Appendix E

Geology and Soils Technical Memorandum

MEMORANDUM

To:	Michael Haberkorn, Gatzke Dillon & Ballance
From:	Perry Russell, Dudek
Subject:	SDSU Brawley Sciences Building Project Technical Memo – Geology and Soils
Date:	August 16, 2023
cc:	Sarah Lozano, Kirsten Burrowes, Dudek
Attachments:	A – Figures B – Geotechnical Report

Dudek has conducted an evaluation pursuant to the requirements of the California Environmental Quality Act (CEQA), California Public Resources Code 21000, et seq., to determine the presence and potential impacts related to geology and soils associated with the proposed San Diego State University (SDSU) Imperial Valley Campus Brawley Sciences Building Project (project or proposed project), located east of Brawley, California. This technical memorandum provides the results of the geology and soils investigation.

1 Project Location and Setting

The project is located at 560 California State Route (SR) 78 (also referred to as Ben Hulse Highway) in Imperial County, east of the city of Brawley. Regional access to the campus is provided by SR 111 and SR 86 to the west and northwest, respectively, and SR 115 to the east (See Attachment A: Figure 1). The proposed project site is surrounded by agricultural uses to the north, south, and west. Undeveloped land and a solar farm are located directly east of the proposed project site. The proposed Science Building would be constructed northeast of existing campus Building 101, and the associated parking lot. Project construction staging areas would occupy the area of campus located southeast of the site and north of SR 78 (See Attachment A: Figure 2).

2 Project Description

In September 2003, CSU certified an environmental impact report and approved a Campus Master Plan for development of the SDSU Brawley Campus (Brawley Campus or campus), which would serve as an extension of the existing SDSU Imperial Valley Campus (IVC) located in Imperial County. The IVC is an extension of SDSU's main campus located in San Diego and furthers the university's regional educational mission to provide additional educational opportunities to the outlying communities of Imperial County. The approved Campus Master Plan and certified environmental impact report (EIR) provided sufficient environmental analysis and authorization necessary for enrollment of up to 850 full-time equivalent (FTE) students and corresponding faculty and staff, and a framework for development of the facilities necessary to serve the approved campus enrollment.

The Brawley Campus is approximately 200 acres in size and is located east of the city of Brawley (city). Currently, the Campus has been partially built out with educational and support facilities, although much of the campus remains undeveloped or used for active agriculture. As noted above, the environmental impacts associated with development of the Brawley Campus, including a student enrollment up to 850 FTE, were evaluated at a program level of review in the previously certified 2003 SDSU Imperial Valley Campus Master Plan Project EIR (2003 EIR) (SCH 200251010). In CSU's effort to build out the IVC consistent with the previously approved Campus Master Plan, SDSU now proposes construction and operation of a sciences research and instruction facility that would be located on the Brawley Campus.

The proposed project involves the construction and operation of a STEM building (science, technology, engineering, and mathematics) that would house teaching labs, lecture spaces, faculty/administration offices, research spaces, and conference rooms, as well as mechanical, electrical, and telecom support spaces. The proposed project does not include/propose any increase in the previously authorized and approved maximum student enrollment of 850 FTE.

The proposed project site is approximately 3.2-acres in size and the construction staging areas would occupy approximately 1-acre in the area of campus located southeast of the site and north of SR 78. The project includes 61,119 sf of on-site landscaping, including the construction of bio-retention areas to capture stormwater runoff from stormwater drainages systems that will be located throughout the project site. Hardscape improvements will include 41,297 sf of sidewalks and pedestrian walkways, which will connect the project site to existing campus buildings and parking lot.

Additionally, the project will require new points of connection to domestic water, fire water, and sewer lines from existing utility lines to serve the new building, as well as new domestic water line infrastructure. Potable water will be provided by the city of Brawley, as well as sewer and wastewater collection services. New utility infrastructure will also be required to support electrical services for the building, as well as a back-up diesel operated generator.

The proposed project building would have an area of 36,900 gross sf and would be approximately 35 feet in height. The project is projected to be built over the course of 19 months, with construction estimated to begin in January 2024.Construction and equipment staging would require 1 acre of space within the campus, directly east of the existing building (Building 101) and parking lot. The project would involve site preparation, grading, and excavation associated with project construction. Excavation depths are anticipated to be 2 to 5 feet. Waste (i.e., excavated gravel/soil) generated during project construction would be balanced within the site.

3 Analysis Methodology

The analysis presented here considers the potential environmental impacts of the proposed project relative to existing conditions. Establishment of the project site's existing geology and soils conditions has been prepared using information contained in the previously certified 2003 SDSU Imperial Valley Campus Master Plan Project EIR (SDSU 2003), combined with updated information, as applicable from the California Geological Survey (CGS), Southern California Earthquake Data Center, U.S. Geological Survey (USGS), Imperial County General Plan (Seismic and Public Safety Element), and Imperial County General Plan EIR. In addition, the results of a March 2023, project-specific geotechnical report, by Group Delta (Attachment B, Geotechnical Report), have been incorporated into the existing conditions section and impact analysis.

4 Geology and Soils

4.1 Existing Conditions

Regional Geology

The Brawley Campus lies within the Salton Trough, the dominant landform within Imperial County. The Salton Trough encompasses the Coachella, Imperial, and Mexicali valleys and extends north from the Gulf of California. The lowest part of the basin is the bed of the prehistoric Lake Cahuilla, with its ancient beach line at about 35 feet above mean sea level. The deepest portion is covered by the Salton Sea with a water surface level measured at 226 feet below mean sea level at its highest level in April 1986. The geologic structure of the trough is a result of an evolving “rift” in the earth’s crustal plates. As the crust thins due to the “spreading” of the trough, magma rises closer to the surface, heating deep groundwater. Nonmarine and alluvium sediments cover large portions of the area. An unexposed succession of Tertiary- and Quaternary-age sedimentary rocks lies below the alluvial and lake bottom sediments, ranging in depth from 11,000 feet or greater at the margins to over 20,000 feet in the central portions of the Salton Trough. The valley is drained by an 8,360 square mile watershed, which eventually empties into the Salton Sea (SDSU 2003).

Soils

Soils in the Brawley Campus area consist of over 100 feet of late Pleistocene to Holocene lacustrine (i.e., lake) deposits associated with ancient Lake Cahuilla, overlain by shallow fill. Borings drilled on-site indicated the site is underlain by 1 to 2 feet of fill material, consisting of fat clay. Laboratory testing of these surficial clays indicate these soils have a high to very high expansion potential and range in consistency from soft to stiff. The underlying lacustrine sediments are typically unconsolidated to poorly consolidated and porous, consisting generally of clay, silt, and silty sand. Borings drilled on-site, to a maximum depth of 88.5 feet, encountered several approximate 2- to 3-foot thick beds of silty sand between depths of 18 to 28 feet below ground surface (bgs). An additional approximate 10-foot thick layer of medium dense to dense, sandy materials was also encountered at depths of 50 to 60 feet bgs (Appendix A).

The artificial fill material is derived from native surficial soils. Between one half and two thirds of the Brawley Campus is covered by soils generally identified as Imperial, described as nearly level, moderately well drained, silty clay in lacustrine basins. Imperial-Glenbar occurs over the remainder of the site. This soil type refers to nearly level, moderately slow draining silty clay loams in the lacustrine basin (SDSU 2003).

Faulting and Seismicity

Surface fault rupture is the displacement of ground surface that occurs along a fault line during an earthquake event. Based on criteria established by the CGS, faults are classified as either Holocene-active, pre-Holocene, or age-undetermined. Faults are considered active when they have shown evidence of movement within the past 11,700 years (i.e., Holocene epoch). Pre-Holocene faults, also known as potentially active faults, are those that have shown evidence of movement more than 11,700 years ago and generally before 1.6 million years (Quaternary age). Faults whose age of most recent movement is not known or is unconstrained by dating methods or by limitations in stratigraphic resolution are considered age-undetermined and inactive (CGS 2018).

The Alquist-Priolo Earthquake Fault Zoning Act (formerly known as the Alquist-Priolo Special Studies Zones Act) established state policy to identify active faults and determine a boundary zone on either side of a known fault trace, called the Alquist-Priolo Earthquake Fault Zone. The delineated width of an Alquist-Priolo Earthquake Fault is based on the location, precision, complexity, or regional significance of the fault and can be between 200 and 500 feet in width on either side of the fault trace. If a site lies within a designated Alquist-Priolo Earthquake Fault Zone, a geologic fault rupture investigation must be performed to demonstrate that a proposed building site is not threatened by surface displacement from the fault, before development permits may be issued (CGS 2018).

The Imperial Valley area is subjected to frequent seismic events, with related concerns of ground shaking and liquefaction. The most noteworthy of the numerous faults traversing the Salton Trough is the Holocene-active Coachella section of the San Andreas Fault. Two other major northwest-trending Holocene-active fault zones bounding the Salton Trough include the San Jacinto Fault on the northwest and the Elsinore Fault on the southwest (Figure 3, Regional Faulting). The potential for future large earthquakes on the San Andreas and San Jacinto fault zones is based on potential rupture scenarios associated with both fault zones, as movement on the San Jacinto Fault is dependent on movement of the southern San Andreas Fault Zone. Based on historic and pre-historic fault ruptures, the maximum worst-case earthquake on these two interrelated fault zones would be moment magnitude (Mw) 8.0. However, the probable maximum magnitude of is Mw 6.5 to Mw 7.5 for the San Jacinto Fault and Mw 6.8 to Mw 8.0 for the San Andreas Fault (Sanders 1993, USGS 2002, Scharer and Yule 2020, SCEDC 2023).

The Holocene-active Imperial and Brawley faults are the closest faults to the Brawley Campus. Recent studies indicate that these two faults are interrelated. As illustrated in Figure 3, the northern terminus of the Brawley Fault is approximately 2 miles south of the Brawley Campus and the northern terminus of the Imperial Fault is approximately 3.5 miles southwest of the campus (CGS 2023a). The Brawley Campus is not located in an Alquist-Priolo Earthquake Fault Zone associated with the Brawley or Imperial faults (CDMG 1990). The Imperial Fault Zone is the principal element of the San Andreas Fault System within the Salton Trough. Ground surface rupture has occurred twice during historic times, including 1940 and 1979 (and possibly in 1915), as evidence by offset of historic alluvium, lacustrine deposits, and cultural features. The 1940 earthquake produced surface rupture offsets up to 23 feet near the U.S.-Mexico border. Data from these earthquake events suggest a slip rate of 15 to 20 millimeters/year for the Holocene epoch (past 11,700 years). As discussed below, slip is transferred north through the Brawley Seismic Zone, and some slip may be transferred to the San Jacinto Fault Zone. The recurrence interval is 30 to 40 years for a 1979-style earthquake event and 270 to 700 years for a 1940-style earthquake. Others have postulated recurrence intervals of 40 years, 137 years, and 37 years, respectively, for the northern, central, and southern segments of the fault. In addition, the maximum probable earthquake magnitude for the Imperial Fault is Mw 6.5 to Mw 7.0 (Treiman 1999, SCEDC 2022, USGS 2022).

As illustrated in Figure 3, the Brawley Seismic Zone extends southeast 30 kilometers across the Salton Trough, from the southern-most tip of the San Andreas Fault to the Imperial Fault in the south. This seismic zone accommodates continental plate motion and rifting along the Pacific-North American plate boundary, at rates up to 17 millimeters per year, transferring slip from the San Andreas Fault to the Imperial Fault. The southern segment of the Brawley Seismic Zone is located approximately 3 miles west of the Brawley Campus. Seismicity along this seismic zone consists mostly of short-duration earthquake sequences of up to 10 days duration, and consist of foreshocks, mainshocks, and aftershocks. Approximately 4 to 6 kilometers of right lateral offset along the seismic zone and the presence of volcanic buttes reflect rift tectonics of crustal thinning, as well as recent volcanics at the south shore of the Salton Sea (Hauksson et al. 2021, USGS 2002).

The largest recorded earthquake in Imperial County occurred on the Imperial Fault in May 1940. This Richter magnitude 7.1 earthquake was centered on the international border, east of Calexico, and could be traced for approximately 50 miles, from the Volcano Lake in Mexico, north through the Imperial Valley, just north of Brawley. The newly completed All-American Canal was offset approximately 14 feet by movement on the fault and nine people died from the earthquake. In addition, a magnitude 6.6 earthquake occurred along the Imperial Fault in October 1979. The epicenter was 7 miles east of Calexico. No lives were lost but numerous structures and canals were damaged, including settlement of the All-American Canal up to 4 feet. Earthquake damage was estimated at $30 million. In addition, a magnitude 7.2 earthquake occurred near Calexico in April 2010.

Other substantial earthquakes in Imperial County include those occurring in 1892 (M7.1), 1915 (M6.3 and 7.1), 1930 (M5.7), 1950 (M5.4), 1957 (M5.2), 1968 (M6.5), 1980 (M6.1), 1981 (M5.8), 1987 (M6.2 and 6.8), and 2010 (M7.2). In addition to the faults described above, other active faults in the region include the Superstition Hills, Superstition Mountain, Laguna Salada, and Cerro Prieto faults. Currently, portions of the County are effected by a minor earthquake with a magnitude of 4.5 or less every few months. The County may experience an earthquake with a magnitude of 5.5 or greater every five years and dozens of micro-seismic events, with magnitudes of 2.0 or less, on a daily basis (CGS 2019, Imperial County Planning and Development Services 1993a, USGS 2011b, Attachment B). Based on the project-specific geotechnical report (Attachment B), the estimated peak ground acceleration at the site, associated with a Mw 6.7 earthquake, is 0.6g (percent of gravity).

Fluid injection and geothermal energy extraction in the North Brawley Geothermal Field, located within the Brawley Seismic Zone, have been linked to seismic hazards. After a few years of geothermal operations at the North Brawley Geothermal Field, located within the Brawley Seismic Zone, several magnitude 4 to 5 earthquakes occurred in 2012, followed by a long period of few earthquakes. Ground deformation was analyzed in the area, combining radar images, GPS, and leveling to reveal how the ground moved before, during, and after the 2012 events, with centimeter-scale accuracy (Materna et al. 2022). Another potential source of concern in geothermal fields is faults that slip without generating seismic waves. Silent slip, or fault creep, may play a role in controlling the location and duration of earthquake swarms. The processes behind silent or aseismic slip at geothermal fields are not well understood, largely because they are difficult to measure (Materna et al. 2022).

