5	
A	COMPACTOR PRESSURE	100	130	200			[PSI]
B	INITIAL MOISTURE	3.7	3.7	3.7			[%]
C	BATCH SOIL WEIGHT	1200	1200	1200			[G]
D	WATER ADDED	130	115	100			[ML]
E	WATER ADDED (D*(100+B)/C)	11.2	9.9	8.6			[%]
F	COMPACTION MOISTURE (B+E)	14.9	13.6	12.3			[%]
G	MOLD WEIGHT	2078.3	2010.0	2019.3			[G]
H	TOTAL BRIQUETTE WEIGHT	3198.4	3104.6	3159.2			[G]
I	NET BRIQUETTE WEIGHT (H-G)	1120.1	1094.6	1139.9			[G]
J	BRIQUETTE HEIGHT	2.54	2.43	2.45			[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))	116.3	120.1	125.5			[PCF]
L	EXUDATION LOAD	2201	4475	7659			[LB]
M	EXUDATION PRESSURE (L/12.54)	176	357	611			[PSI]
N	STABILOMETER AT 1000 LBS	46	37	28			[PSI]
O	STABILOMETER AT 2000 LBS	118	100	80			[PSI]
P	DISPLACEMENT FOR 100 PSI	4.79	3.96	3.39			[Turns]
Q	R VALUE BY STABILOMETER	16	27	42			
R	CORRECTED R-VALUE (See Fig. 14)	16	26	42			
S	EXPANSION DIAL READING	0.0004	0.0029	0.0070			[IN]
T	EXPANSION PRESSURE (S*43,300)	17	126	303			[PSF]
U	COVER BY STABILOMETER	0.88	0.77	0.61			[FT]
V	COVER BY EXPANSION	0.13	0.97	2.33			[FT]

TRAFFIC INDEX:	5.0
GRAVEL FACTOR:	1.53
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	23
R-VALUE BY EXPANSION:	23
R-VALUE AT EQUILIBRIUM:	23

*Note: Gravel factor estimated from pavement section using CTM 301, Section C, Part b.

REV. 2, DATED 1/31/15

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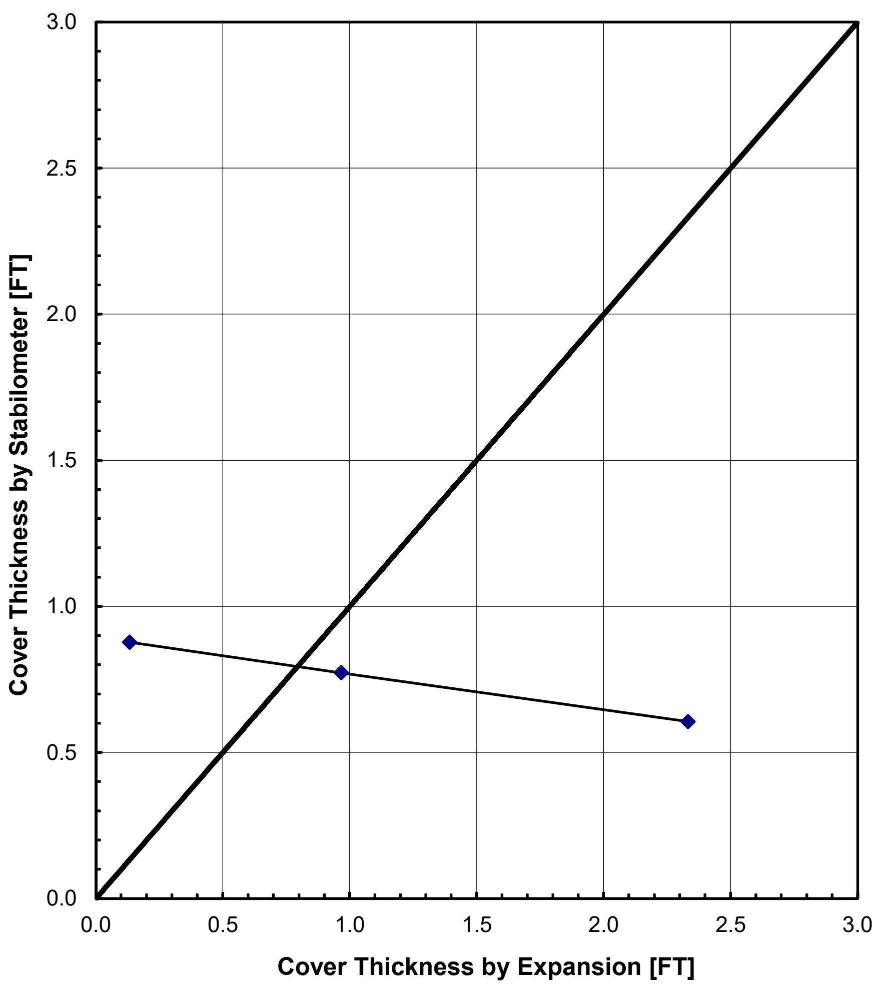
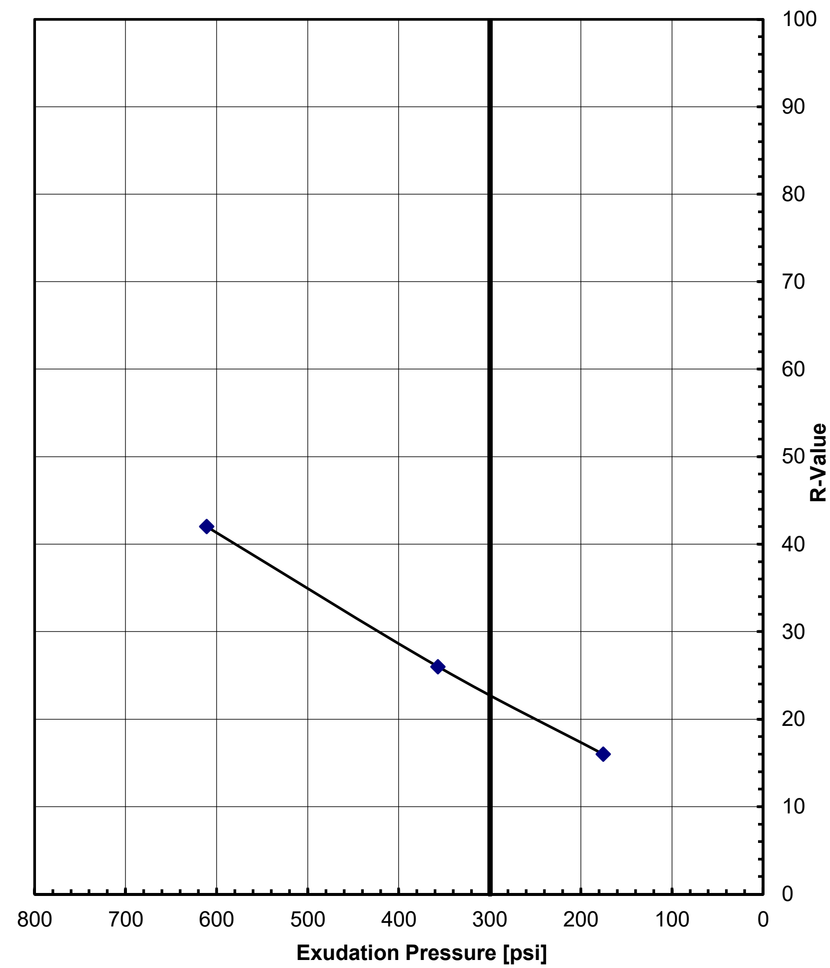
R-VALUE TEST RESULTS
CT301

Document No. 22-0036
Project No. SD713
FIGURE B-5.2.1

Sample: B-6, 0' - 5'

R-Value at Equilibrium: 23





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COVER AND EXUDATION CHARTS

Document No. 22-0036
Project No. SD713
FIGURE B-5.2.2

APPENDIX C
LIQUEFACTION ANALYSIS

APPENDIX C

LIQUEFACTION ANALYSES

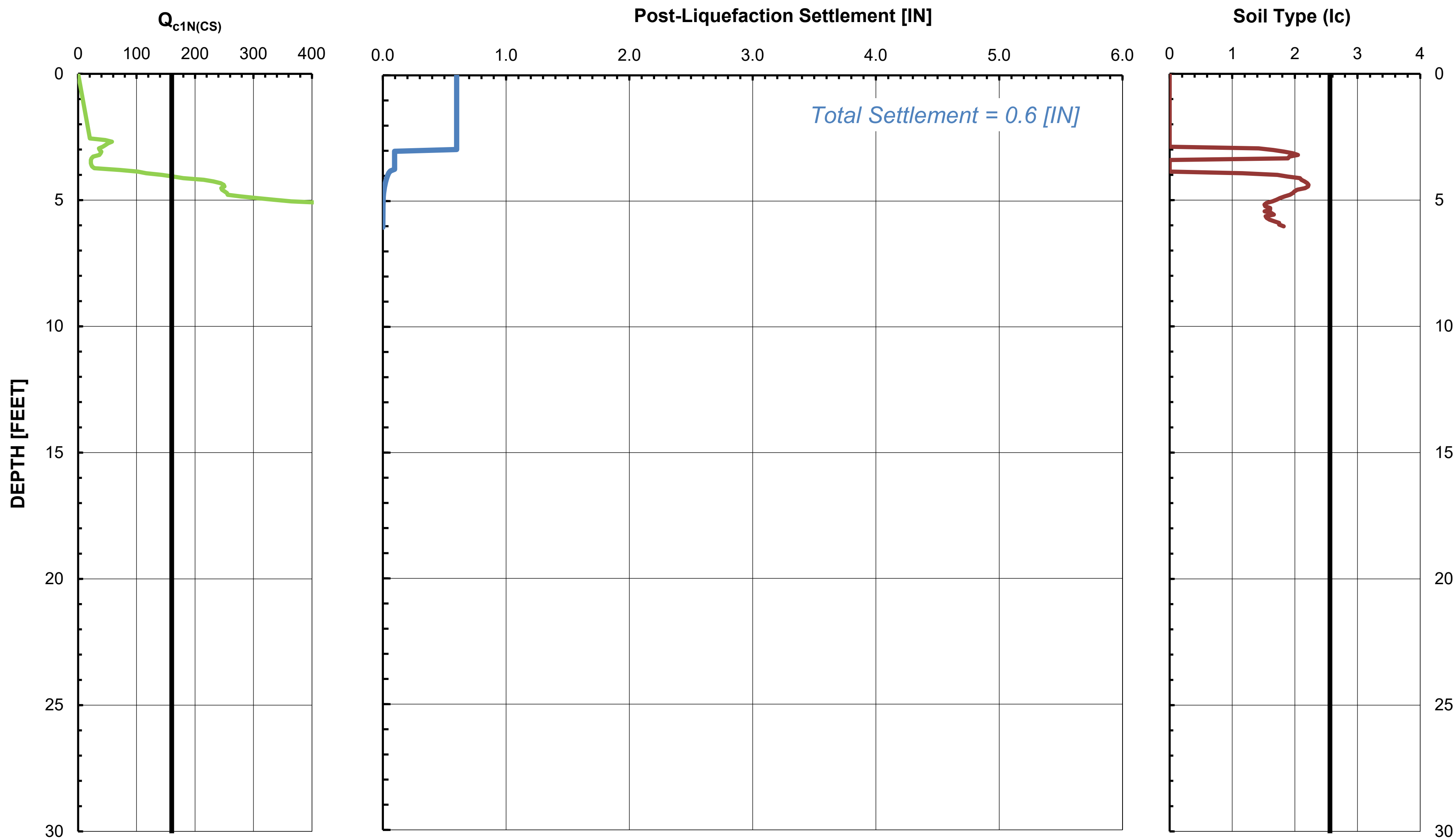
Liquefaction analyses were performed using the data gathered from the CPT soundings. The results are shown in Figures C-1 to C-6. The analyses were based on the procedures originally developed by Seed and Idriss and were conducted in general accordance with the recommended procedures for liquefaction analyses described in Section C4.4 of ASCE 61-14 (ASCE, 2014). The tip resistance (q_t) was normalized for overburden pressure and corrected for fines content (Youd et al., 2001). The fines correction was based on the Soil Behavior Type Index Ic (Robertson, 2010).