Liquefaction and Lateral Spreading

Liquefaction involves a sudden loss in strength of saturated, cohesionless soils that are subject to ground shaking during an earthquake and results in temporary transformation of the soil to behave more like a fluid mass. For liquefaction to occur, three conditions are required: (1) ground shaking of sufficient magnitude and duration; (2) a groundwater level at or above the level of susceptible soils during the ground shaking (i.e., generally at depths less than 40 feet); and (3) soils that are susceptible to liquefaction. Similarly, lateral spreading can result in ground cracking and may occur when a site is sloped or near a free-face and there is a sufficiently continuous liquefiable layer on which the overlying soils can move laterally. Ground settlement may occur during seismic shaking as a result of liquefaction.

The Brawley Campus has not been included in regional liquefaction analyses by the CGS (2023b). However, the unconsolidated sediments of the Salton Trough, especially in saturated areas such as irrigated lands, are subject to failure during earthquakes as a result of liquefaction (Imperial County Planning and Development Services 1993a). Liquefaction caused by the M7.2 El Mayor-Cucapah earthquake was widespread throughout the southern Imperial Valley. Ground motions of 0.3g to 0.6g (percent of gravity) were recorded in the majority of liquefaction areas (USGS 2011).

Groundwater was encountered in on-site borings at depths of 8 to 12 feet bgs. As previously discussed, borings drilled on-site encountered several approximate 2- to 3-foot thick beds of silty sand between depths of 18 to 28 feet bgs, as well as an additional approximate 10-foot thick layer of medium dense to dense, sandy materials at depths of 50 to 60 feet bgs. Geotechnical analyses indicated that these sandy layers are potentially liquefiable under high seismic loads. Liquefaction induced differential settlement and seismic compaction, which is the densification of loose to medium dense granular soils that are above groundwater, are likely to occur in the event of a large earthquake at the site. The estimated liquefaction-induced differential settlement is approximately 0.5 inch or less over a horizontal distance of 30 feet. Since the project site is relatively flat, the potential for substantial liquefaction-induced lateral displacement is low (Attachment B).

Subsidence

Subsidence is the permanent collapse of the pore space within a soil or rock and downward settling of the earth’s surface relative to its surrounding area. Subsidence can result from the extraction of water, oil, or geothermal resources, and the addition of water to the land surface—a condition called “hydrocompaction,” or peat loss. The compaction of subsurface sediment caused by the withdrawal or addition of fluids can cause subsidence. Land subsidence can disrupt surface drainage; reduce aquifer storage; cause earth fissures; damage buildings and structures; and damage wells, roads, and utility infrastructure.

According to the USGS Survey Areas of Land Subsidence in California map, there have been no recorded instances of subsidence in the Brawley Campus area associated with groundwater pumping, peat loss, or oil extraction (USGS 2023). However, natural subsidence has been occurring within the Salton Trough, averaging nearly two inches per year at the center of the Salton Sea, and decreasing to zero near the Mexican border. The subsidence is generally uniform, but local depressions have formed, such as the Mesquite Sink, located along Highway 86, between Imperial and Brawley.

In addition, subsidence in geothermal fields can occur when large fluid volume production leads to the decrease of pore pressure inside reservoirs. This decline disturbs the pressure stability and overburden pressure compresses the pores, resulting in a drop in the ground surface. The decrease in ground surface elevation can not only result in damage to buildings, pipelines, and canals, but may interrupt the balance in the nearby ecosystem (Sektiawan et al. 2016). Significant ground movement, in the form of ground subsidence and horizontal movement, may accompany geothermal development in the Imperial Valley. Regional and local survey nets are being monitored to detect and measure possible ground movement caused by future geothermal developments. Precise measurement of surface and subsurface changes are required to differentiate man-induced changes from natural processes (USGS 2013). Two geothermal facilities are located approximately 3 miles and 4 miles northwest of the Brawley Campus (Imperial County Planning and Development 2013).

Satellite radar interferometry (InSAR) was applied to detect surface deformation associated with geothermal development and concluded that distinct areas of subsidence are present in three geothermal fields in the Imperial Valley, including the Salton Sea, Heber, and East Mesa geothermal fields. In addition, ground uplift was observed at the Heber geothermal field (Eneva et al. 2012). These geothermal fields are located approximately 15 miles northwest, 19 miles south, and 18 miles southeast of the Brawley Campus, respectively (Imperial County Planning and Development 2013).

Land subsidence can be avoided by re-injecting all production water back into the aquifer it was withdrawn from so that pressure changes are minimized. Subsidence can be reduced through monitoring combined with aquifer management. Aquifers must be managed to balance groundwater recharge and groundwater discharge at both local and basin-wide scales. Management tools include 1) ensuring all water used for geothermal heat extraction is pumped back into the aquifer, 2) replacing water lost from the aquifer by increasing groundwater recharge to the basin-fill aquifer through conjunctive management of groundwater and surface water resources, and importation of water from other basins, 3) dispersing high-discharge wells to reduce localized land subsidence, and 4) reducing overall groundwater withdrawals in the basin (USGS 2012). In addition, well field programs covering production and injection plans in Imperial County are required by the Bureau of Land Management and CalGEM for each major geothermal project and are subject to review by CalGEM and the County (Imperial County Planning and Development Services 1993b).

Slope Stability

The topography of the Brawley Campus is relatively flat to gently sloping; therefore, there is no potential for slope instability such as landslides to occur.

5 Impact Analysis and Conclusions

5.1 Thresholds of Significance

The thresholds of significance used to evaluate the impacts of the proposed project related to geology and soils are based on Appendix G of the CEQA Guidelines (Cal. Code Regs., Title 14, Chptr. 3, sections 15000-15387.). A significant impact under CEQA would occur if the proposed project would:

a) Directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving:
    i. Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault?
    ii. Strong seismic ground shaking?
    iii. Seismic-related ground failure, including liquefaction?
    iv. Landslides?
b) Result in substantial soil erosion or the loss of topsoil?
c) Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction, or collapse?
d) Be located on expansive soil, as defined in the 2022 California Building Code, creating substantial direct or indirect risks to life or property?
e) Have soils incapable of adequately supporting the use of septic tanks or alternative waste water disposal systems where sewers are not available for the disposal of waste water?

5.2 Impact Analysis

a) ***Would the project directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving:***

i. ***Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault?***

Impacts related to rupture of a known earthquake fault were evaluated in Section 3.2, Geology/Soils, of the 2003 EIR, which concluded that the Brawley Campus is not within the limits of the Alquist-Priolo Special Studies Zones of the Imperial and Brawley faults. Accordingly, the 2003 EIR did not provide an impact conclusion regarding potential rupture of a known earthquake fault.

The proposed project involves construction and operation of a new campus building within the footprint of Building 107, as identified in the approved Campus Master Plan and analyzed in the previously certified 2003 EIR. As discussed above, the Holocene-active Imperial and Brawley faults are the closest faults to the Brawley Campus. As illustrated in Figure 3, the northern terminus of the Brawley Fault is approximately 2 miles south of the Brawley Campus and the northern terminus of the Imperial Fault is approximately 3.5 miles southwest of the campus. The Brawley Campus is not located in an Alquist-Priolo Earthquake Fault Zone associated with either of these faults. No new information or substantial changes in circumstances have occurred requiring new or additional analysis with regard to rupture of a known earthquake fault at the project site. As a result, surface fault rupture is not anticipated at the project site and the project would not directly or indirectly cause potential substantial adverse effects involving the rupture of a known earthquake fault. **No impact** would occur.

ii. ***Strong seismic ground shaking, or***

iii. ***Seismic-related ground failure, including liquefaction?***

Impacts related to seismic ground shaking, seismic related ground failure, and liquefaction were evaluated in Section 3.2, Geology/Soils, of the 2003 EIR, which concluded that although no geotechnical conditions have been identified to preclude development of the IVC Brawley projects as planned, geology/soils impacts would be significant because of the hazards from seismic activity if proper construction techniques are not observed at the detailed design and construction stages. Mitigation measures were provided that require SDSU to 1) avoid adverse discontinuities in strength between major structural elements, 2) prior to detailed site planning, conduct a subsurface geotechnical and soils study to ensure structural integrity, and 3) adhere to recommendations of the geotechnical and soils study in developing grading and construction plans (Mitigation Monitoring and Reporting Program [MMRP] page 11-1)[1]. With implementation of the mitigation measures, impacts were determined to be less than significant.

[1] **3.2 Geology and Soils Mitigation Measures** included on page 11-1 of the 2003 EIR: (1) Adverse discontinuities in strength between major structural elements shall be avoided. (2) Prior to detailed site planning, a subsurface geotechnical and soils study shall be

Updated information since completion of the 2003 EIR related to seismicity, including liquefaction and fluid injection, are summarized below, as well as in Section 4.1, Existing Conditions. The Imperial Valley area is subjected to frequent seismic events, with related concerns of ground shaking and liquefaction. The most noteworthy of the numerous faults traversing the Salton Trough is the Holocene-active Coachella section of the San Andreas Fault. As described above in Section 4.1, Existing Conditions, two other major northwest-trending Holocene-active fault zones bounding the Salton Trough include the San Jacinto Fault on the northwest and the Elsinore Fault on the southwest (Figure 3). In addition, the Holocene-active Imperial and Brawley faults are located south of the Brawley Campus and the Brawley Seismic Zone is located approximately 3 miles west of the Brawley Campus. Fluid injection and geothermal energy extraction in the North Brawley Geothermal Field, located within the Brawley Seismic Zone, have been linked to seismic hazards.

The unconsolidated sediments of the Salton Trough, especially in saturated areas such as irrigated lands, are subject to failure during earthquakes as a result of liquefaction. As a result, the proposed project would potentially be subject to liquefaction in the event of a large earthquake. Seismic induced ground shaking can also result in differential settlement and seismic densification because of variations in soil composition, thickness, and initial density.

Since certification of the 2003 EIR, the CEQA significance criteria have been revised (per Appendix G of the 2022 CEQA Statute and Guidelines). Seismic impacts on any given project are no longer considered potentially significant. Rather, impacts would only be considered significant in the event the project directly or indirectly caused seismic impacts to occur. Because construction and operation of the proposed building would not induce seismicity, no impacts would occur.

Regardless, the following is an updated discussion of protocol that would be followed with respect to seismic engineering of the proposed building. As required by the 2022 California Building Code (CBC), the proposed Brawley Campus building and associated infrastructure improvements would be constructed in accordance with the recommendations of the project-specific geotechnical report (Attachment B), which includes recommendations for remedial grading and foundation design to address strong seismic ground shaking, liquefaction, differential settlement, and seismic densification. Accordingly, while referred to as “recommendations” in the referenced report, each recommendation is, in fact, required by law to be implemented. More specifically, the geotechnical report recommendations require the use of thickened and heavily reinforced conventional building foundations or post-tensioned slabs to reduce the potential for distress to the proposed building associated with post-liquefaction settlement. The geotechnical recommendations are consistent with *CGS Note 48, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings* (CGS 2022). Design and construction to these standards would provide an acceptable level of earthquake safety for students, employees, and the public who occupy the building, to the extent feasible.

In addition, the project would be designed in accordance with the CSU Seismic Requirements (CSU 2020), which include specific requirements for the construction of new buildings, to ensure that all

conducted to determine the shrink-swell potential and to develop design specific measures to ensure structural integrity. Grading and construction plans shall conform to recommendations of the study.

CSU buildings provide an acceptable level of earthquake safety for students, employees, and the public, per the CBC. The CSU Seismic Policy applies to all structures within the bounds of a CSU campus master plan. These seismic requirements set forth procedures to follow in order to manage current construction programs and limit future seismic risk to acceptable levels. CSU has established campus-specific seismic ground motions parameters that supersede CBC values and implement a conservative evaluation on CBC Structural Risk Category assignments.

The CSU Seismic Requirements require that all major capital building projects, such as the proposed project, be peer reviewed by the Division of State Architect (DSA), prior to and during construction. The DSA provides design and construction oversight for K–12 schools, community colleges, and various other state-owned and leased facilities. The DSA also develops accessibility, structural safety, fire and life safety, and historical building codes and standards utilized in various public and private buildings throughout California. This review process starts at project inception and continues until construction completion. Peer review concurrence letters are typically issued at completion of the Schematic and Construction Documents Phases and during the course of construction on deferred submittals that have a seismic component. Resolution of outstanding Seismic Review Board peer review comments is required before start of construction, and resolution of Seismic Review Board construction phase submittals is required prior to occupancy. In addition, the project would be submitted to the CSU Architecture and Engineering, Building Code Plan Check Review process. All approved plans for construction would include a stamp that verifies the design would be completed in compliance with appropriate CSU Seismic Requirements. The stamp would also indicate that the project has been reviewed consistent with Chapter 16 of the CBC and the State Earthquake Protection Law.

Furthermore, the CGS serves as an advisor under contract with the DSA to review engineering geology and seismology reports for compliance with state geologic hazard regulations. For all facility construction, SDSU will be required to send all engineering, geotechnical, and soils reports normally required to comply with the CBC to the CGS to ensure such reports also comply with applicable geologic hazard regulations (i.e., the Field Act and the Seismic Hazards Mapping Act). The CGS has outlined the required scope of geology, seismology, and geologic hazards evaluations under California Code of Regulations, Title 24. Among other things, the reports must be prepared by appropriately licensed professionals and must include adequate site characterization, estimates of earthquake ground motions, assessment of liquefaction/ settlement potential, slope stability analysis, identification of adverse soil conditions (e.g., expansive or corrosive soils), and mitigation recommendations for all identified issues. Final DSA approval of the proposed building will not occur unless DSA receives the final acceptance letter from CGS.

The proposed building and infrastructure improvements would be constructed under the supervision of a California Geotechnical Engineer and/or California Certified Engineering Geologist. In addition, construction and operation of proposed project facilities would not increase the potential for earthquakes or seismically induced ground failure to occur. As a result, the project would not directly or indirectly cause potential substantial adverse effects involving strong seismic ground shaking or seismic-related ground failure, including liquefaction. **No impacts** would occur.

iv. Landslides?

The Initial Study (IS) prepared for the 2003 EIR determined that no impact would occur with regard to landslides. The topography of the Brawley Campus and surrounding area is relatively flat to gently sloping. With implementation of the required recommendations provided in the project-specific geotechnical report, slope instability would not adversely impact the proposed development (Attachment B). In addition, because the topography of the site is relatively flat, grading and construction would not cause slope instability to occur. As a result, the project would not directly or indirectly cause potential substantial adverse effects involving landslides. **No impacts** would occur.

b) Would the project result in substantial soil erosion or the loss of topsoil?