For each CPT sounding, the uncorrected Cone Resistance, Normalized Cone Resistance, the Soil Behavior Type (SBT), Factor of Safety against liquefaction, and estimated vertical settlement are plotted versus depth. The seismic demand used for the liquefaction analyses was equal to the Maximum Considered Earthquake Geometric Mean acceleration adjusted for site effects (PGA_M~ 0.644g), based on the requirements of Section 11.8.3 of ASCE 7-16 for a Seismic Design Category D. A groundwater level of roughly 15-feet below pad grades was assumed for all the analyses.

The vertical settlement plots for each CPT sounding show the estimated range of dynamic settlement resulting from a seismic demand equal to the MCE acceleration of 0.644g. At depths where the seismically induced shear stress exceeds the stress required to cause liquefaction, the Factor of Safety is less than 1.0, and seismic settlement may occur. However, fine-grained soils with an Ic value greater than 2.6 are considered too clayey to liquefy, and granular soils with a normalized tip resistance greater than 160 are considered too dense to liquefy. Soils that are both loose enough and sandy enough to liquefy contribute to the post-liquefaction settlement.

Each of the CPT analyses were conducted using three different assumptions. In the first figure for each CPT sounding (Case A), a spreadsheet was used to estimate seismic settlement with no data averaging. These analyses were then compared to results from a commercially available program CLiq V3.3.1.14, with the CPT data averaged across 3 depth increments (Case B), and with a thin layer correction applied (Case C). The results of these parametric liquefaction analyses are tabulated below, along with the average settlement from the three different methods.

Figure No.	Exploration No.	A) Settlement (Raw CPT Data)	B) Settlement (Data Averaging)	C) Settlement (Thin Layer)	Average Settlement
C-1	CPT-1	0.6 Inches	0.4 Inches	0.4 Inches	**0.5 Inches**
C-2	CPT-2	1.5 Inches	0.8 inches	0.8 Inches	**1.0 Inches**
C-3	CPT-3	0.5 Inches	0.3 Inches	0.2 Inches	**0.3 Inches**
C-4	CPT-4	1.0 Inches	0.1 Inches	0.1 Inches	**0.4 Inches**
C-5	CPT-5	1.4 Inches	0.5 Inches	0.4 Inches	**0.8 Inches**
C-6	CPT-6	1.5 Inches	0.9 Inches	0.5 Inches	**1.0 Inches**



GROUP DELTA

DYNAMIC SETTLEMENT (CPT-1)
(Seismic Demand ~ 0.644g)

Document No. 22-0036
Project No. SD724
FIGURE C-1

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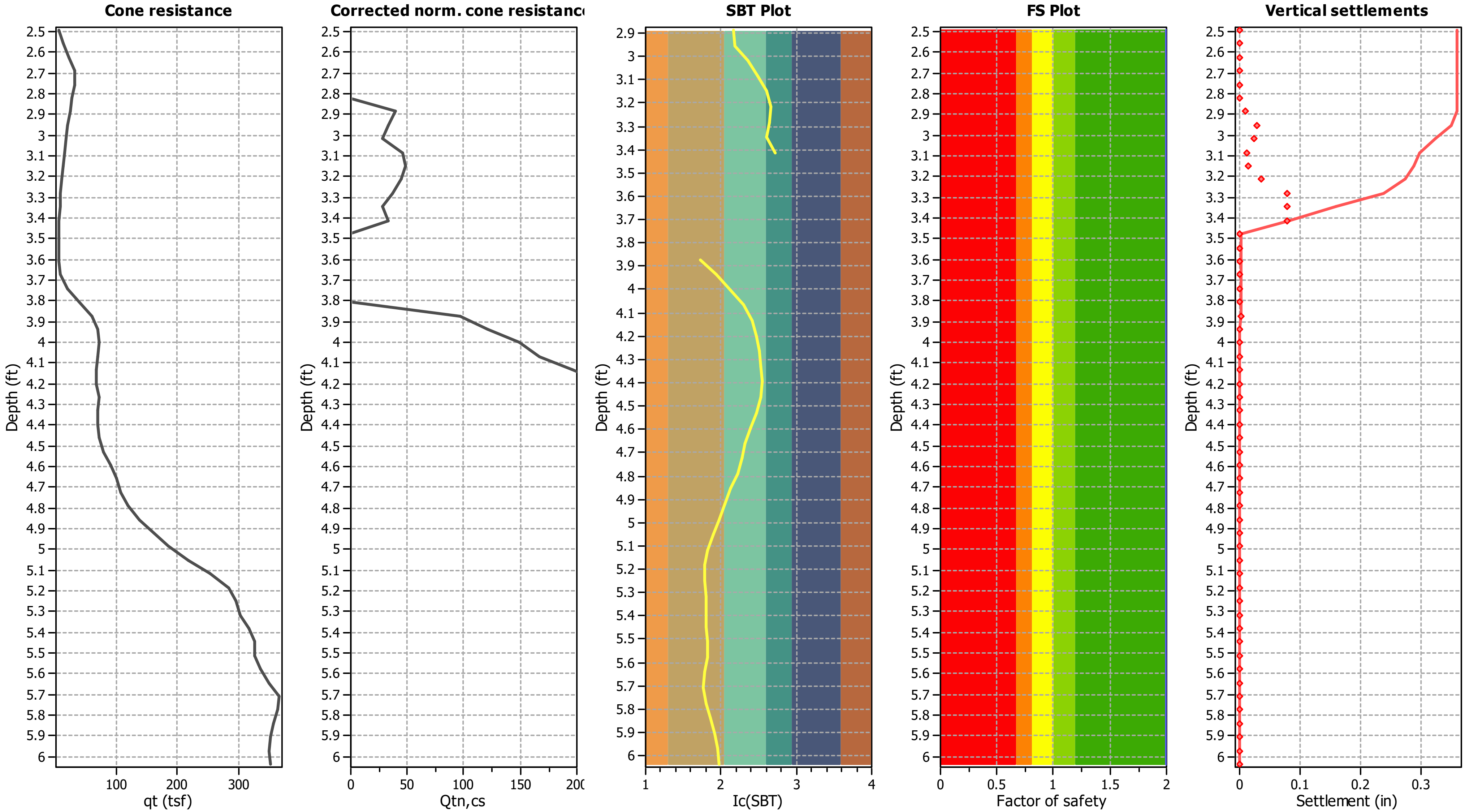
San Diego, California 92126
http://www.GroupDelta.com

Project: **USD Group Biofuels Terminal**
Location: **837 19th Street, National City, CA**

CPT: CPT-1
Total depth: 6.04 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	No		