The 2003 EIR and IS prepared for the 2003 EIR did not specifically address soil erosion and loss of topsoil. Therefore, a discussion regarding the proposed project's potential to result in substantial soil erosion or the loss of topsoil is provided below.

The proposed project site is approximately 1.5-acres in size and the construction staging areas would occupy approximately 52,000 sf in the area of campus located southeast of the site and north of SR 78. The project would involve site preparation, grading, and excavation associated with project construction. Excavation depths are anticipated to be 2 to 5 feet, followed by soil backfill and compaction. Project grading and construction would temporarily expose onsite soils to wind and water erosion, which in turn could result in sedimentation of downstream drainages. However, because project construction would involve ground disturbance in excess of 1 acre, grading and construction would be completed in accordance with the requirements outlined in the National Pollutant Discharge Elimination System (NPDES) Construction Stormwater General Permit (2009-0009-DWQ), effective July 1, 2010 (NPDES Construction General Permit), which includes the development of a Stormwater Pollution Prevention Plan (SWPPP). The SWPPP would identify potential water quality pollutants (including erosion-induced sedimentation), identify minimum best management practices (BMPs) to prevent offsite sedimentation, and develop a construction site monitoring plan for the project. After construction, the project site would be developed with impermeable surfaces and 21,760 sf of on-site landscaping, thus eliminating the potential for soil erosion. As a result, the Project would not result in substantial soil erosion or the loss of topsoil and impacts would be **less than significant.**

c) Would the project be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction, or collapse?

The IS completed for the 2003 EIR concluded that no impacts would occur with respect to potentially unstable geologic units, including landslides, lateral spreading, subsidence, liquefaction, and collapse. Since certification of the 2003 EIR, the CEQA significance criteria have been revised (per Appendix G of the 2022 CEQA Statute and Guidelines). Geologic hazard impacts on any given project are no longer considered potentially significant. Rather, impacts would only be considered significant in the event the project directly or indirectly caused geologic hazard impacts to occur. Therefore, the following is an updated discussion of potential impacts related to geologic hazards, as well as an updated discussion of protocol that would be followed with respect to geotechnical engineering of the proposed building. In addition, updated information since completion of the 2003 EIR related to liquefaction and subsidence are summarized below. New information pertaining to liquefaction and subsidence is also presented in Section 4.1, Existing Conditions.

As described for Thresholds a-ii and a-iii, although the project would be susceptible to strong seismically induced ground shaking and liquefaction, project design and construction would be completed in compliance with the 2022 CBC and CGS Note 48, pertaining to seismic design for California public schools. In compliance with the CBC, project design and construction would be completed in accordance with the recommendations of the project-specific geotechnical report (Attachment B). The proposed building would also be subject to review and plan approval by the DSA and the CSU Architecture and Engineering, Building Code Plan Check Review process, prior to and during construction. Compliance with the CBC, DSA review and approval, and CSU Architecture and Engineering review would help to offset potential risks to structures and people associated with liquefaction and collapsible soils. In addition, constructing the proposed building within a liquefaction-prone area would not, in and of itself, increase liquefaction risks to surrounding uses. Although the project site is potentially susceptible to liquefaction, no slopes are present on the site, thus eliminating the potential for lateral spreading to occur (Attachment B). As described for Threshold a-iv, the project would not be susceptible to landslides.

Natural subsidence has been occurring within the Salton Trough, averaging nearly two inches per year at the center of the Salton Sea, and decreasing to zero near the Mexican border. This natural subsidence is relatively uniform over large areas. In addition, subsidence in geothermal fields can result in damage to buildings and related infrastructure. Two geothermal facilities are located approximately 3 miles and 4 miles northwest of the Brawley Campus, respectively. As described under Section 4.1, Existing Conditions, satellite radar interferometry (InSAR) was applied to detect surface deformation associated with geothermal development and concluded that distinct areas of subsidence are present in three geothermal fields in the Imperial Valley, including the Salton Sea, Heber, and East Mesa geothermal fields. In addition, ground uplift was observed at the Heber geothermal field. These geothermal fields are located approximately 15 miles northwest, 19 miles south, and 18 miles southeast of the Brawley Campus, respectively. Therefore, subsidence as a result of geothermal activity does not appear to be occurring at the project site. Well field programs covering production and injection plans in Imperial County are required by the Bureau of Land Management and CalGEM for each major geothermal project and are subject to review by CalGEM and the County, thus minimizing the potential for subsidence to occur. In addition, construction and operation of the proposed Brawley Campus building would not result in substantial adverse impacts such that collapse would occur. As a result, the project would not be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction, or collapse. Impacts would be **less than significant.**

d) Would the project be located on expansive soil, as defined in the 2022 California Building Code, creating substantial direct or indirect risks to life or property?

Impacts related to expansive soils were evaluated in Section 3.2, Geology/Soils, of the 2003 EIR, which concluded that although no geotechnical conditions have been identified to preclude development of the IVC Brawley projects as planned, geology/soils impacts are significant because of the hazards from expansive soils if proper construction techniques are not observed at the detailed design and construction stages. Mitigation measures were provided that would require SDSU to 1) prior to detailed site planning, conduct a subsurface geotechnical and soils study to determine the shrink-swell potential, and 2) adhere to recommendations of the geotechnical and soils study in developing grading and construction plans (MMRP page 11-1)[2]. With implementation of the mitigation measures, impacts were determined to be less than significant.

Borings drilled on-site indicated the site is underlain by 1 to 2 feet of fill material, consisting of fat clay. Laboratory testing of these surficial clays indicate these soils have a high to very high expansion potential. Swelling and shrinking soils can result in differential movement of structures including floor slabs and foundations, and project site work including hardscape, utilities, and sidewalks. Project design and construction would occur in compliance with recommendations of the project-specific geotechnical report (Attachment B) and the provisions of the 2022 CBC, which requires that grading, structural design, and construction be completed such that potentially expansive soils would not adversely affect foundations, piping, and related infrastructure. More specifically, based on the geotechnical report required recommendations, thickened foundations and slabs, underlain by at least 5 feet of imported granular non-expansive, compacted fill will be utilized to reduce the potential for future distress to the building associated with soil expansion. Alternatively, a post-tensioned slab-on-grade would be used to support the proposed building. Project design would also be completed in accordance with the DSA and CSU Architecture and Engineering review process. As a result, construction of the project on potentially expansive soils would not create substantial direct or indirect risks to life or property. Impacts would be **less than significant**, and no additional mitigation is required.

e) Would the project have soils incapable of adequately supporting the use of septic tanks or alternative waste water disposal systems where sewers are not available for the disposal of waste water?

The IS completed for the 2003 EIR concluded that no impacts would occur with respect to the use of septic tanks or alternative waste water disposal systems. No new information is available regarding this environmental criteria. The proposed building would be connected to existing sewer infrastructure operated by the city of Brawley. As a result, septic tanks or alternative wastewater disposal systems would not be used in association with the project. **No impacts** would occur.

[2] **3.2 Geology and Soils Mitigation Measures** included on page 11-1 of the 2003 EIR: (1) Adverse discontinuities in strength between major structural elements shall be avoided. (2) Prior to detailed site planning, a subsurface geotechnical and soils study shall be conducted to determine the shrink-swell potential and to develop design specific measures to ensure structural integrity. Grading and construction plans shall conform to recommendations of the study.

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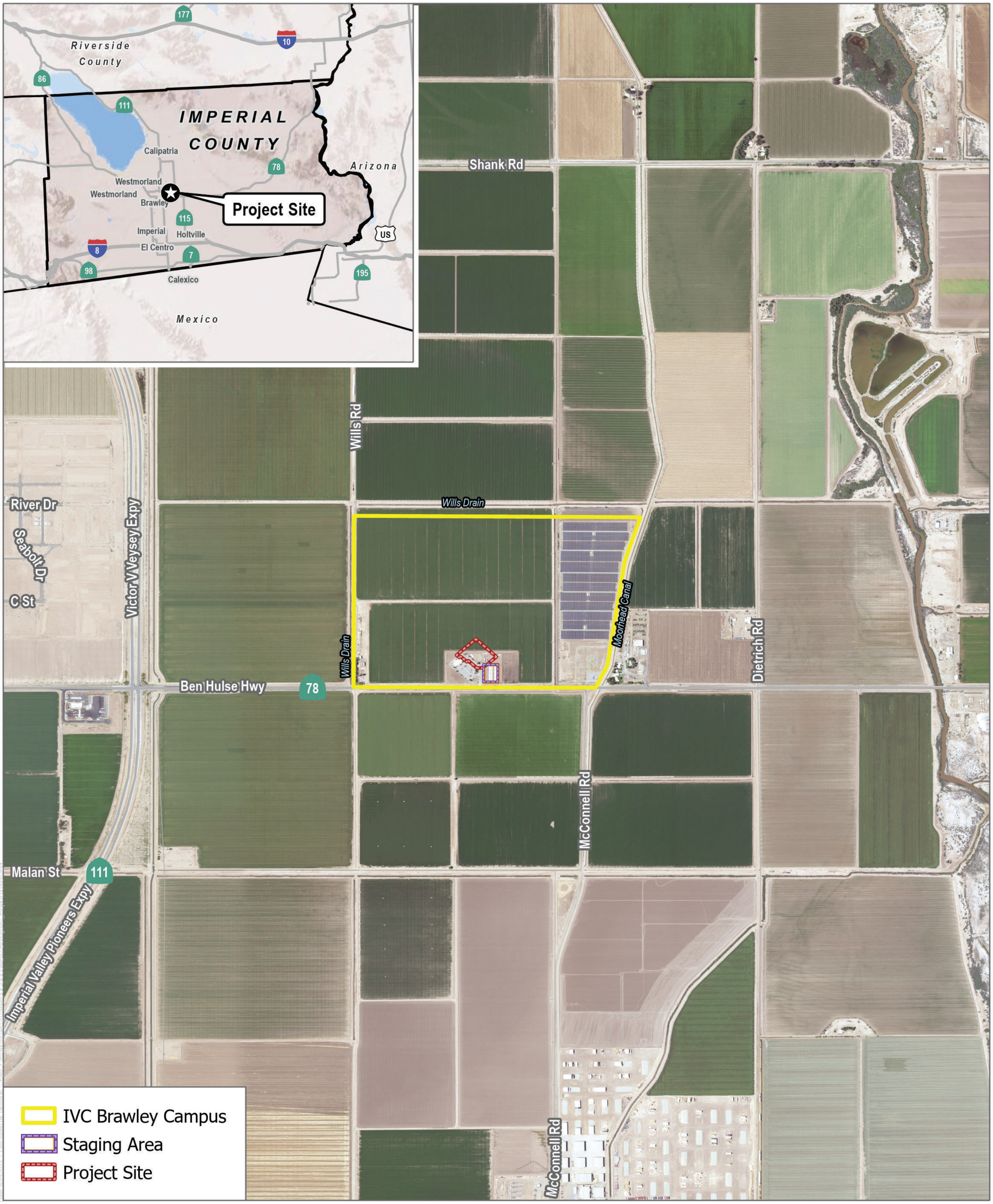
Attachment A
Figures