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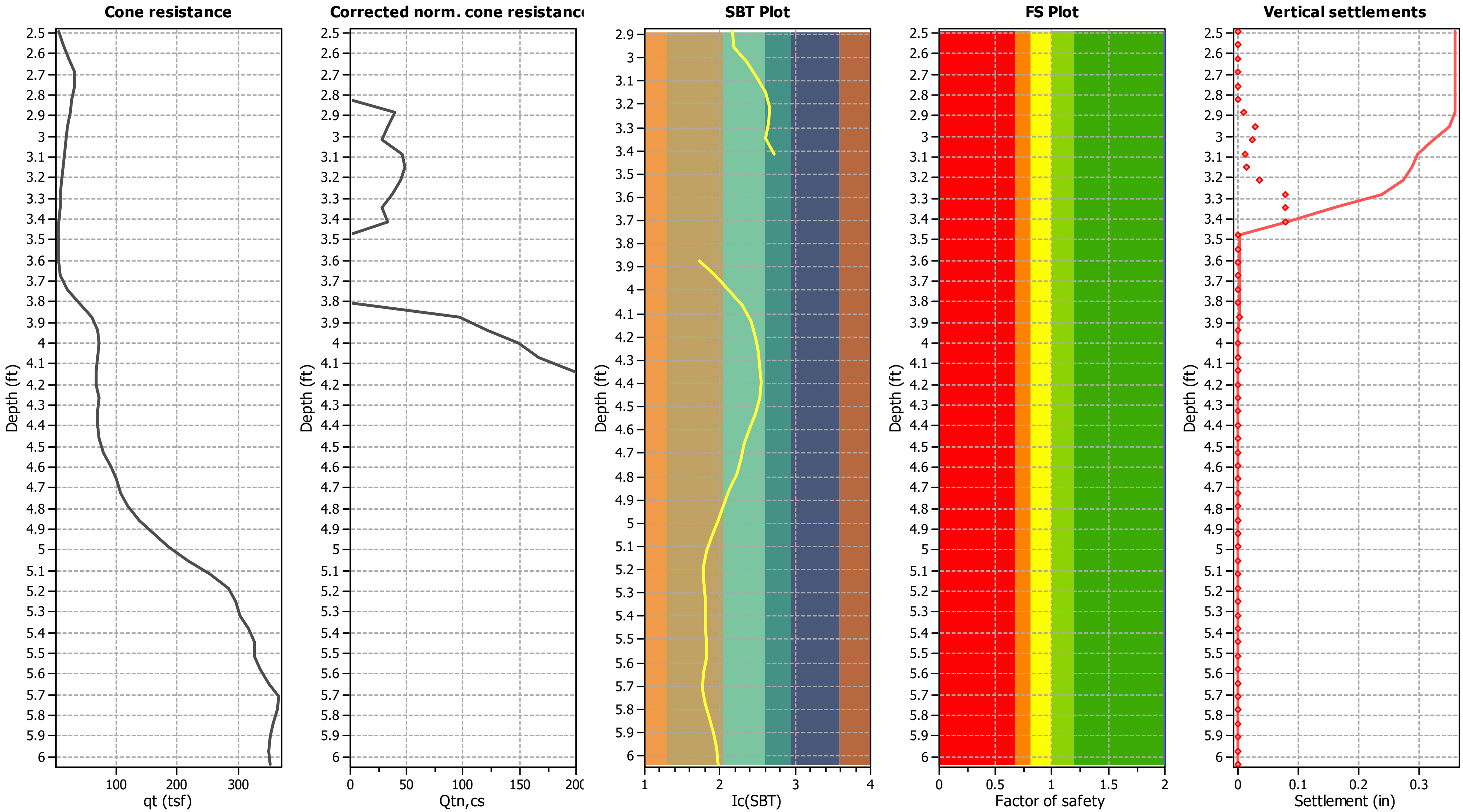
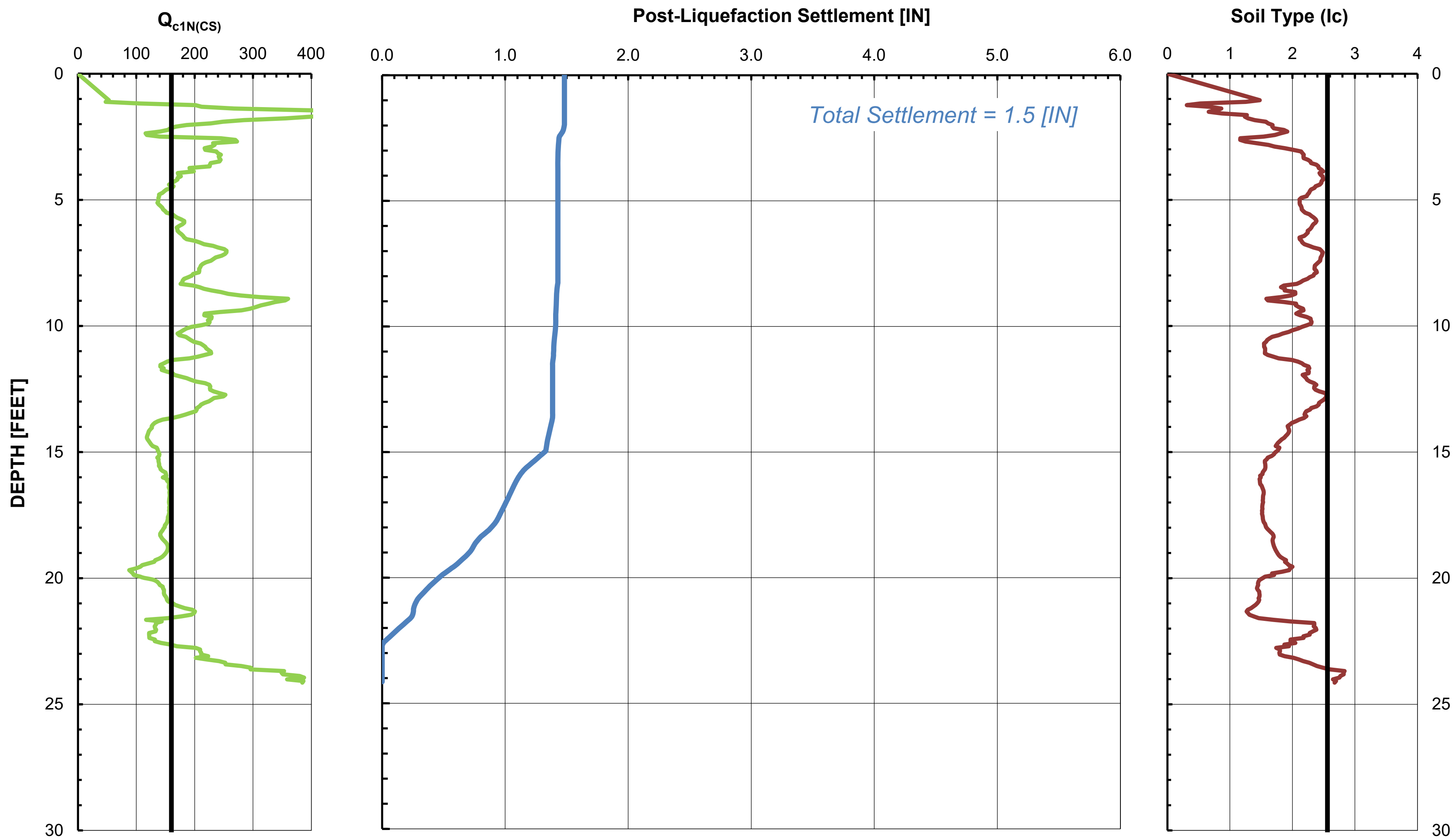
San Diego, California 92126
http://www.GroupDelta.com

Project: USD Group Biofuels Terminal
Location: 837 19th Street, National City, CA

CPT: CPT-1
Total depth: 6.04 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



GROUP DELTA

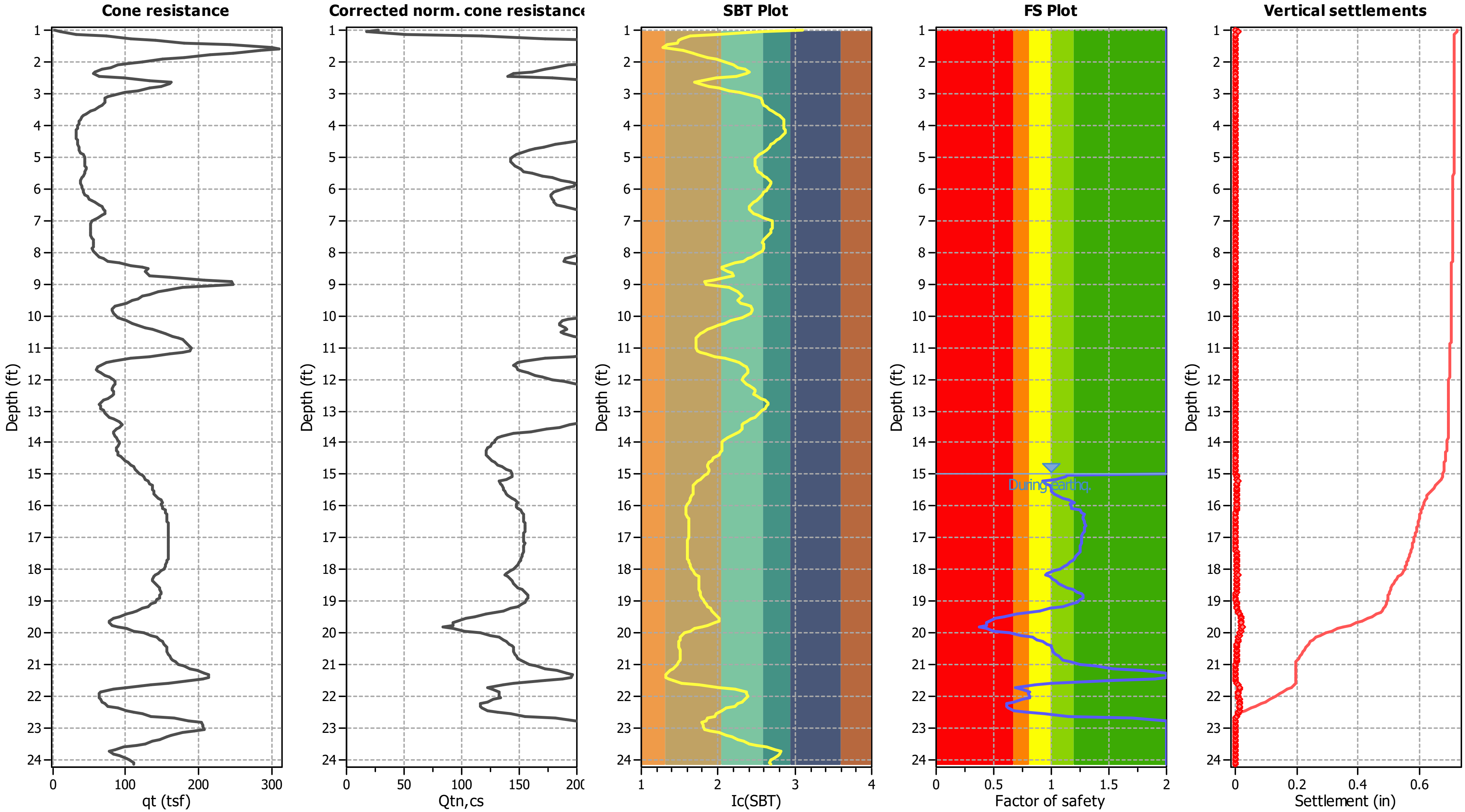
DYNAMIC SETTLEMENT (CPT-2)
(Seismic Demand ~ 0.644g)

Document No. 22-0036
Project No. SD724
FIGURE C-2

GROUP DELTA
Group Delta Consultants
9245 Activity Road, Suite 103
San Diego, California 92126
http://www.GroupDelta.com

Project: USD Group Biofuels Terminal
Location: 837 19th Street, National City, CA