SOURCE: NAIP 2020, Open Streets Map 2019

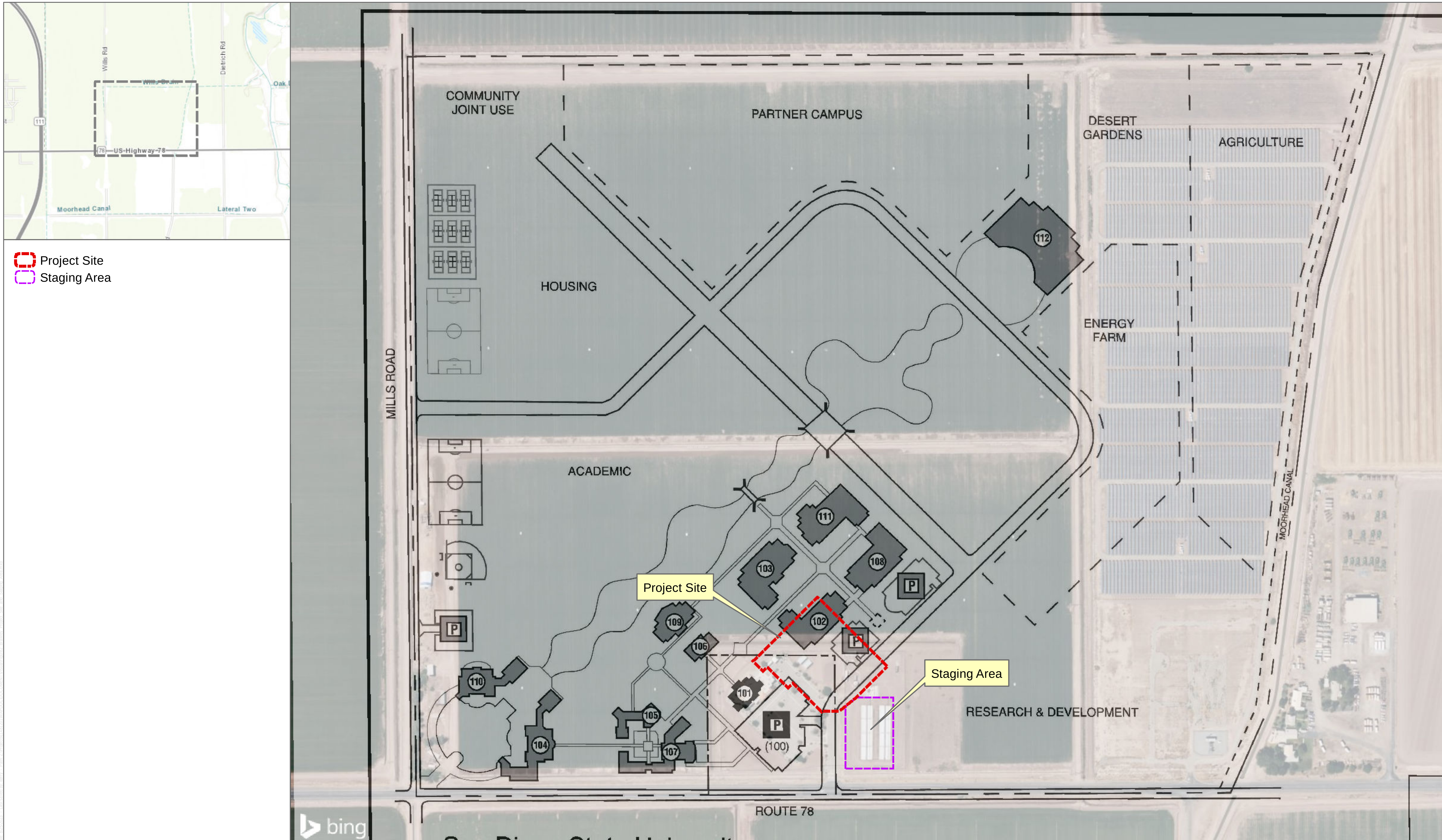
DUDEK

0 1,000 2,000 Feet

FIGURE 1

Regional/Campus Location

SDSU Brawley Sciences Building Project



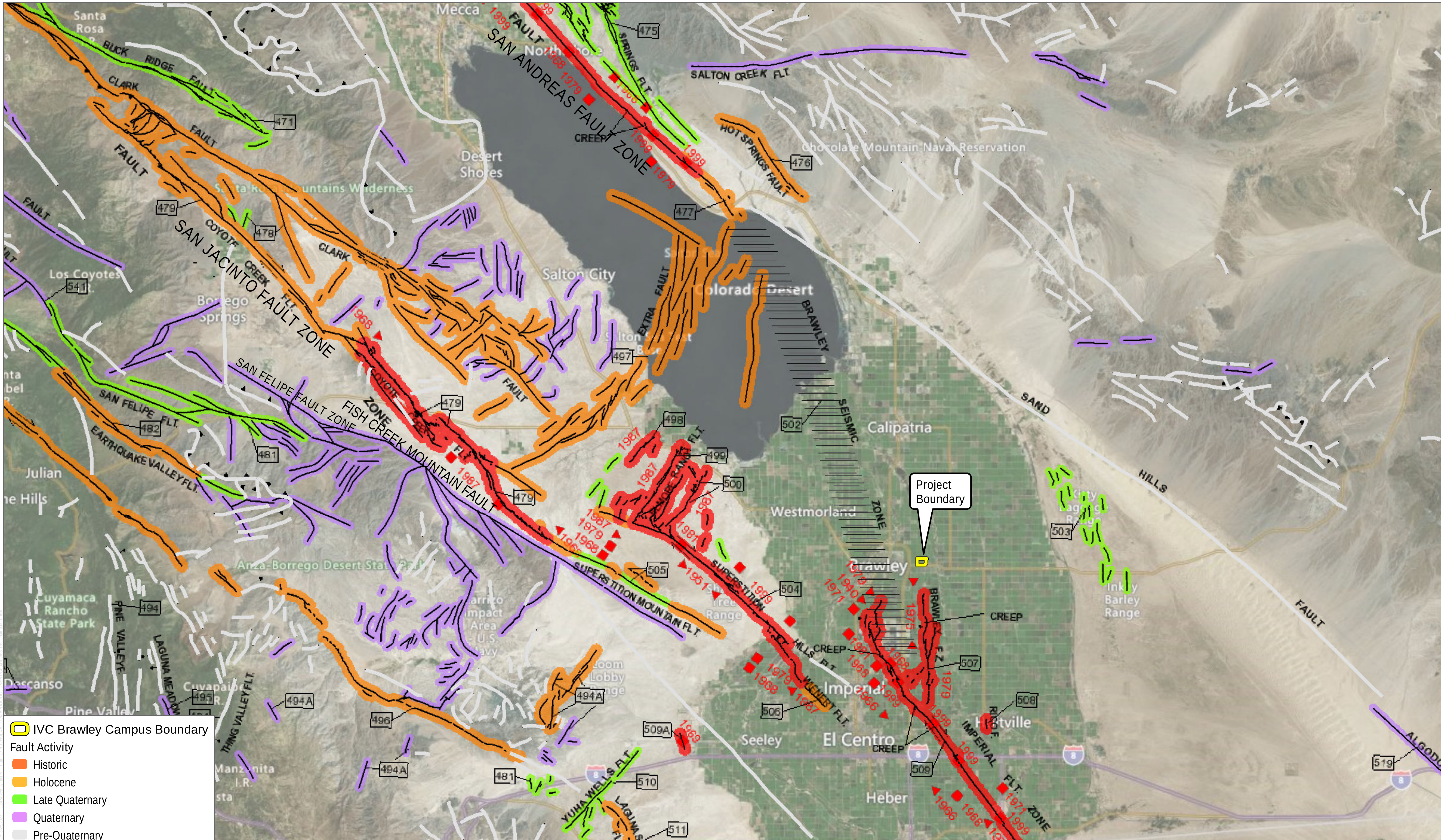
SOURCE: AERIAL-BING MAPPING SERVICE 2022; CAMPUS MASTER PLAN 2003

DUDEK 0 175 350 Feet

FIGURE 2

SDSU Brawley Project Site and Staging Area

SDSU Brawley Sciences Building Project



SOURCE: USGS 2022; CGS 2022; County of Imperial; Bing Maps

DUDEK

0 5 10 Miles

FIGURE 3

Regional Faulting

SDSU Brawley Sciences Building Project

Attachment B

Geotechnical Report

REPORT OF GEOTECHNICAL INVESTIGATION
SDSU BRAWLEY STEM FACILITY
SAN DIEGO STATE UNIVERSITY
BRAWLEY, CALIFORNIA

Prepared for

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Facilities Planning, Design & Construction
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Prepared by

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Project No. SD725A
March 27, 2023

San Diego State University
Facilities Planning, Design & Construction
5500 Campanile Drive
San Diego, California 92182-1624

Project No. SD725A
March 27, 2023

Attention: Ms. Amanda Scheidlinger

SUBJECT: REPORT OF GEOTECHNICAL INVESTIGATION
SDSU Brawley STEM Facility
San Diego State University
Brawley, California

DRAFT

Ms. Scheidlinger:

Group Delta Consultants, Inc. (Group Delta) are pleased to submit this report of geotechnical investigation for the planned Science, Technology, Engineering, and Mathematics (STEM) Facility at the San Diego State University campus in Brawley, California. This report summarizes our conclusions regarding the geologic site constraints, and provides geotechnical recommendations for remedial grading, foundation, slab, and pavement section design.

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

GROUP DELTA CONSULTANTS

Christopher K. Vonk, G.E. 3216
Senior Geotechnical Engineer

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Principal Engineering Geologist

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Mr. Justin Dorsey, OCMI (jdorsey@ocmi.com)

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APPENDICES

1.0 INTRODUCTION

The following report provides geotechnical recommendations for the proposed Science, Technology, Engineering, and Mathematics (STEM) Facility to the San Diego State University campus in Brawley, California. The general location of the site is shown in Figure 1A, Site Location. The campus location is shown in more detail in Figure 1B, Site Vicinity. The approximate locations of the subsurface explorations that we completed at the site are shown in Figure 2, Exploration Locations.

1.1 Scope of Services

Our geotechnical services were provided in general accordance with the provisions of the referenced proposal (Group Delta, 2023). The purpose of this work was to characterize the geotechnical constraints to site development, and to provide recommendations for grading and design of the new foundations, slabs, utilities, retaining walls, drainage improvements and pavements. The recommendations provided herein are based on subsurface investigation, the findings from laboratory tests, our engineering analyses, and our previous experience at the site and with similar geologic conditions in the site vicinity. In summary, we provided the following services for this project.

- A visual reconnaissance of the surface characteristics of the site and surrounding areas, and a review of the relevant reports listed in the *References* section of this report.
- A subsurface exploration of the site including three geotechnical borings and five Cone Penetration Test (CPT) soundings along with shear wave velocity measurements. The exploration locations are shown on Figure 2. The Boring Records and CPT data are provided in the figures of Appendix A.
- Laboratory tests were conducted on soil samples collected from the geotechnical borings. Laboratory tests included moisture content, dry density, sieve analysis, Atterberg Limits, Expansion Index, soil corrosivity, unconfined compressive strength, and consolidation. The laboratory test results are summarized in Appendix B.
- Engineering analysis of the field and laboratory data to develop geotechnical recommendations for site preparation, remedial earthwork, foundation, pavement and retaining wall design, soil reactivity, site drainage, and moisture protection.
- Preparation of this report summarizing our findings, conclusions and providing geotechnical recommendations for the proposed STEM facility.

1.2 Site Description

The Brawley Campus of San Diego State University (SDSU) is located at 560 State Route 78 (SR-78) in Brawley, California. The campus in situated within the Imperial Valley about 15 miles south of the Salton Sea, as shown on Figure 1A, Site Location. The campus is located immediately north of SR-78, east of Willis Road and west of McConnell Road, as shown on Figure 1B, Site Vicinity. The campus contains an existing single-story building surrounded by asphalt concrete paved parking areas, landscape areas, and shade structures. The location of the proposed STEM facility is predominantly dirt surfaced and extends into a portion of the site that is currently used for agriculture purposes. The approximate site limits are shown on Figure 2, Exploration Locations.

The SDSU Brawley campus is relatively flat-lying and located more than 130-feet below mean sea level. The campus is surrounded by active agricultural fields. These fields are irrigated by a complex system of canals and drains that are maintained by the Imperial Irrigation District (IID), such as the Moorhead Canal to the east of the site shown in Figure 1B. The crops are drained through a series of shallow subdrains which carry excess irrigation water laterally into open drainage channels such as the Wills Drain One to the east and Lateral One to the southwest (see Figure 1B). One of these open drainage channels runs east-west through the center of the site.

1.3 Proposed Development

We understand that the proposed STEM facility will consist of a two-story structure constructed within the boundaries of the site limits shown on Figure 2, Exploration Locations. The building will likely consist of a tilt-up concrete or steel-framed structure supported on conventional shallow reinforced concrete foundations or a post-tensioned slab. Other new site improvements may include new sidewalks and pavement areas, as well as various new landscape areas, subsurface utilities, and retaining walls.

2.0 FIELD AND LABORATORY INVESTIGATION

The following sections describe the current and prior field and laboratory investigations performed near the proposed development.

2.1 Current Investigation

Our current field investigation included performing three geotechnical borings (B-1 through B-3) and five Cone Penetration Test (CPT) soundings (CPT-6 through CPT-10) on February 17, 2023. The maximum depth explored was about 85 feet below grade. The CPTs were advanced using a 30-ton truck mounted CPT rig, and the borings were completed using truck-mounted drill rig using hollow-stem auger and rotary wash methods. Bulk, Shelby tube, disturbed Standard Penetration Tests (SPT), and less disturbed modified California samples were collected from the borings and were subsequently transported to our laboratory for further visual evaluation and laboratory testing. The exploration locations are shown on Figure 3. The Boring Records and CPT data are provided in Appendix A.

Shear wave velocity measurements were collected at the location of sounding CPT-10 at 5-foot depth intervals to the maximum depth explored. The interval shear wave velocity data is presented in Appendix A and indicates an average shear wave velocity (Vs_{30}) of 190 m/s (or 625 ft/s) in the upper 100 feet.

The laboratory testing program included gradation and hydrometer analyses and Atterberg Limits to aid in material classification according to the Unified Soil Classification System (USCS). Tests were also conducted to help evaluate the soil expansion and corrosivity potential. Unconfined compressive strength and consolidation tests were also performed to evaluate the undrained shear strength and compressibility parameters of the underlying clayey materials. The laboratory test results are shown in Appendix B.

2.1 Prior Investigation

Group Delta previously performed a geotechnical investigation for an addition located on the east side of the existing structure at the site (Group Delta, 2022). The subsurface exploration program included five CPT soundings (CPT-1 through CPT-5) on March 22nd, 2022. The maximum depth explored was about 88½ feet below grade. Shear wave velocity measurements were collected at the location of sounding CPT-5 at 5-foot depth intervals. The interval shear wave velocity data is presented in Appendix A and indicates an average shear wave velocity (Vs30) of 185 m/s (or 610 ft/s) in the upper 100 feet. Bulk soil samples were collected at each CPT sounding location for laboratory testing and geotechnical analysis.

The laboratory testing program included gradation and hydrometer analyses and Atterberg Limits to aid in material classification according to the Unified Soil Classification System (USCS). Tests were also conducted to help evaluate the soil expansion and corrosivity potential. The maximum density and optimum moisture content of a bulk soil sample were determined and used to help remold a fill sample for shear testing.

The CPT locations are shown on Figure 2, Exploration Locations. The CPT data and laboratory testing data are provided in Appendix C, Data from Prior Geotechnical Study. Salient findings from this prior investigation are incorporated in the following sections of this report.

3.0 GEOLOGY AND SUBSURFACE CONDITIONS

The site is located within the north-central portion of the Salton Trough, a topographic and structural depression bound to the north by the Coachella Valley and to the south by the Gulf of California. The Salton Trough is a region of transition from the extensional tectonics of the East Pacific Rise to the transform tectonic environment of the San Andreas system. Late Cenozoic extension of the Gulf of California formed this deep topographic and structural depression.

The Salton Trough is an actively growing rift valley in which sedimentation has almost kept pace with tectonism (Elders, 1979). As rifting continued, the Colorado River delta filled the trough, and conditions gradually changed from marine, to deltaic, to subaerial river and lake deposits. Today, the Mesozoic-age crystalline basement rocks of the trough are covered by about 15,000 feet of Cenozoic marine and nonmarine sedimentary deposits.

The site is located in an area that has been covered by lakes during the Quaternary time. The most recent of the lakes that formed in the Salton Trough was known as Lake Cahuilla, which was formed by flooding of the Colorado River and existed until approximately 300 years ago (Elders, 1979). The old shoreline of Lake Cahuilla can be traced along the Santa Rosa Mountains north of the site, and averages about 40 feet above mean sea level. The site is underlain at depth by hundreds of feet of lacustrine (lake) deposits, overlain by shallow fill.

The general geology in the site vicinity is shown on Figure 3, Geology. A geotechnical cross section of the site is provided in Figure 4. Logs interpreting the subsurface conditions we encountered in the geotechnical borings and CPT soundings during the current investigation are provided in Appendix A, and logs from prior investigation are provided in Appendix C. The geologic materials encountered at the site are described below.