CPT: CPT-2
Total depth: 24.15 ft

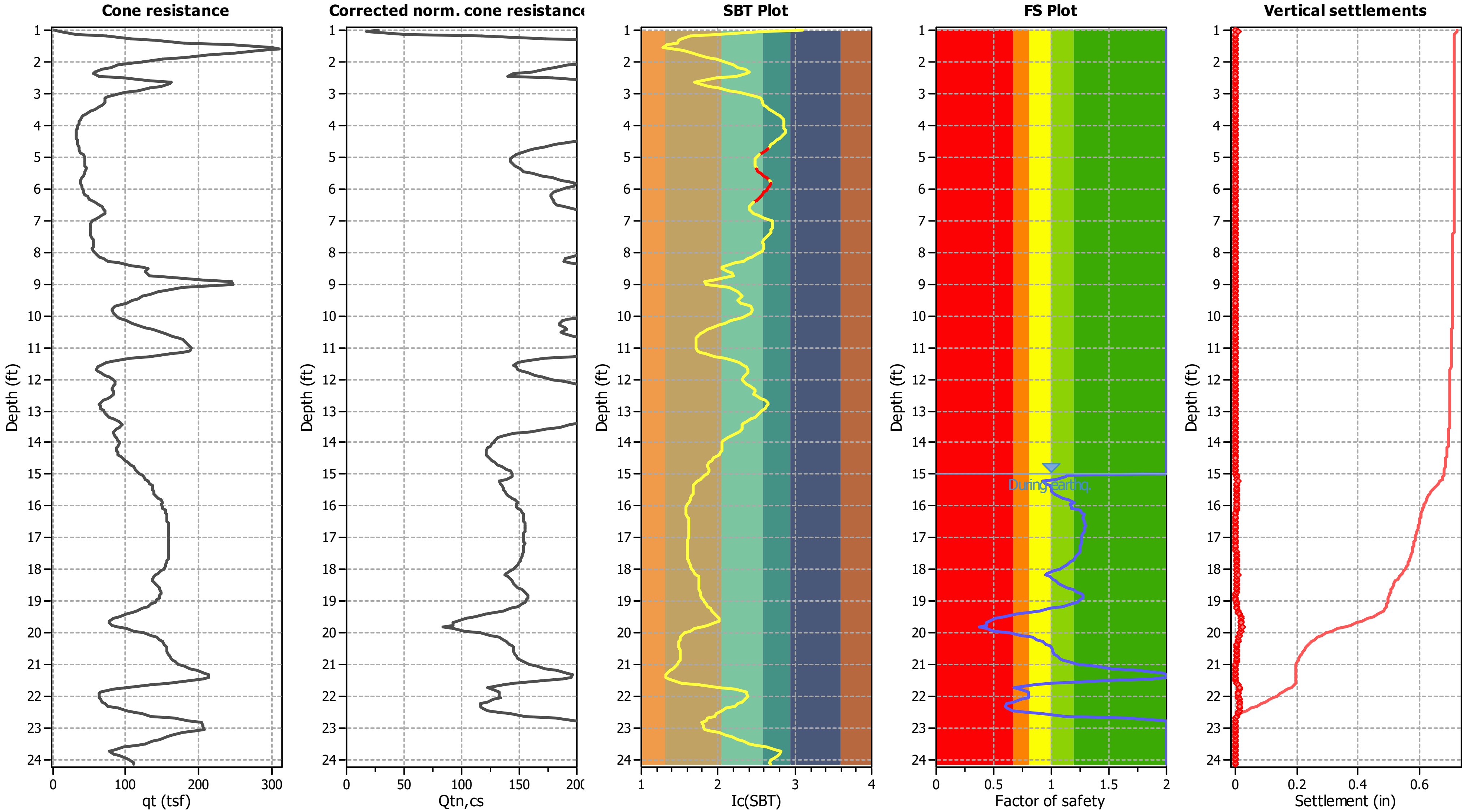


Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	No		

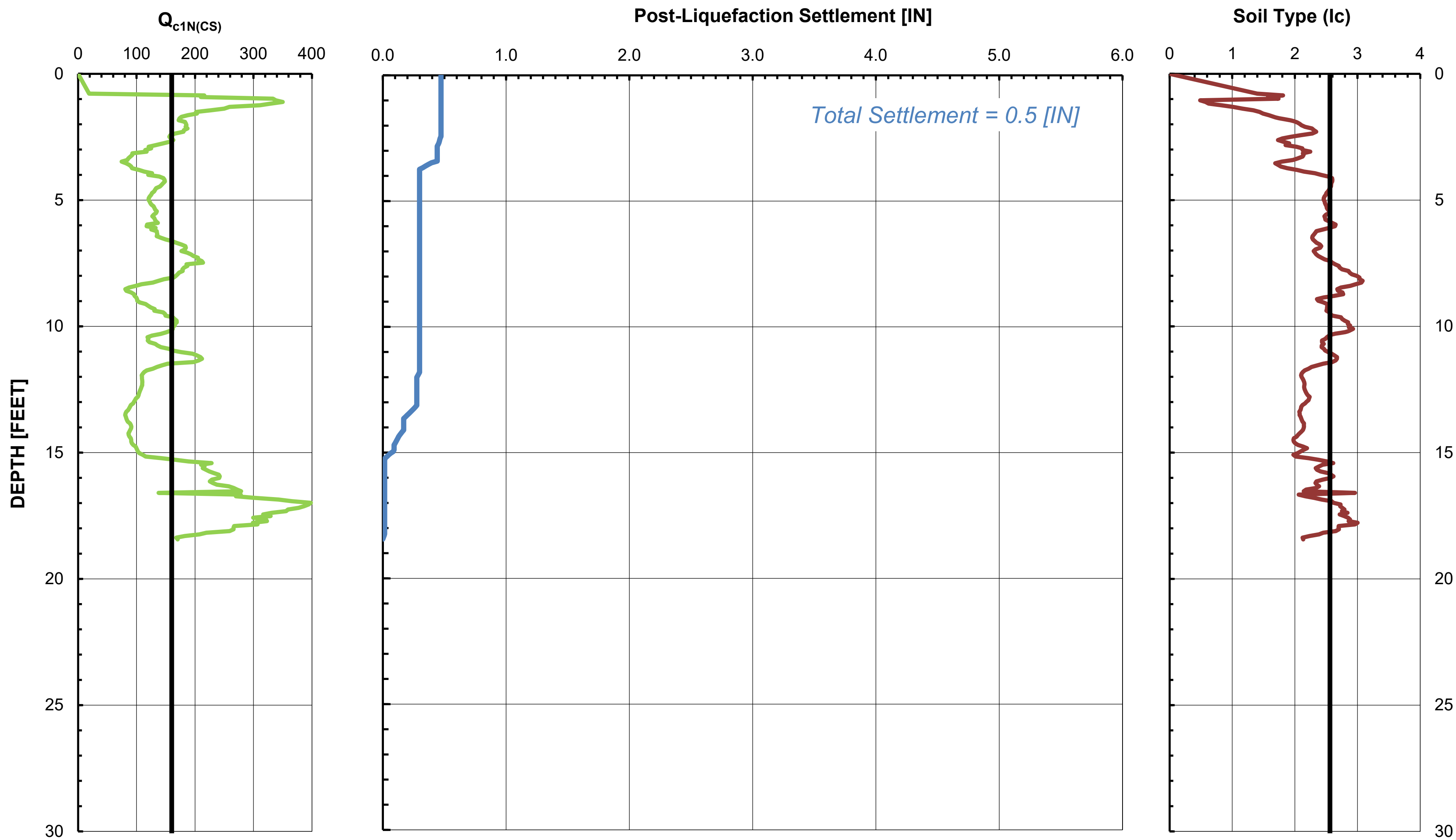
Group Delta Consultants
9245 Activity Road, Suite 103
San Diego, California 92126
http://www.GroupDelta.com

Project: USD Group Biofuels Terminal
Location: 837 19th Street, National City, CA

CPT: CPT-2
Total depth: 24.15 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



GROUP DELTA

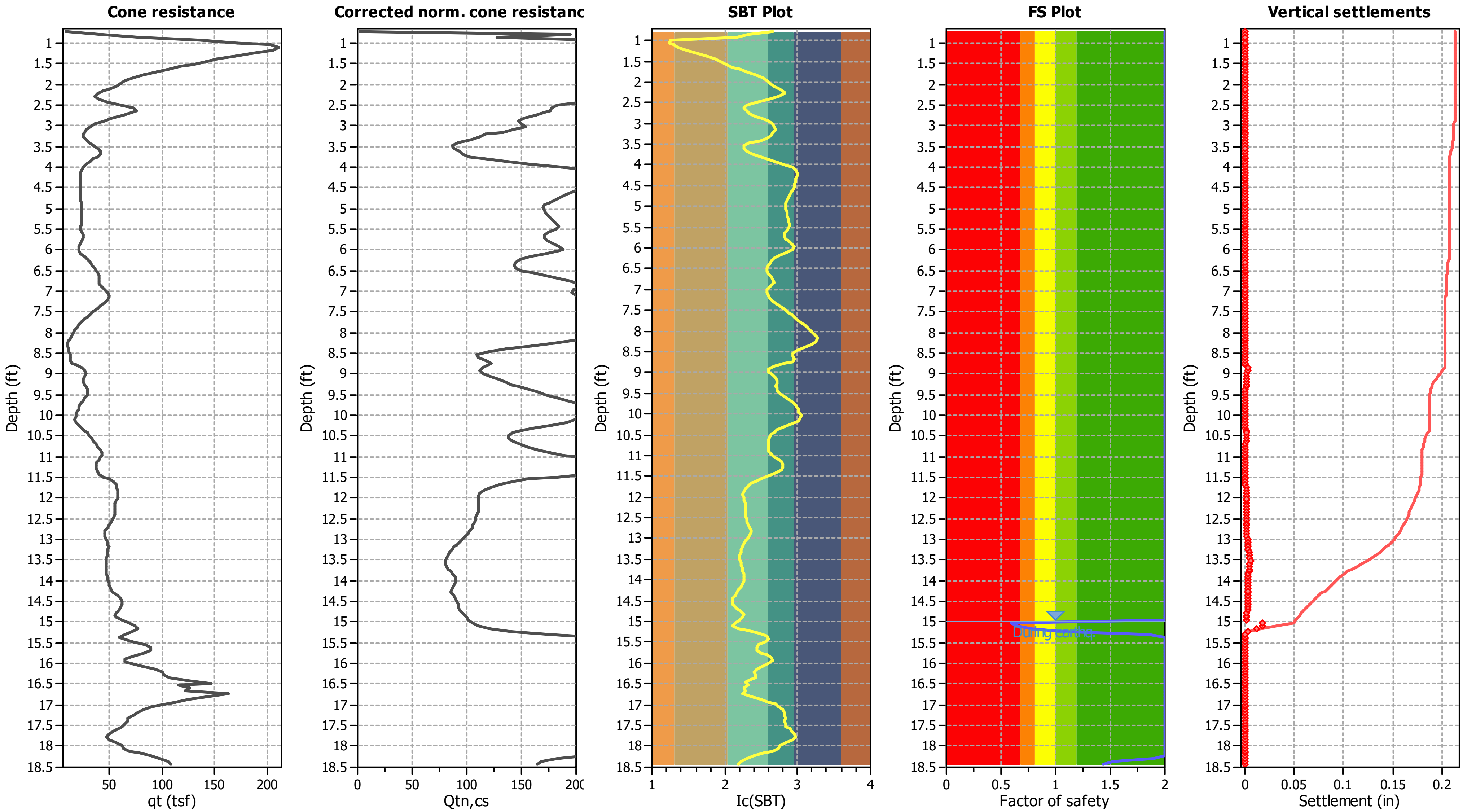
DYNAMIC SETTLEMENT (CPT-3)
(Seismic Demand ~ 0.644g)

Document No. 22-0036
Project No. SD724
FIGURE C-3

Group Delta Consultants
9245 Activity Road, Suite 103
San Diego, California 92126
http://www.GroupDelta.com

Project: **USD Group Biofuels Terminal**
Location: **837 19th Street, National City, CA**

CPT: CPT-3
Total depth: 18.44 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	No		

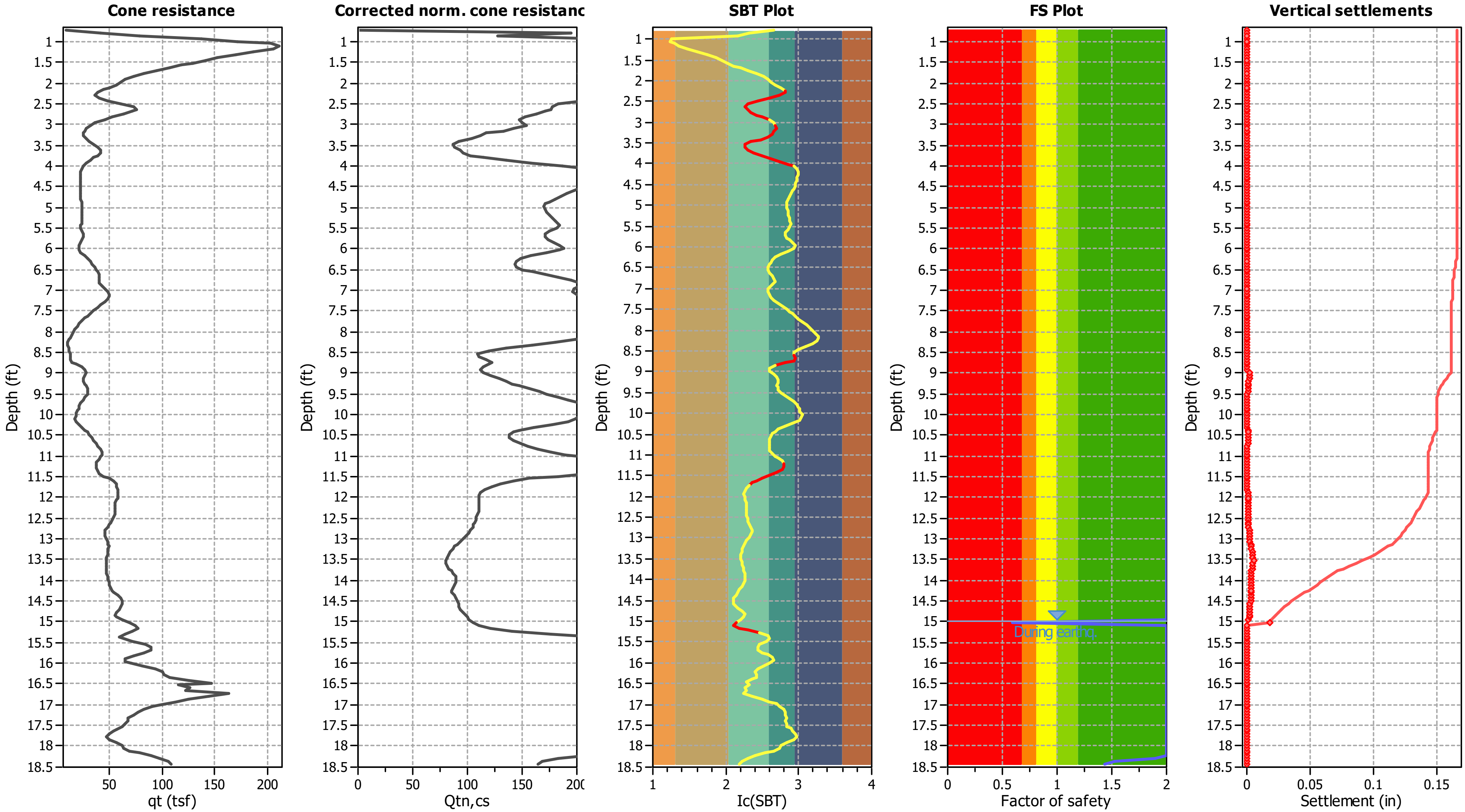
Group Delta Consultants
9245 Activity Road, Suite 103
San Diego, California 92126
http://www.GroupDelta.com

Project: USD Group Biofuels Terminal

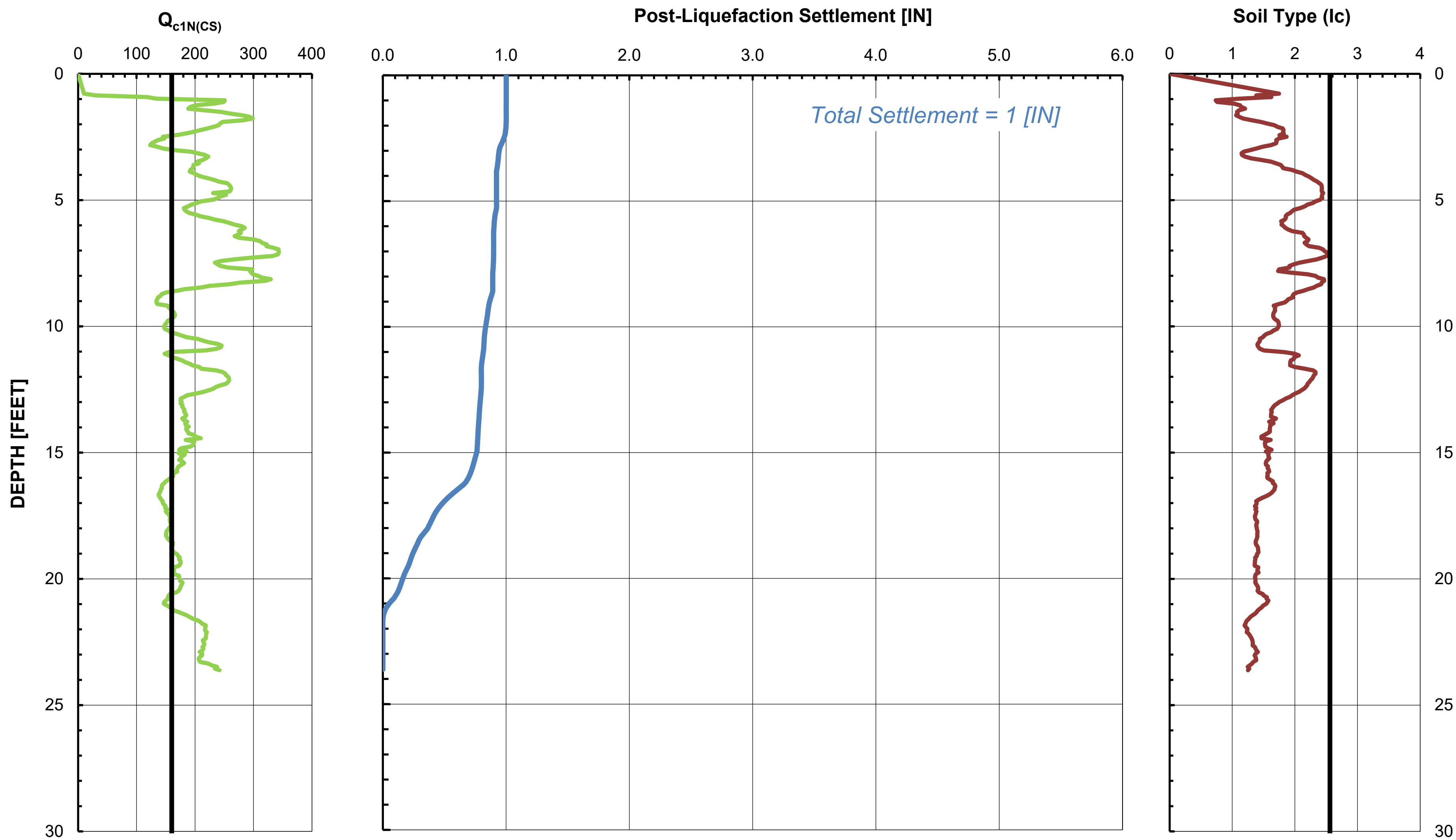
Location: 837 19th Street, National City, CA