3.1 Lacustrine Deposits

The entire site is underlain by deep lacustrine deposits associated with the ancestral Lake Cahuilla. The lacustrine sediments are estimated to be well over 100 feet thick (Kovach et al., 1962). The lake sediments are typically fine grained, and generally consist of interbedded clays (Unified Soil Classification Symbol CL and CH), with thin lenses of silt (ML) and occasional beds of silty sand (SM). The granular soils within the lake deposits are typically medium dense in consistency. The clays range from low to high plasticity, and range in consistency from soft to stiff.

Laboratory tests indicate that the surficial clays have a high to very high expansion potential and severe soluble sulfate and chloride contents. The estimated undrained shear strength (Su) for the predominately clayey lacustrine deposits typically ranges from about 0.75 to 2 kips per square foot (ksf). The fine-grained lacustrine deposits would therefore be considered medium stiff to stiff in consistency. Shear wave velocity measurements at the location of sounding CPT-10 indicated an average shear wave velocity of about 625 ft/s (or 190 m/s).

Several roughly 2- to 3-foot thick beds of silty sand (SM) were encountered in the explorations at depths ranging from between 18 to 28 feet below existing grade. An approximately 10-foot thick layer of sandy materials was also interpreted in CPT-10 from depths between approximately 50 to 60 feet. These layers were also encountered in previous explorations performed to the west (Group Delta, 2022). The CPT tip resistance in these sandy layers generally exceeded 120 tons per square foot (tsf), and SPT-corrected blow counts generally ranged between 20 and 35, which is indicative of a medium dense to dense material. Our analyses indicate that these zones of material are potentially liquefiable under high seismic demand, as described in the *Earthquake Induced Ground*

Failure section of this report. The location and extent of these continuous, potentially liquefiable, granular lacustrine deposits are also shown in the Geotechnical Cross Section, Figure 4.

3.2 Fill

Approximately one to two feet of fill and/or disturbed agricultural soil were encountered in each of our explorations. Similarly, a few feet of fill were encountered in our prior explorations performed to the west (Group Delta, 2022). The surficial materials generally consist of fat clay (CH) with little or no sand. The fill soils have a high to very high potential for expansion and are considered severely corrosive. At the location of CPT-1 within the parking lot area, the existing pavement section consisted of 3 inches of asphalt concrete over 12 inches of aggregate base. Note that the existing pavements are cracked due to the highly expansive nature of the subgrade.

3.3 Groundwater

Groundwater was measured at a depth of approximately 10 feet below existing site grades. Pore pressure dissipation tests were also conducted within the CPT soundings. The equilibrium pore water pressure measured by these tests was used to estimate the groundwater elevations. These dissipation analyses indicate that the groundwater levels at the site vary from a depth of about 8 to 12 feet below existing site grades. Note that groundwater levels do fluctuate over time due to changes in groundwater extraction, irrigation, or antecedent rainfall. It should also be noted that changes in rainfall, irrigation practices (particularly related to agricultural areas around and within the site that are flood irrigated), or site drainage may produce seepage or locally perched groundwater conditions at any depth within the fill or lacustrine deposits underlying the site.

4.0 GEOLOGIC HAZARDS

The site is located within the Salton Trough of the Colorado Desert geomorphic province, which is one of the most seismically active areas in California, as shown on Figure 5A, Fault Locations. The Salton Trough is the zone of transition between the ocean floor spreading regime in the Gulf of California and the right-lateral, strike-slip regime of the San Andreas system. Geologic hazards at the site are related to the potential for strong ground shaking due to an earthquake on one of several nearby active faults, as well as the potential for associated soil liquefaction and dynamic settlement. Each of the potential geologic hazards is described in more detail below.

4.1 Strong Ground Motion

The site is in a seismically active area. There are several active faults in the site vicinity that have produced moderate to large earthquakes within the past 100 years. The Imperial Fault Zone ruptured with a magnitude 6.9 earthquake in 1940, and again with a magnitude 6.4 earthquake in 1979 (USGS, 1982). The trace of the ground rupture from the 1940 earthquake was located about 5 miles east of the site (see Figure 3 and Figure 5B for the approximate 1940 ground rupture location). Additionally, there are several other known active faults close to the site, including the Superstition Hills and Superstition Mountain fault zones to the northwest, and the Laguna Salada and Cerro Prieto fault zones to the south (see Figures 3 and 5A). The Superstition Hills fault experienced a magnitude 6.7 earthquake in 1987 (Magistrale et al., 1989). In 2010, a magnitude 7.2 earthquake occurred on the Laguna Salada fault zone south of the international border (Gonzalez-Ortega et al., 2014). These earthquakes caused damage to structures throughout Imperial Valley, including soil liquefaction, settlement, and surficial slumps along the Imperial Irrigation District canal and drains (USGS, 1982; Gonzalez-Ortega et al., 2014; Holzer et al., 1989).

The new building will likely be subjected to numerous small to moderate magnitude earthquakes, as well as occasional larger magnitude earthquakes from nearby active faults over its expected life span. The resulting strong ground motions associated with this hazard may be managed by structural design per the governing edition of the California Building Code and California State University (CSU) Seismic Requirements (CSU, 2020). Seismic design parameters are provided in the *Recommendations* section of this report.

4.2 Ground Rupture

Ground rupture results from movement on an active fault reaching the ground surface. The site is not located within an Alquist-Priolo Active Fault Zone and no known active faults are present in the immediate site vicinity, as shown on Figure 5B, Alquist-Priolo Special Studies Zones. Potential for ground rupture from active faulting should therefore be considered low.

4.3 Earthquake-Induced Ground Failure

Potentially liquefiable soils underlie the site. Figure 3, Geology, illustrates that the site is mapped in an area underlain by Quaternary Lake Deposits (i.e., Lacustrine Deposits) that are known to be potentially susceptible to liquefaction and its secondary effects (e.g., earthquake-induced ground failure).

4.3.1 Background

Liquefaction is the sudden loss of soil shear strength within saturated, loose to medium dense, sands and non-plastic silts. Liquefaction is caused by the build-up of pore water pressure during strong ground shaking from an earthquake. Secondary effects of liquefaction are sand boils, settlement and instabilities within sloping ground that occur as lateral spreading, seismic deformation and flow sliding. Lateral spreading is the horizontal deformation of gently sloping ground (slope less than 6 percent), and seismic deformation is the horizontal movement of more steeply sloping ground, both of which can occur during strong ground shaking. Flow sliding is an overall instability of more steeply sloping ground that can occur following or near the end of strong ground shaking, depending on its duration. Associated with liquefaction is seismic compaction, which is the densification of loose to medium dense granular soils that are above groundwater. Of these, liquefaction-induced settlement and seismic compaction are considered more likely to occur given the site surface and subsurface conditions, as discussed below.

4.3.2 Vertical Settlement Analyses

4.3.2.1 Volumetric Settlements

The computer program CLiq (GeoLogismiki, 2019) was used to perform liquefaction triggering calculations using several CPT-based methods, including those recommended by the NCEER Workshops (Youd and Idriss, 2001) and Boulanger and Idriss (2014). CLiq also calculates the estimated free-field volumetric settlement (below groundwater) and seismic compaction (above groundwater). The analyses adopted the following input parameters:

Peak Ground Acceleration (PGA_M): 0.6g
Earthquake Magnitude (Mw): 6.7
Groundwater Level:......................... 10 feet Below Ground Surface

The PGA_M was evaluated using the maximum of the: 1) most recent version of the CSU Seismic Requirements (CSU, 2020), and; 2) maximum considered earthquake geometric mean (MCE_G) peak ground acceleration adjusted for Site Class effects (PGA_M) obtained from the ASCE 7 Hazard Tool (ASCE, 2023) in accordance with ASCE 7-16 (ASCE, 2017) and the 2022 California Building Code (CBSC, 2022). The controlling magnitude used in the liquefaction evaluation was selected by reviewing deaggregation results obtained from the USGS Unified Hazard Tool (USGS, 2023). A design groundwater level of 10 feet below ground surface was adopted based on our groundwater measurements and our interpretation of the soil saturation of in-situ soil samples and CPT pore pressure dissipation test data.

The analyses were performed using data collected from the recent CPTs performed at the site (CPT-6 through CPT-10). The correlated CPT parameters were compared to the results of our field and laboratory testing collected from borings B-1 through B-3. The CPT Soil Behavior Type (SBT) correlated from the CPT data was adjusted to best fit the observations, classifications, and material properties of the soils within the borings.

In accordance with Special Publication 117A (CGS, 2008) and general geotechnical engineering practices, a factor of safety against liquefaction of 1.3 was adopted in the analyses, and the liquefaction analyses was limited to a depth of 60 feet to incorporate the potentially liquefiable layer that extends down to 60 feet.

The liquefaction settlement analyses include depth weighting proposed by Cetin et al. (2009), which consists of a simple linear weighting factor that weights the volumetric strain with depth. This reduces the impact of volumetric strains at large depths. The weighting starts at one at the ground surface and reduces to zero at the weighting limit depth, selected to be the depth of analysis for this project (i.e., 60 feet).

4.3.3 Vertical Settlement Summary

Based on the results of the triggering analyses there are several potentially liquefiable zones within the subsurface profile. In general, the potentially liquefiable soils consist of occasional thin beds that are generally less than 2-foot-thick each, but some up to 4-feet thick locally. The estimated liquefaction-induced volumetric settlement is approximately 1-inch or less at each exploration location. The estimated liquefaction-induced differential settlement is approximately 0.5-inch or less over a horizontal distance of 30 feet.

4.3.4 Instability of Sloping Ground

Since the site is essentially level and the buildings are not located immediately adjacent to sloping ground, the potential for significant liquefaction-induced lateral displacement should be low.

4.4 Landslides

Evidence of ancient landslides or slope instabilities was not observed during our literature review or site reconnaissance and the site is essentially level. Provided that our geotechnical recommendations are properly implemented during construction, it is our opinion that slope instability does not adversely impact the proposed development.

4.5 Tsunamis, Seiches, and Flooding

The distance between the subject site and the gulf precludes damage due to seismically induced waves (tsunamis) or seiches within the Gulf of California. The Salton Sea is located about 15 miles north of the site at more than 230 feet below mean sea level, which is more than 100-feet below the existing site elevations. The Alamo River is located about one mile east of the site, and the New River is located about 3 miles northwest of the site (see Figure 5B). However, the normal water surface elevations in these rivers are roughly 20 to 40 feet below site grades. Further, the site is mapped in Federal Emergency Management Agency (FEMA) zone designated, “Areas determined to be outside the 0.2% annual chance floodplain” (FEMA, 2008). Consequently, the potential for earthquake induced or other flooding at the site is considered to be low. However, the flooding hazard at the site should be evaluated by the project civil engineer.

5.0 GEOTECHNICAL CONDITIONS

Fill and lacustrine deposits underly the site, as discussed in the *Geology and Subsurface Conditions* section of this report. Geotechnical conditions associated with these units are discussed below.

5.1 Expansive Soils

Laboratory tests indicate the surficial soils at the site should have a “High” to “Very High” Potential Expansion. The results of three Expansion Index tests conducted on bulk soil samples obtained from the ground surface to a depth of about 5 feet below existing grades ranged from 92 to 132, averaging 113 with a median of 116 (i.e., High Potential Expansion). Appendix B provides the test results. Similar Expansion Index test results were obtained from samples collected from our prior investigation to the west of the site, as shown in Appendix C (Group Delta, 2022).

5.2 Compressible Soils

Compressible soils underlie the site. Most of these soils are clay that should experience some time dependent consolidation settlement (i.e., long-term settlement). There are also beds of non-plastic silty sand and silt that should settle elastically with the initial fill and structure loading (i.e., short-term settlement). In general, the clay has a medium to high plasticity and we interpret it to be relatively stiff and slightly overconsolidated from consolidation testing, pocket penetrometer tests, unconfined compressive strength testing, CPT interpretations, and Plasticity Index data. The in-situ moisture contents are generally near the Plastic Limit and the Liquidity Indices are less than 0.7, which indicate relatively stiff and low compressibility soils.

Provided minimal fill placement is needed at the site to achieve the proposed finish grades and foundation loading is limited to the bearing pressures provided in the *Recommendations* section of this report, most of the long-term settlement should occur in a relatively short time following initial loading. However, there are zones of thick clay that could experience some time dependent consolidation settlement if significant loading from fill or foundation loads are proposed for the project. *The estimated settlement magnitude and duration associated with proposed fill placements and foundation loads should be evaluated during the design development phase of the project to evaluate the potential impact to the project.*

5.3 Reuse of Onsite Soils

Soils generated from onsite excavations are anticipated to consist of lean and fat clay (CL and CH) and are not considered suitable for re-use as compacted fill without specific recommendations [see the *Post-Tensioned Slabs (Case B – Existing Clay)* section of this report]. Imported fill is anticipated to be needed to replace the highly expansive materials underlying the proposed structures, flatwork, and pavements. Recommendations for imported fill are provided in the *Recommendations* section of this report.

6.0 CONCLUSIONS

The proposed STEM Facility appears to be feasible from a geotechnical perspective, provided that appropriate measures are implemented during design and construction. Several geotechnical conditions exist on site that need to be addressed.

- Laboratory tests indicate that the surficial soils at the site have a high to very high potential for expansion (Expansion Index greater than 90). The use of thickened foundations and slabs underlain by at least five feet of imported granular non-expansive compacted fill could reduce the potential for future distress to the building associated with soil expansion. Alternatively, a post-tensioned slab-on-grade could be used to support the new building. Alternative post-tension slab design parameters are provided for slabs bearing on either imported select sand or compacted on-site clay.
- The site is underlain predominantly by clay soils that are considered compressible. Placement of new fill and foundation loads will induce time dependent settlement. Given that little information is currently available about the proposed structure and site grading, the settlement magnitude and duration associated with proposed fill placements and foundation loads should be evaluated during the design development phase of the project to evaluate potential impacts.
- Soils derived from onsite excavations are not considered suitable for reuse as engineered fill without specific recommendations. Laboratory tests indicate the fill soils primarily consist of lean and fat clay (CL and CH) with a high to very high expansion potential. To reduce the potential for heave related distress, we recommend placing and compacting imported non-expansive granular material beneath structures, pavements, flatwork, and other heave-sensitive improvements.
- Groundwater was encountered at the site at depths ranging from about 8 to 12 feet below existing surface grades. The site is also located in an area of high seismic activity, and the potential does exist for relatively minor earthquake induced liquefaction settlement of the granular lacustrine deposits beneath the site. The use of thickened and heavily reinforced conventional building foundations or post-tensioned slabs could help to reduce the potential for distress to the building associated with post-liquefaction settlement (as well as soil expansion).
- Laboratory tests indicate that the clayey surficial soils at the site present a *severe* risk of sulfate attack and are also *very corrosive* to buried metals. The recommended placement of two to five feet of imported sand beneath the sidewalks and building slabs-on-grade could help to reduce the potential for sulfate attack and corrosion. However, sulfate resistant Type V cement is recommended for use at the site. Various corrosion control measures may also be needed for buried metal structures. A corrosion consultant may be contacted.
- The site is situated within a zone of high seismic activity. The strong ground shaking hazard may be mitigated by structural design in accordance with the applicable provisions of the governing California Building Code and minimum CSU Seismic Requirements. The potential for flooding at the site should be addressed by the project civil engineer.