CPT: CPT-3

Total depth: 18.44 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



GROUP DELTA

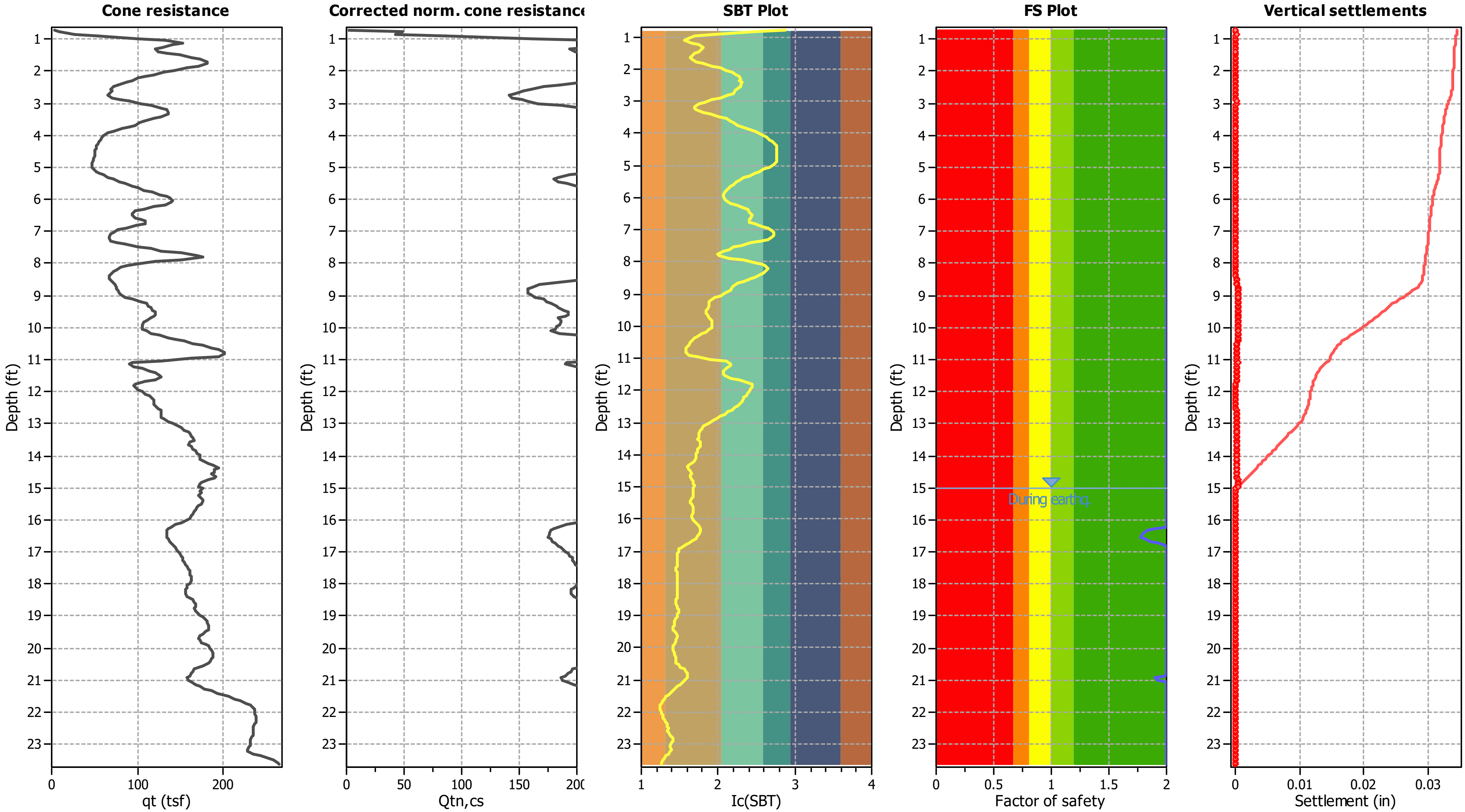
DYNAMIC SETTLEMENT (CPT-4)
(Seismic Demand ~ 0.644g)

Document No. 22-0036
Project No. SD724
FIGURE C-4

Group Delta Consultants
9245 Activity Road, Suite 103
San Diego, California 92126
http://www.GroupDelta.com

Project: **USD Group Biofuels Terminal**
Location: **837 19th Street, National City, CA**

CPT: CPT-4
Total depth: 23.62 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	No		

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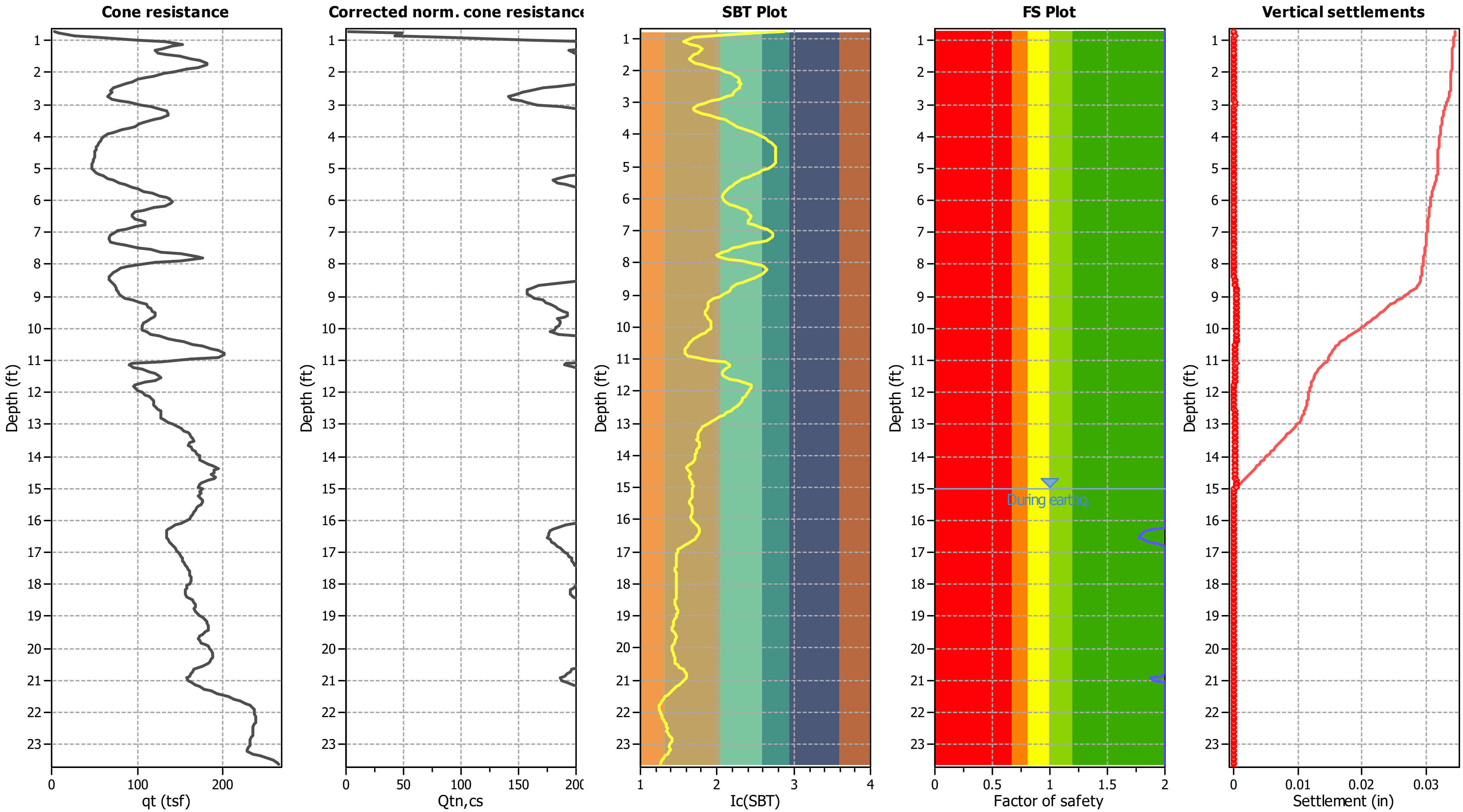
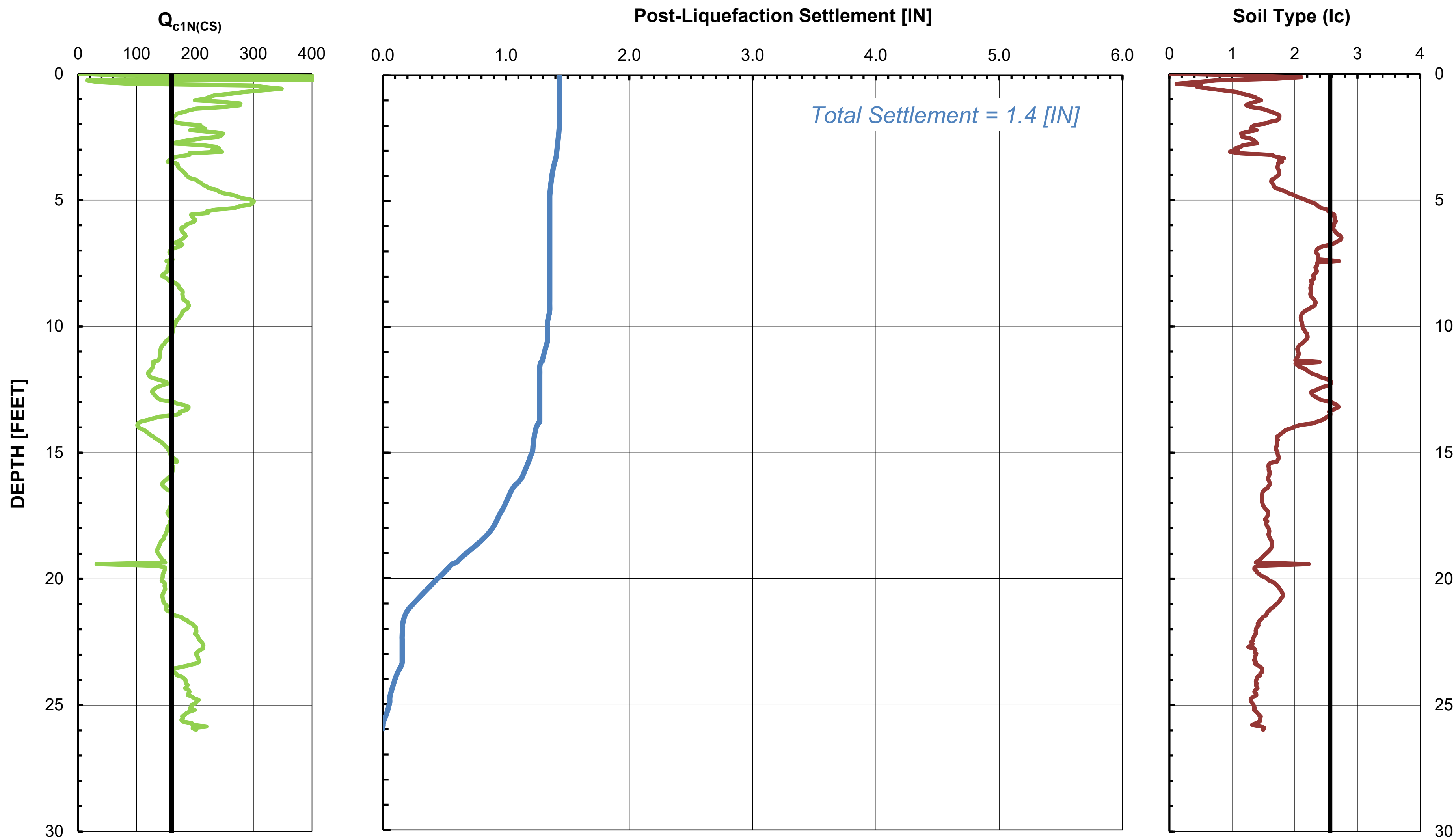
San Diego, California 92126
http://www.GroupDelta.com

Project: **USD Group Biofuels Terminal**
Location: **837 19th Street, National City, CA**

CPT: CPT-4
Total depth: 23.62 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



GROUP DELTA

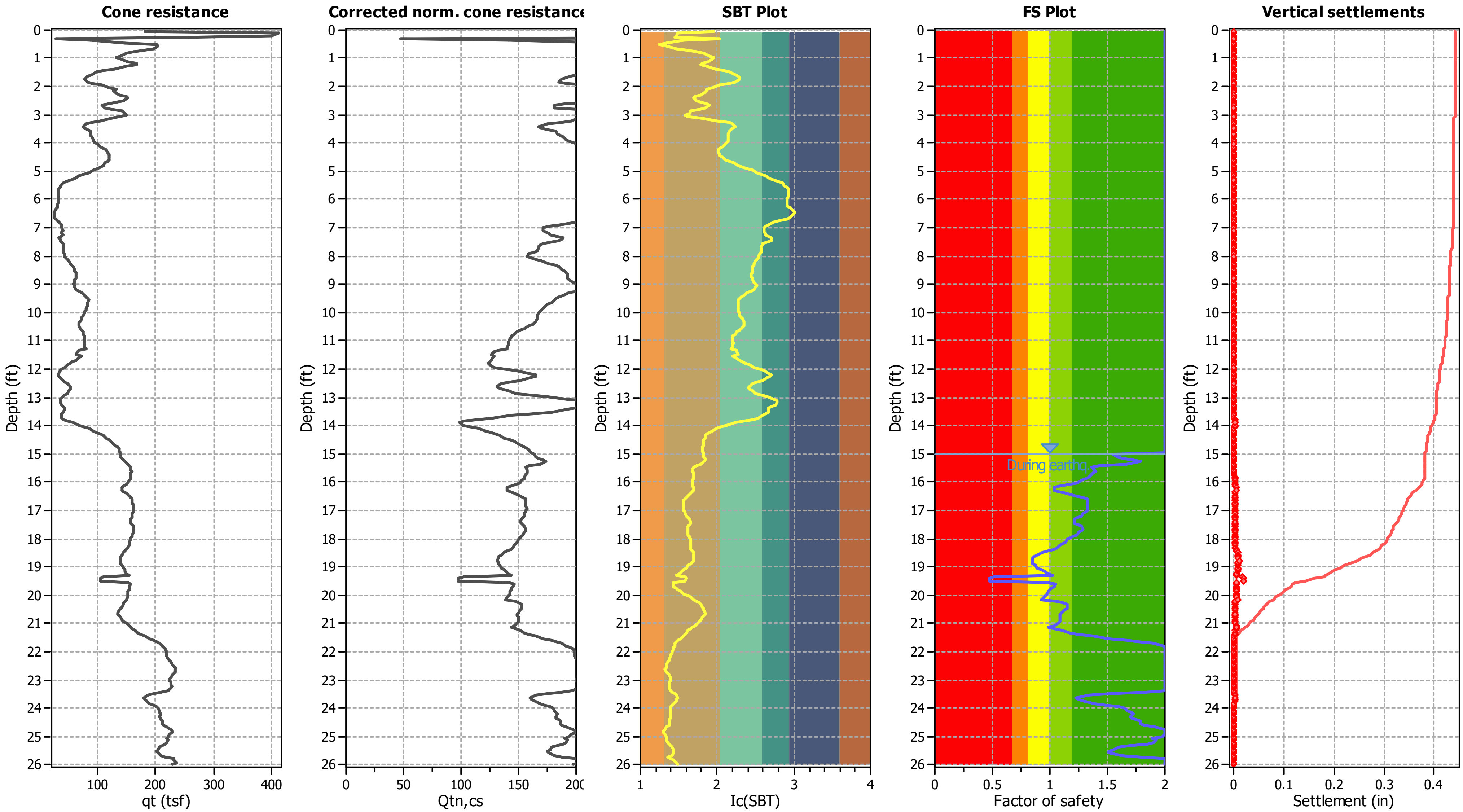
DYNAMIC SETTLEMENT (CPT-5)
(Seismic Demand ~ 0.644g)

Document No. 22-0036
Project No. SD724
FIGURE C-5

Group Delta Consultants
9245 Activity Road, Suite 103
San Diego, California 92126
http://www.GroupDelta.com

Project: **USD Group Biofuels Terminal**
Location: **837 19th Street, National City, CA**

CPT: CPT-5
Total depth: 25.98 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	No		

Group Delta Consultants
9245 Activity Road, Suite 103
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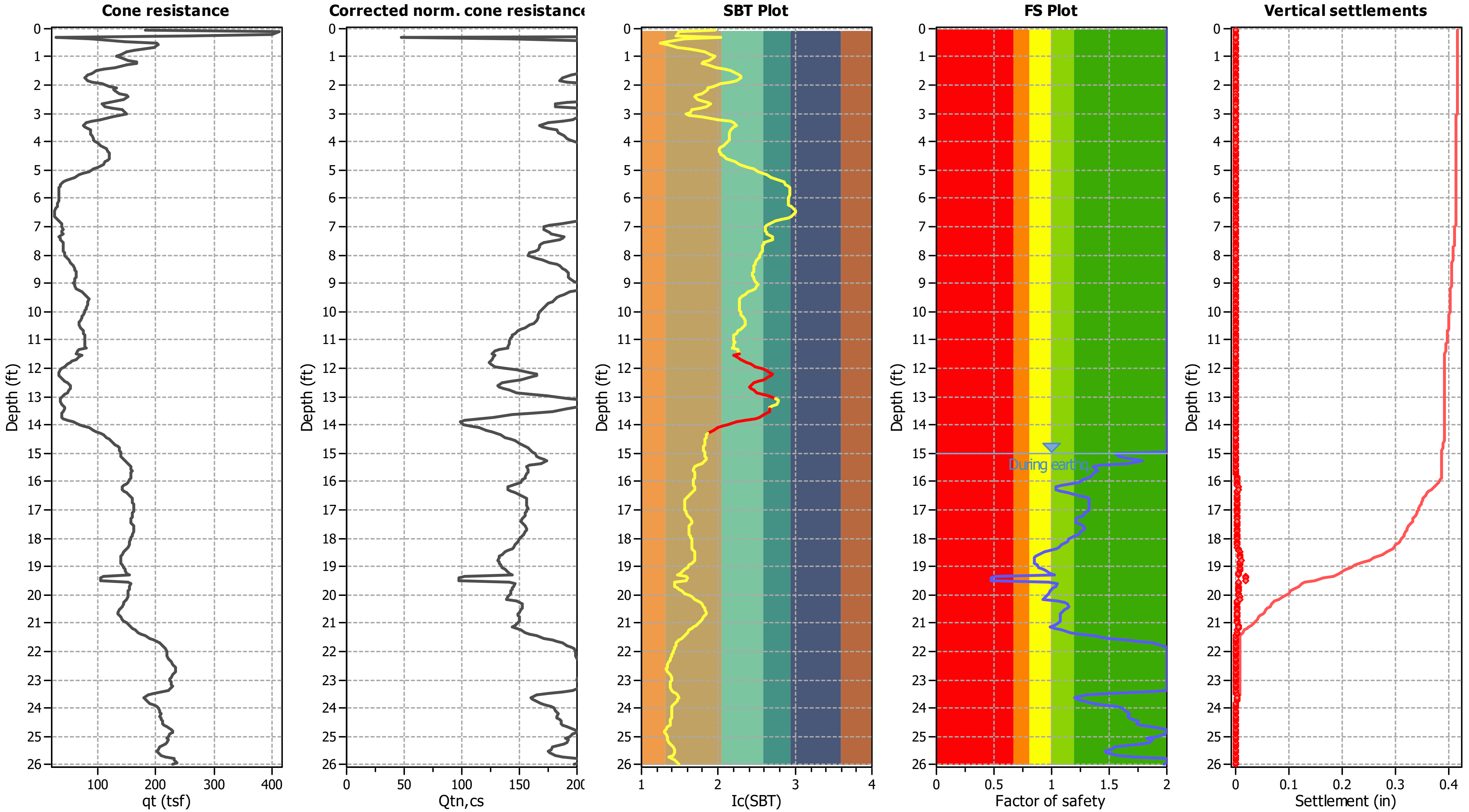
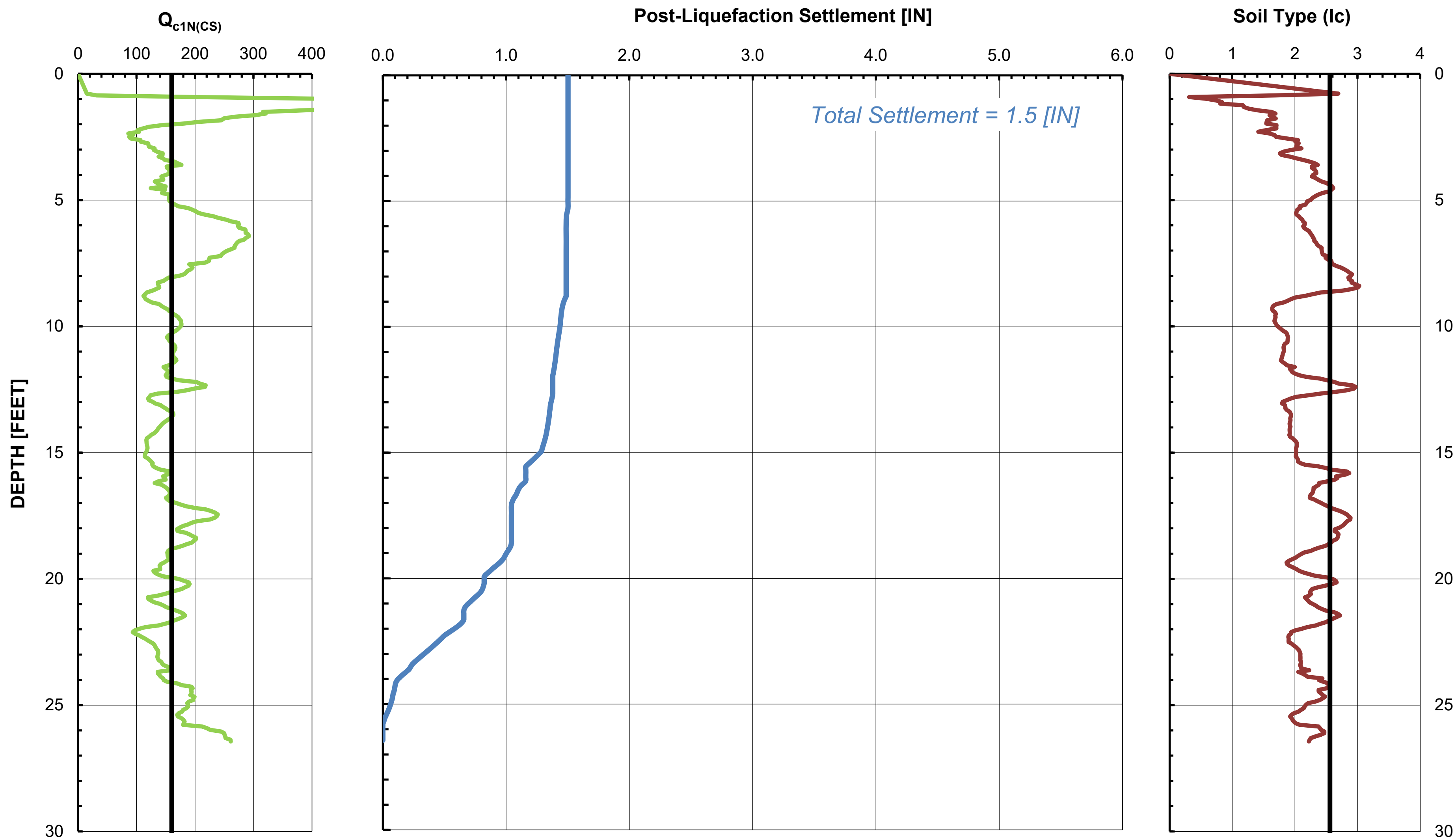
http://www.GroupDelta.com