7.0 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.

7.1 Plan Review

We recommend that grading and foundation plans be reviewed by Group Delta prior to finalization. We anticipate that substantial changes in the development may occur from the preliminary design concepts used for this investigation. Such changes may require additional geotechnical evaluation, which may result in substantial modifications to the remedial grading and foundation recommendations provided in this report.

7.2 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.

7.3 Earthwork

Grading and earthwork should be conducted in general accordance with the requirements of the current California Building Code, CSU Grading Ordinances, and the earthwork recommendations provided within this report. The following recommendations are provided regarding specific aspects of the proposed earthwork. These recommendations should be considered subject to revision based on the conditions observed by the geotechnical consultant during the grading operations.

7.3.1 Site Preparation

General site preparation should begin with the removal of deleterious materials, including any existing structures, vegetation, turf, contaminated soil, trash, and demolition debris. Existing subsurface utilities or groundwater wells that underly the proposed improvements should be properly abandoned and relocated outside of the proposed building footprint. Excavations associated with abandonment operations should be backfilled and compacted as described in *Fill Compaction* Section of this report. Wells, if present, should be abandoned per local and State guidelines. Alternatively, abandoned utilities may be grouted with a two-sack sand-cement slurry under the observation of the project geotechnical consultant.

7.3.2 Improvement Areas

At least two feet of import granular compacted fill with an Expansion Index less than 20 is recommended beneath new concrete sidewalks and exterior flatwork areas. To accomplish this objective, the upper two feet of soil below slab subgrade (i.e., bottom of the slab) should be excavated and removed from the site. The over-excavation should include the soil within 2-feet of the sidewalk perimeter (measured horizontally). The resulting excavation surface should be scarified to a depth of 12 inches, brought to 3-percentage points or more above optimum moisture content and compacted to at least 90 percent of the maximum dry density per ASTM D1557. The excavation bottom should then be backfilled to the planned slab subgrade elevations using an imported non-expansive granular material and be compacted in accordance with the recommendations in the *Fill Compaction* section below. Subgrade compaction should be conducted immediately prior to placing concrete or base.

7.3.3 Building Areas

The clayey lacustrine deposits beneath the proposed building consist of lean clay (CL) and fat clay (CH) that have a “high” to “very high” expansion potential. We recommend that clayey soil beneath the proposed building be removed to 5 feet below the finish pad elevations (i.e., below the bottom of the slab) or 3 feet below the bottom of foundations, whichever is deeper. The remedial excavations should extend at least 5 feet horizontally beyond the perimeter of the proposed building, including any isolated pad footings that are outside of the building footprint. However, the excavations should not pass below a 1:1 plane extending down and out from the bottom outside edge of any existing foundations to avoid undermining and potential distress to existing structures. The resulting excavation surface should be scarified to a depth of approximately 12 inches, brought to 3-percentage points or more above optimum moisture content, and compacted to at least 90 percent of the maximum dry density at per ASTM D1557. The excavation should then be backfilled to the planned slab subgrade elevations using an imported non-expansive (Expansion Index less than 20) granular material and be compacted in accordance with the recommendations in the *Fill Compaction* section below.

7.3.4 Fill Compaction

Fill and backfill should be placed and compacted at or slightly above optimum moisture content per ASTM D1557 using equipment capable of producing a uniformly compacted product. The maximum loose lift thickness should be 8 inches, unless performance observed and testing during earthwork indicates a thinner loose lift is needed, or a thicker loose lift is possible, up to a loose lift thickness of 12 inches.

The minimum recommended relative compaction is 90 percent of the maximum dry density per ASTM D1557. Sufficient observation and testing should be performed by the project geotechnical consultant during grading so that an opinion can be rendered as to the compaction achieved. Rocks or concrete fragments greater than 6 inches in maximum dimension should not be used in compacted fill.

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.

7.3.5 Import Soil

Imported fill sources should be observed and tested by the project geotechnical consultant prior to hauling onto the site to evaluate the suitability for use. In general, imported fill materials should consist of granular soil with 100 percent passing the 3-inch sieve, more than 70 percent passing the ¾-inch sieve, and less than 35 percent passing the No. 200 sieve based on ASTM C136, and have an Expansion Index less than 20 based on ASTM D4829. Import soils should also have a negligible potential for sulfate attack (i.e., sulfate content less than 0.1 percent). Samples of the proposed import should be tested by the geotechnical consultant to evaluate the suitability of these materials for their proposed use.

Additional testing per the guidelines provided by the Department of Toxic Substances Control (DTSC, 2001) is required by the Owner prior to accepting soil for import. The test results should meet the most stringent State and Federal residential screening levels including the most up-to-date DTSC Modified Screening Levels (DTSC-SLs) and United States Environmental Protection Agency Regional Screening Level (RSL).

7.3.6 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 TX7 (or approved similar) 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 recommended compaction within the base, the remainder of the excavation may be backfilled using either compacted soil or aggregate base. If wet soil conditions are encountered where further excavations are needed, an additional 12-inches of free draining open graded material (such as minus ¾-inch crushed rock) should be placed between the stabilizing geogrid and the compacted well graded aggregate base. The open graded material should be completely enveloped in filter fabric (such as Mirafi 140N or approved similar).

7.3.7 Temporary Excavations

Temporary excavations may be needed to construct the planned improvements. Excavations should conform to Cal/OSHA guidelines (2021). In general, we recommended that temporary excavations be inclined no steeper than 1:1 for heights up to 5 feet. Vertical excavations should be shored. Any excavations that encounter groundwater seepage should be evaluated on a case-by-case basis.

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 below assessment of Cal/OSHA Soil Types for temporary slopes is based on preliminary engineering classifications of material encountered in widely spaced explorations.

Based on the findings of our subsurface investigation, the following Cal/OSHA Soil Types may be assumed for planning purposes.

PRELIMINARY CAL/OSHA SOIL TYPES

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

1. This assumes that no groundwater seepage or caving is encountered in the excavations.

7.4 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 structures and top of slopes without ponding. The surface gradient needed to achieve this may depend on the prevailing landscaping. Planters should be designed and built so that water will not seep into the foundation, slab, pavement, or other heave/settlement structure areas. If roof drains are used, the drainage should be channeled by pipe to the storm drain system, or discharge at least 10 feet from buildings. Irrigation should be limited to the minimum needed to sustain landscaping, and consideration should be given to utilizing drought tolerant landscape to further minimize water used for irrigation. Existing drainage channels through the proposed site should be re-routed and graded do drain away from improvement areas. Excessive irrigation, surface water, water line leaks, or rainfall may cause perched groundwater to develop within the underlying soil.

7.5 Storm Water Management

We anticipate that various bioretention basins, swales or pervious paver block pavements may be proposed to promote on-site infiltration for storm water Best Management Practice (BMP). In order to help evaluate the feasibility of on-site infiltration, the infiltration rate of the on-site soil may be estimated using borehole percolation or double ring infiltrometer tests conducted within the planned BMP areas. However, our experience indicates that infiltration testing in clay soils should result in a "No Infiltration" condition per the applicable BMP Design Manual. An infiltration rate of less than 0.01 inches per hour is estimated based on previous infiltration tests we have conducted in similar clay soils. The clays typically have a permeability of 10^{-7} to 10^{-9} cm/s (essentially impermeable).

7.6 Seismic Design

Structures should be designed in general accordance with the governing seismic provisions of the 2022 California Building Code, as well as the minimum seismic design requirements of the California State University (CSU, 2020). Field testing consisting of shear wave measurements in CPT-10 resulted in average shear wave velocity in the upper 30 meters ($V_{S,30}$), or 100 feet, of approximately or 190 m/s (625 ft/s). Based on these measurements, the Site Classification using Chapter 20 of ASCE 7-16 would be Site Class D. The following preliminary seismic design parameters are recommended by the California State University Seismic Requirements (CSU, 2020) for the site.

CSU – SDSU IMPERIAL CAMPUS SEISMIC DESIGN PARAMETERS

Hazard Level	Parameter	Site Class D
BSE-1N	PGA_D	0.40
	S_{D0}	0.40
	S_{DS}	1.00
	S_{D1}	0.68
BSE-2N	PGA_M	0.59
	S_{M0}	0.60
	S_{MS}	1.50
	S_{M1}	1.02

7.7 Foundation Recommendations

The foundations for the new buildings should be designed by the project structural engineer using the following geotechnical parameters. These are only minimum 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 decisions made during design development and the conditions observed by the geotechnical consultant during grading.

7.7.1 Conventional Foundations

The following recommendations assume that remedial grading will be conducted for the building pad area as recommended in the *Earthwork* Section, and that the building pad grade will be underlain by at least 5-feet of imported granular non-expansive compacted fill (Expansion Index of 20 or less). Conventional shallow foundations would be considered appropriate for this condition, as shown in Figure 6.

Allowable Bearing: *2,000 psf (allow ⅓ increase for short-term wind or seismic loads)*

Minimum Footing Width: *12 inches*

Minimum Footing Depth: *24 inches below lowest adjacent soil grade*

Minimum Reinforcement: *Two No. 5 bars at both top and bottom in continuous footings*

7.7.2 Post-Tensioned Slabs

Two different post-tensioned slab foundation design conditions are summarized below. Case A provides recommendations assuming the building will be underlain by at least 5-feet of imported granular non-expansive compacted fill, and Case B assumes that a post-tension slab foundation may be designed to bear directly on recompacted expansive on-site clay. The following recommendations are provided using the Post-Tensioning Institute (PTI) Document *PTI DC10.5-19* (2019).

7.7.2.1 Case A – Select Fill

For Case A, we have assumed that remedial grading will be conducted per our recommendations, and that the proposed building will be underlain by at least 5-feet of imported granular non-expansive compacted fill in accordance with the *Earthwork* Section of this report, overlying the existing expansive clay. The following post-tension slab foundation design parameters are considered applicable to buildings that will be underlain by such conditions. Note that these recommendations should be considered preliminary, and subject to revision based on the as-graded conditions observed by the geotechnical consultant during fine grading of the site.

Post-Tension Slab Design Parameters (Case A):

Moisture Variation Distance, e_m:	*Center Lift:*	*5.5 feet*
	Edge Lift:	*2.5 feet*
Differential Soil Movement, y_m:	*Center Lift:*	*0.7 inches*
	Edge Lift:	*1.2 inches*
Allowable Bearing:	*2,000 psf at slab subgrade*	

7.7.2.2 Post-Tensioned Slabs (Case B – Existing Clay)

As an alternative to remedial grading to replace the highly expansive clays with imported sand as described in Case A above, a post-tension slab foundation may be designed to bear directly on the highly expansive on-site clay. For Case B, the undocumented fill soils underlying the proposed structure should be excavated and replaced as a uniformly compacted fill beneath the building (as a minimum). The undocumented fill depth is anticipated to extend approximately two to three feet below existing grades at the site. The clayey fill soil should be compacted to at least 90 percent relative compaction at 3-percentage points or more above optimum moisture content per ASTM D1557. The following post-tension slab foundation design parameters are considered appropriate for a building underlain by recompacted clayey fill soils.

Post-Tension Slab Design Parameters (Case B):

Moisture Variation Distance, e_m:	*Center Lift:*	*5.5 feet*
	Edge Lift:	*3.0 feet*
Differential Soil Movement, y_m:	*Center Lift:*	*2.5 inches*
	Edge Lift:	*4.0 inches*
Allowable Bearing:	*2,000 psf at slab subgrade*	

7.7.3 Settlement

Total and differential settlements of the proposed structure due to the allowable bearing loads provided above are not expected to exceed 1.5 and 0.75 inches in 30 feet, respectively. In addition to static settlement, the site may experience post-liquefaction total and differential settlements on the order of approximately 1-inch and 0.5 inches in 30 feet, respectively, as discussed in *Earthquake Induced Ground Failure* Section.

7.7.4 Lateral Resistance

Lateral loads against the structure may be resisted by friction between the bottoms of footings and slabs and the underlying soil, as well as passive pressure from the portion of vertical foundation members embedded into compacted fill. A coefficient of friction of 0.25 and a passive pressure of 250 psf per foot of depth may be used for level ground conditions.

7.8 On-Grade Slabs

Conventional concrete building slabs should be at least 6 inches thick and should be reinforced with at least No. 3 bars on 12-inch centers, each way. Slab thickness, control joints, and reinforcement should be designed by the project structural engineer and should conform to the requirements of the current California Building Code and based on the proposed slab loading.

7.8.1 Moisture Protection for Slabs

Moisture protection should comply with requirements of the current CBC, American Concrete Institute, and the desired functionality of the interior ground level spaces. The project 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. The membrane may be placed between the concrete slab and the clean sand or finished subgrade immediately below the slab, provided it is protected from puncture and repaired per the manufacturer’s recommendations if damaged. Note that the CBC specifies that a capillary break such as 4 inches of clean sand be used beneath building slabs (as defined and installed per the California Green Building Standards), along with a Vapor Retarder.

7.9 Exterior Slabs

Exterior slabs and sidewalks subjected to pedestrian traffic and light vehicle loading (e.g., golf carts) should be at least 4 inches thick and underlain by 2-feet of imported granular non-expansive compacted fill in accordance with the *Improvement Areas* section of this report. 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 would consist of 6x6 W2.9/W2.9 welded wire fabric placed securely at mid-height of the slab.

7.10 Earth-Retaining Structures

Backfilling retaining walls with expansive soil can increase lateral pressures well beyond normal active or at-rest pressures. Retaining walls should be backfilled with import granular material with an Expansion Index of less than 20. The on-site soils do not meet this criterion. 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. Retaining wall foundations should be designed using the recommendations included in the *Shallow Foundations* section of this report.

7.10.1 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 pounds per cubic foot (pcf). The active pressure should be used for walls free to yield at the top at least ½ percent of the wall height. Retaining walls that are located adjacent to vehicular traffic areas may be designed to resist a uniform lateral surcharge pressure of 100 pounds per square foot (psf), resulting from a typical 300 psf traffic surcharge acting behind the wall. Retaining walls should contain adequate drainage to relieve the buildup of hydrostatic pressures. Our recommended wall drainage details are shown in Figure 7.

7.11 Preliminary Pavement Design

For all pavement areas, the upper 12 inches of clayey subgrade soil (below the pavement aggregate base section) should be removed. This removal should extend 2 feet or more beyond the outside edge of the pavement perimeter measured horizontally. The resulting excavation surface should be scarified immediately prior to constructing the pavements, brought to optimum moisture, and compacted to at least 90 percent of the maximum dry density at 3-percentage points or more above optimum moisture content per ASTM D1557. The excavation bottom should then be backfilled to the planned pavement subgrade (i.e., bottom of the aggregate base section) using an imported non-expansive (expansion index less than 20) granular soil (i.e., subbase). Aggregate base and subbase should be compacted to 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Aggregate base should conform to the Standard Specifications for Public Works Construction (*SSPWC),* Sections 200-2.2, -2.4, or -2.5 (PWSI, 2021). Asphalt concrete should conform to Section 203-6 of the *SSPWC* and should be compacted to 91 and 97 percent of the Rice density per ASTM D2041 (PWSI, 2021).

7.11.1 Asphalt Concrete

Based on our previous experience, we anticipate that the clayey on-site soils have an R-Value of 5 or less. Preliminary asphalt concrete pavement design was conducted using the Caltrans Design Method (2018). We anticipate that a Traffic Index ranging from 5.0 to 6.0 may apply to new pavement areas. The project civil engineer should review the assumed Traffic Indices to determine if and where they may be applicable. Based on the minimum R-Value of 5 and the assumed range of Traffic Indices, the following pavement sections would apply.

SUMMARY OF PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS

PAVEMENT TYPE	TRAFFIC INDEX	ASPHALT SECTION	BASE SECTION	SUBBASE SECTION[1]
Passenger Car Parking	5.0	3 Inches	10 Inches	12 Inches
Light Truck Traffic Areas	6.0	4 Inches	12 Inches	12 Inches

***1) NOTE**: One foot of imported granular non-expansive subbase should be placed beneath the pavement section to reduce the potential for cracking due to soil heave/shrink behavior.*

7.11.2 Portland Cement Concrete

Concrete pavement design was conducted in general accordance with the simplified design procedure of the Portland Cement Association (1984). 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. The flexural strength of the pavement concrete should be confirmed during construction using ASTM C78. For concrete pavement design, the subgrade materials were assumed to provide “low” support, based on our experience with similar materials. Using these assumptions and the same traffic indices presented previously, we recommend that the PCC pavement sections at the site consist of at least 6 inches of concrete placed over 6 inches of compacted aggregate base over 12 inches of imported granular non-expansive subbase (Expansion Index less than 20).

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

7.12 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 below.

7.12.1 Thrust Blocks

Lateral resistance for thrust blocks may be evaluated using a passive pressure value of 250 pounds per square foot (psf) per foot of embedment, assuming a triangular distribution and level ground conditions. This value may be used for thrust blocks embedded into compacted fill soils as well as the underlying lacustrine deposits, provided that these soils are located above the groundwater table.

7.12.2 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 700 pounds per square inch (psi) is recommended for the general conditions, assuming granular bedding material is placed around the pipe and the soils are located above the groundwater table.

7.12.3 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, disintegrated granite or granular materials with a Sand Equivalent of 20 or more. Where open graded material (e.g., ¾-inch minus crushed rock) is used as bedding and shading around and above the pipe, we recommend that open graded material should be completely enveloped in filter fabric (such as Mirafi 140N).

Where pipeline or trench 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 with a Sand Equivalent of 20 or more 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.

7.13 Reactive Soils

In order to assess the sulfate exposure of concrete in contact with the site soils, samples were tested for pH, resistivity, water-soluble sulfate and chloride content, as shown in Appendix B. The sulfate test results indicate that the on-site soils present a *severe* potential for sulfate attack based on commonly accepted criteria (Bentivegna et al., 2020). A *negligible* sulfate content is recommended for any imported soils and should be confirmed through laboratory testing prior to import.

The saturated resistivity and chloride content of the near surface soils are indicative of a *corrosive* to *very corrosive* soil with respect to buried metals based on commonly accepted criteria (Caltrans, 2021). Typical corrosion control measures should be incorporated into the project design, such as providing minimum clearances between reinforcing steel and soil, and sacrificial anodes for any buried metal structures. A corrosion consultant may be contacted for specific recommendations.

8.0 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 humans 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.

9.0 REFERENCES

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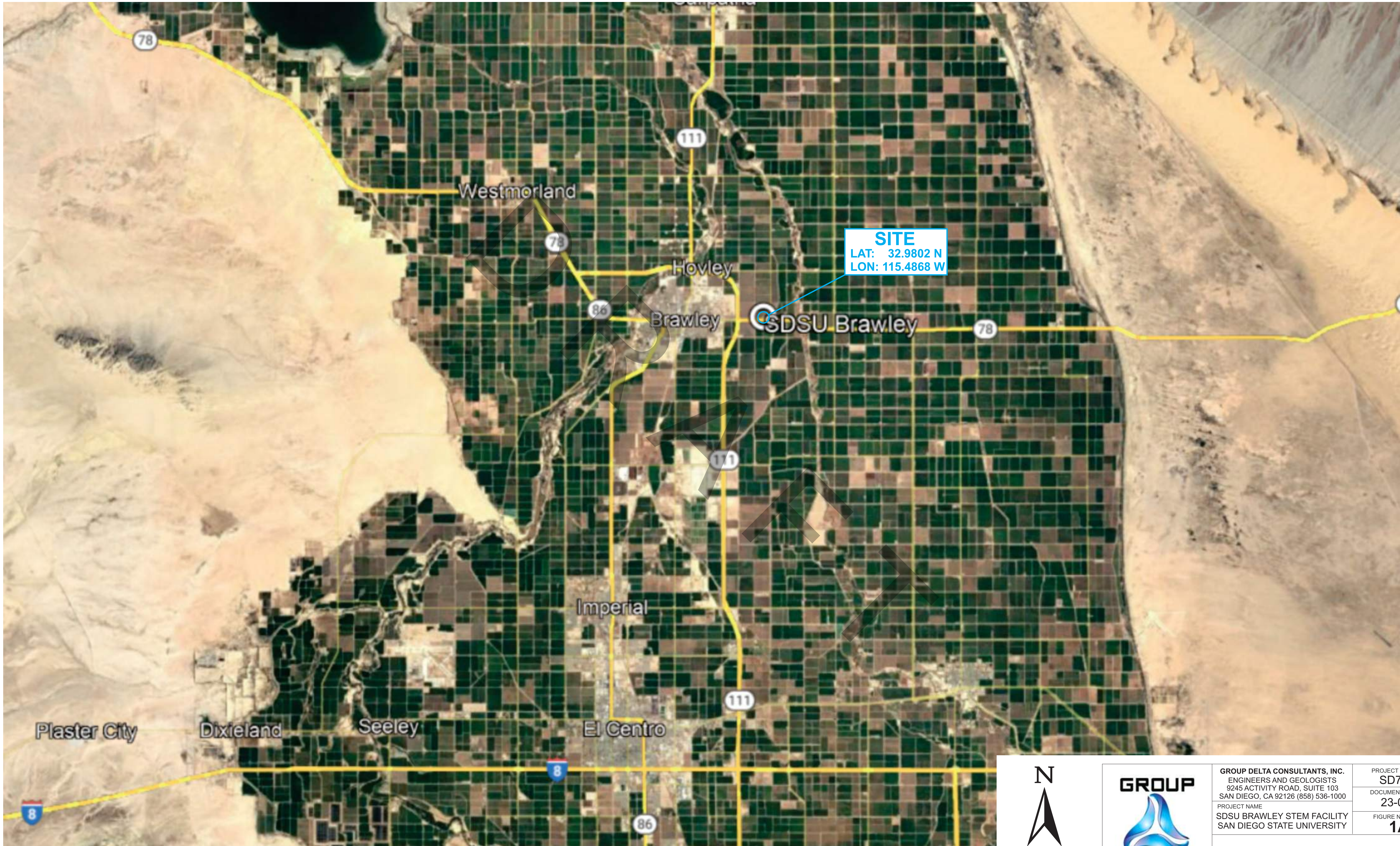
FIGURES



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NO SCALE

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CA 92126 (858) 536-1000

PROJECT NUMBER	SD725A
DOCUMENT NUMBER	23-0017
PROJECT NAME	SDSU BRAWLEY STEM FACILITY SAN DIEGO STATE UNIVERSITY
FIGURE NUMBER	1A

SITE LOCATION

GROUP DELTA

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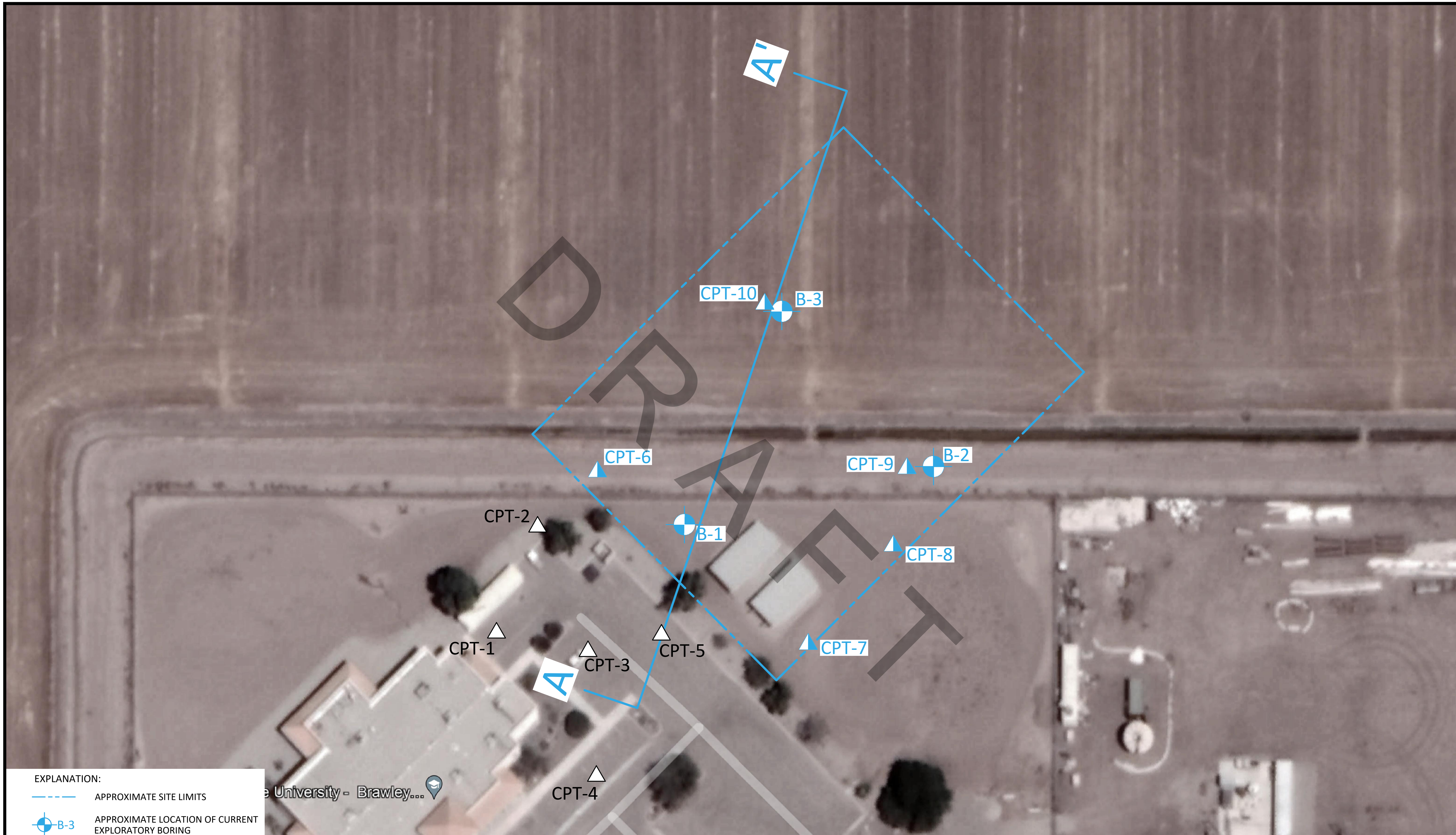
PROJECT NAME SDSU BRAWLEY STEM FACILITY SAN DIEGO STATE UNIVERSITY	DOCUMENT NUMBER 23-0017
	FIGURE NUMBER **1B**

SITE VICINITY



EXPLANATION:

- APPROXIMATE SITE LIMITS
- B-3 APPROXIMATE LOCATION OF CURRENT EXPLORATORY BORING
- CPT-10 APPROXIMATE LOCATION OF CURRENT CONE PENETRATION TEST
- B-1 APPROXIMATE LOCATION OF PREVIOUS CONE PENETRATION TEST (GROUP DELTA, 2022)
- A A' APPROXIMATE LOCATION OF GEOTECHNICAL CROSS SECTION

N

0 50' 100'

REFERENCE: GOOGLE, INC (2023) GOOGLE EARTH PRO, AERIAL IMAGERY DATED: AUGUST 23, 2020

SDSU BRAWLEY STEM FACILITY SAN DIEGO STATE UNIVERSITY	EXPLORATION LOCATIONS	GROUP DELTA	
		PROJECT NUMBER	FIGURE NUMBER
		SD725A	2