Project: **USD Group Biofuels Terminal**
Location: **837 19th Street, National City, CA**

CPT: CPT-5
Total depth: 25.98 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



GROUP DELTA

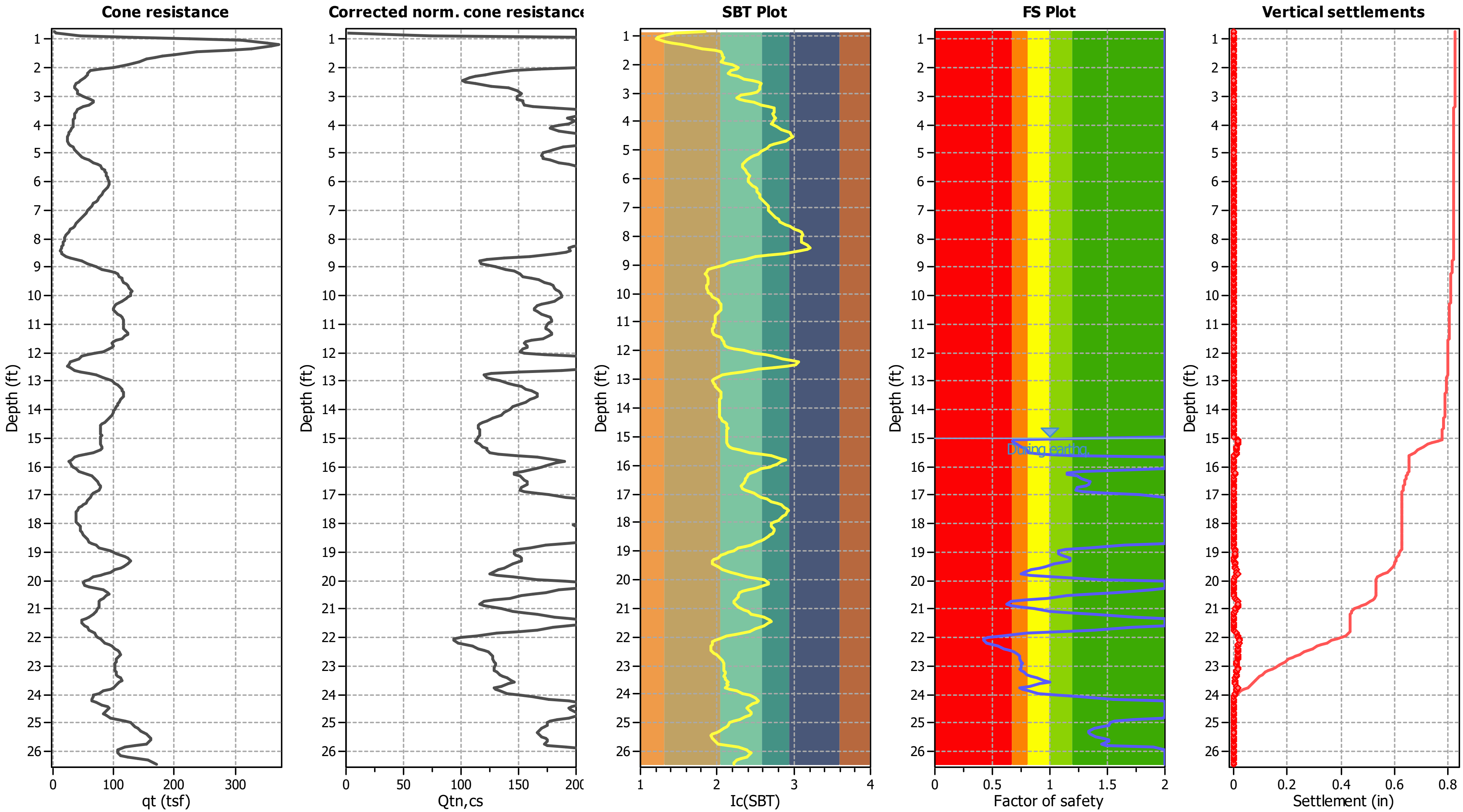
DYNAMIC SETTLEMENT (CPT-6)
(Seismic Demand ~ 0.644g)

Document No. 22-0036
Project No. SD724
FIGURE C-6

Group Delta Consultants
9245 Activity Road, Suite 103
San Diego, California 92126
http://www.GroupDelta.com

Project: **USD Group Biofuels Terminal**
Location: **837 19th Street, National City, CA**

CPT: CPT-6
Total depth: 26.44 ft

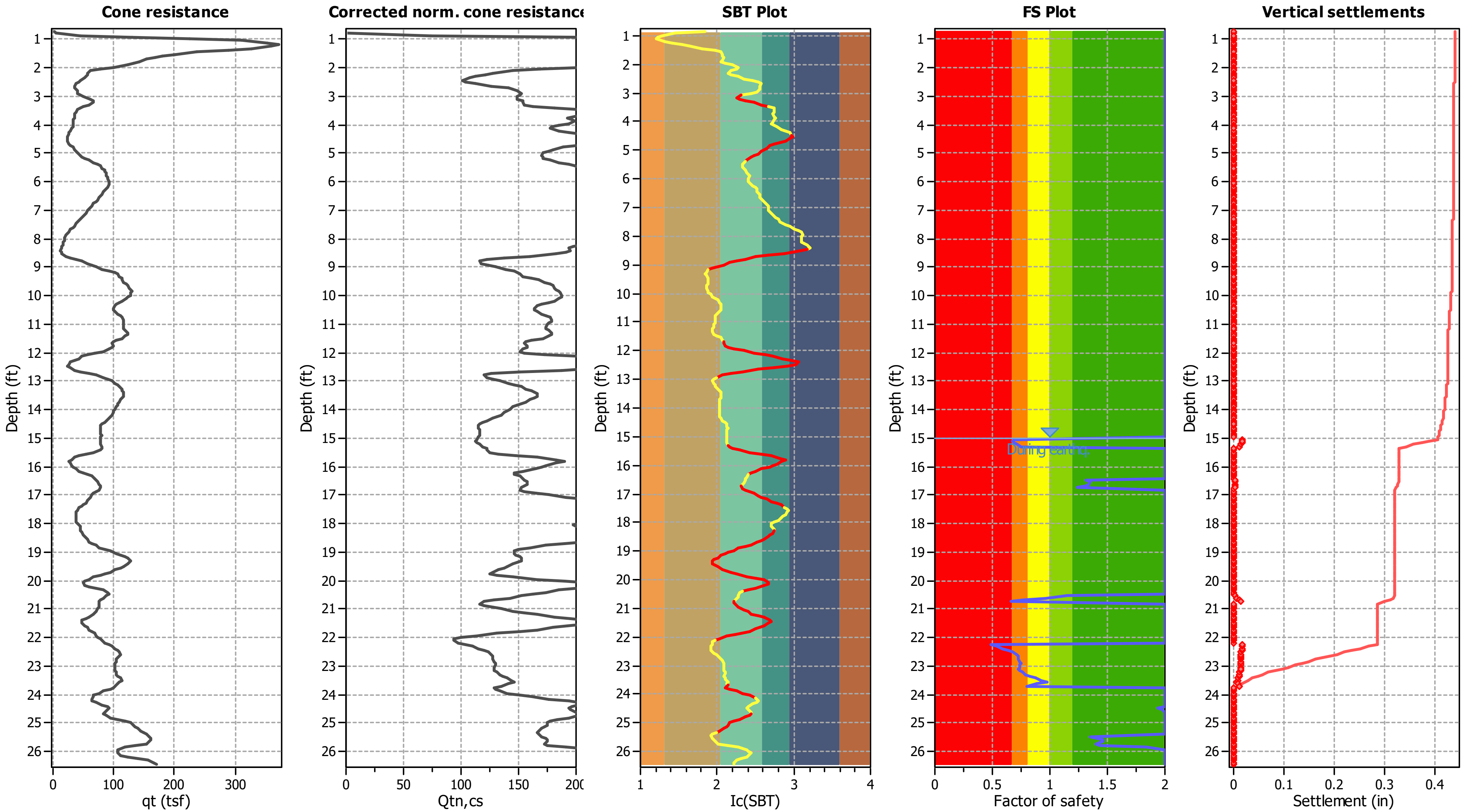


Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	No		

Group Delta Consultants
9245 Activity Road, Suite 103
San Diego, California 92126
http://www.GroupDelta.com

Project: **USD Group Biofuels Terminal**
Location: **837 19th Street, National City, CA**

CPT: CPT-6
Total depth: 26.44 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		