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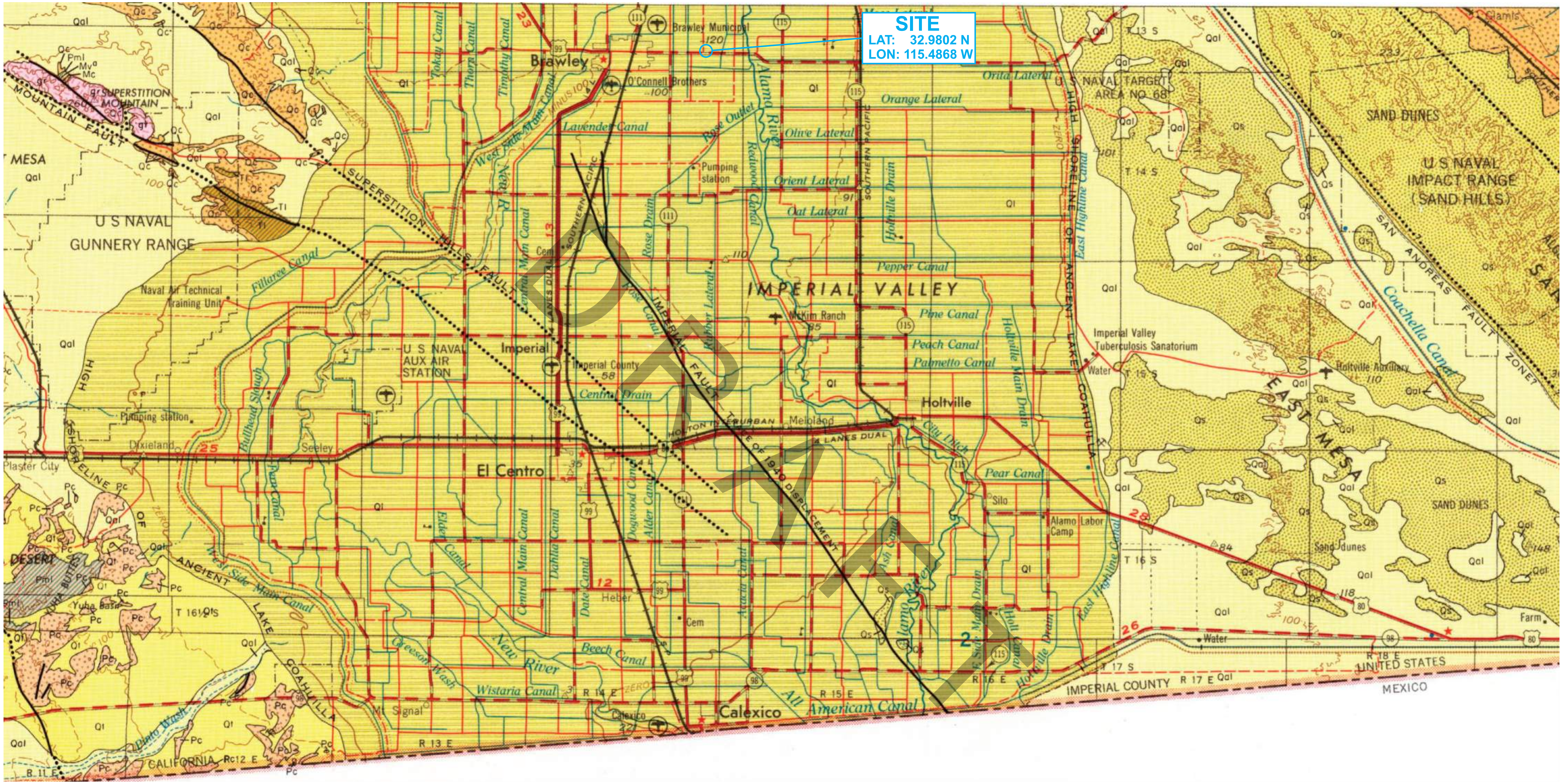


EXPLANATION:

- Ql — Quaternary lake deposits
- Qal — Alluvium
- Qs — Dune sand
- Qt — Quaternary nonmarine terrace deposits
- Qc — Pleistocene nonmarine

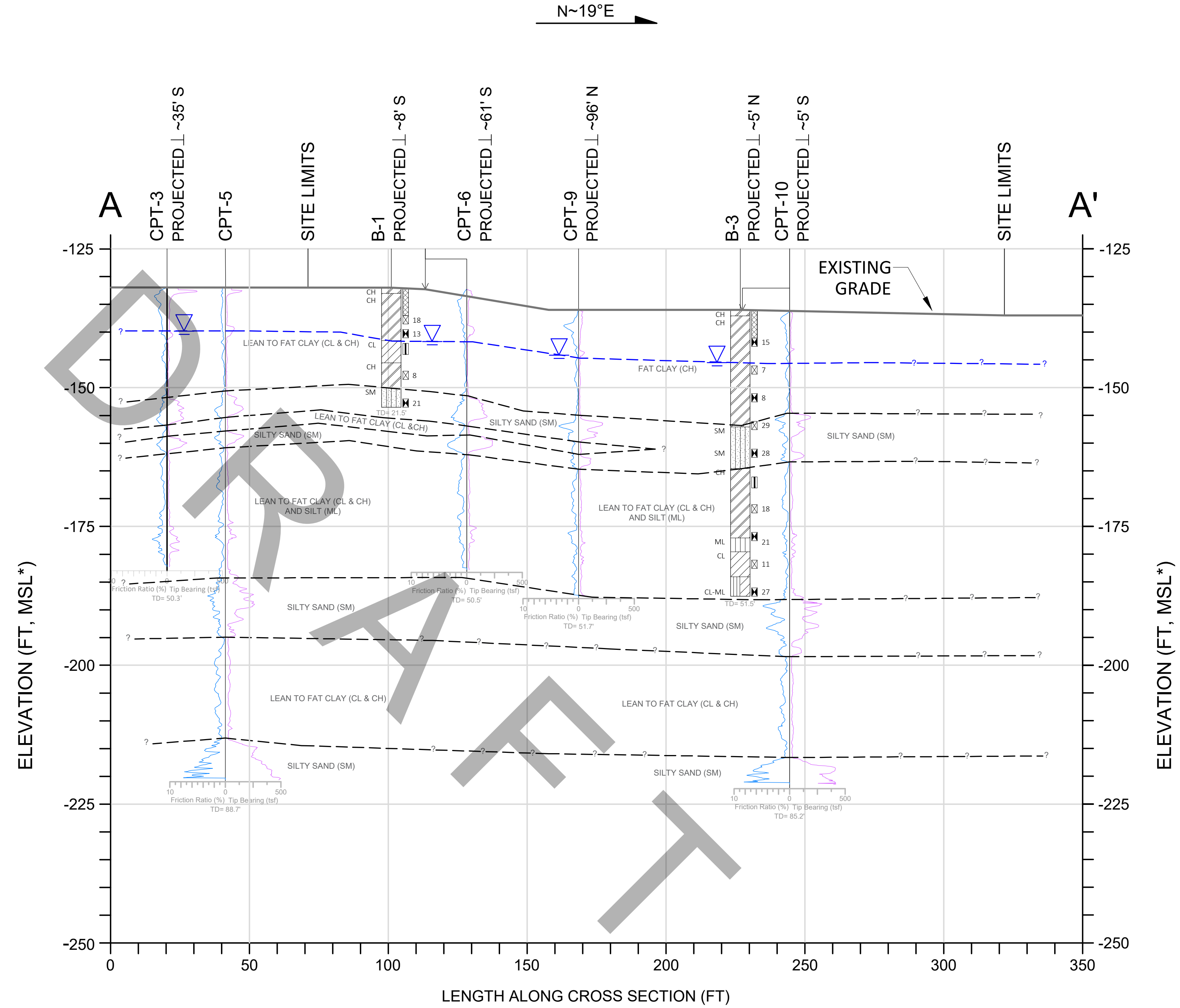
Reference: Rudolph Strand (1962). Geologic Map of California, San Diego-El Centro, Scale 1:250,000.

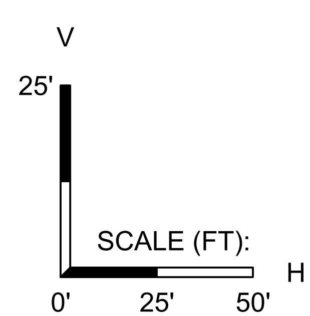
N

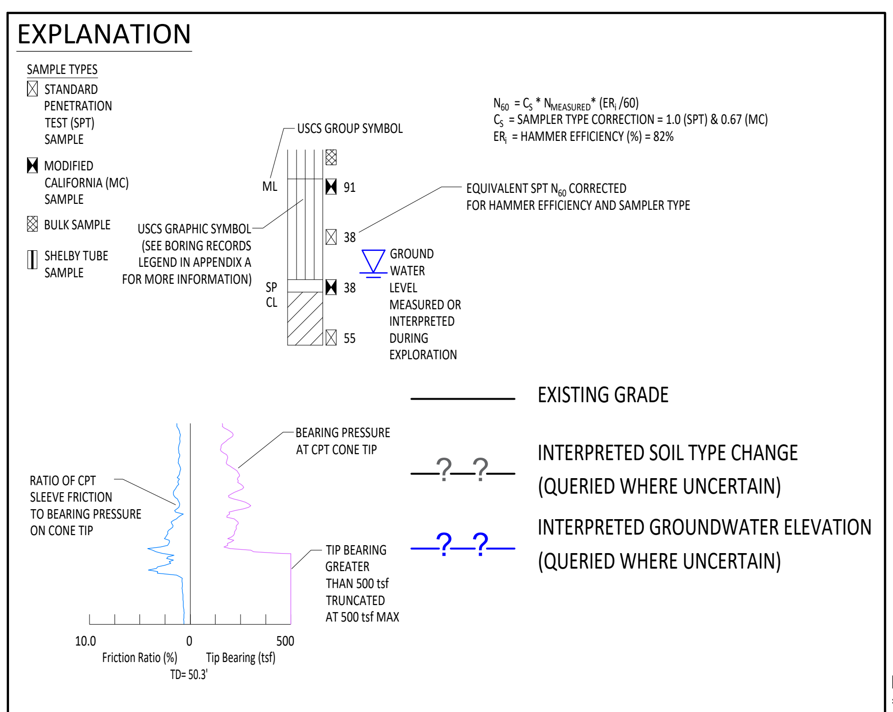
NO SCALE

GROUP DELTA

GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD725A DOCUMENT NUMBER 23-0017
PROJECT NAME SDSU BRAWLEY STEM FACILITY SAN DIEGO STATE UNIVERSITY	FIGURE NUMBER 3
GEOLOGY	







REFERENCE: GOOGLE, INC (2023) GOOGLE EARTH PRO, AERIAL IMAGERY DATED: AUGUST 23, 2020

NOTE: DIRECTIONS, LOCATIONS, ELEVATIONS, AND SCALE ARE APPROXIMATE
*ELEVATION ESTIMATED USING GOOGLE EARTH PRO (GOOGLE INC., 2023)

SDSU BRAWLEY STEM FACILITY
SAN DIEGO STATE UNIVERSITY

GEOTECHNICAL CROSS SECTION A-A'

GROUP DELTA

PROJECT NUMBER	FIGURE NUMBER
SD725A	4

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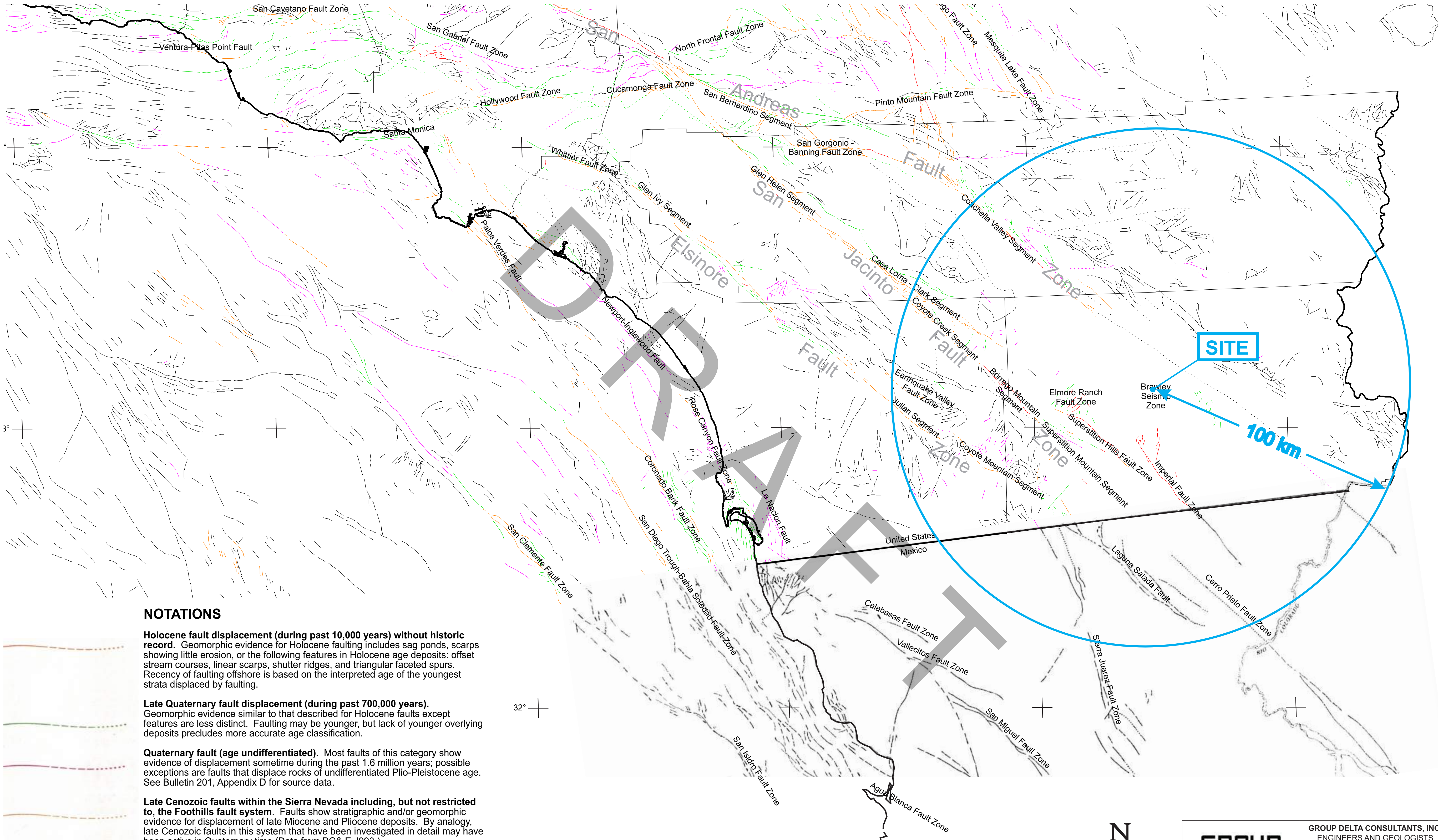


NOTATIONS

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.)

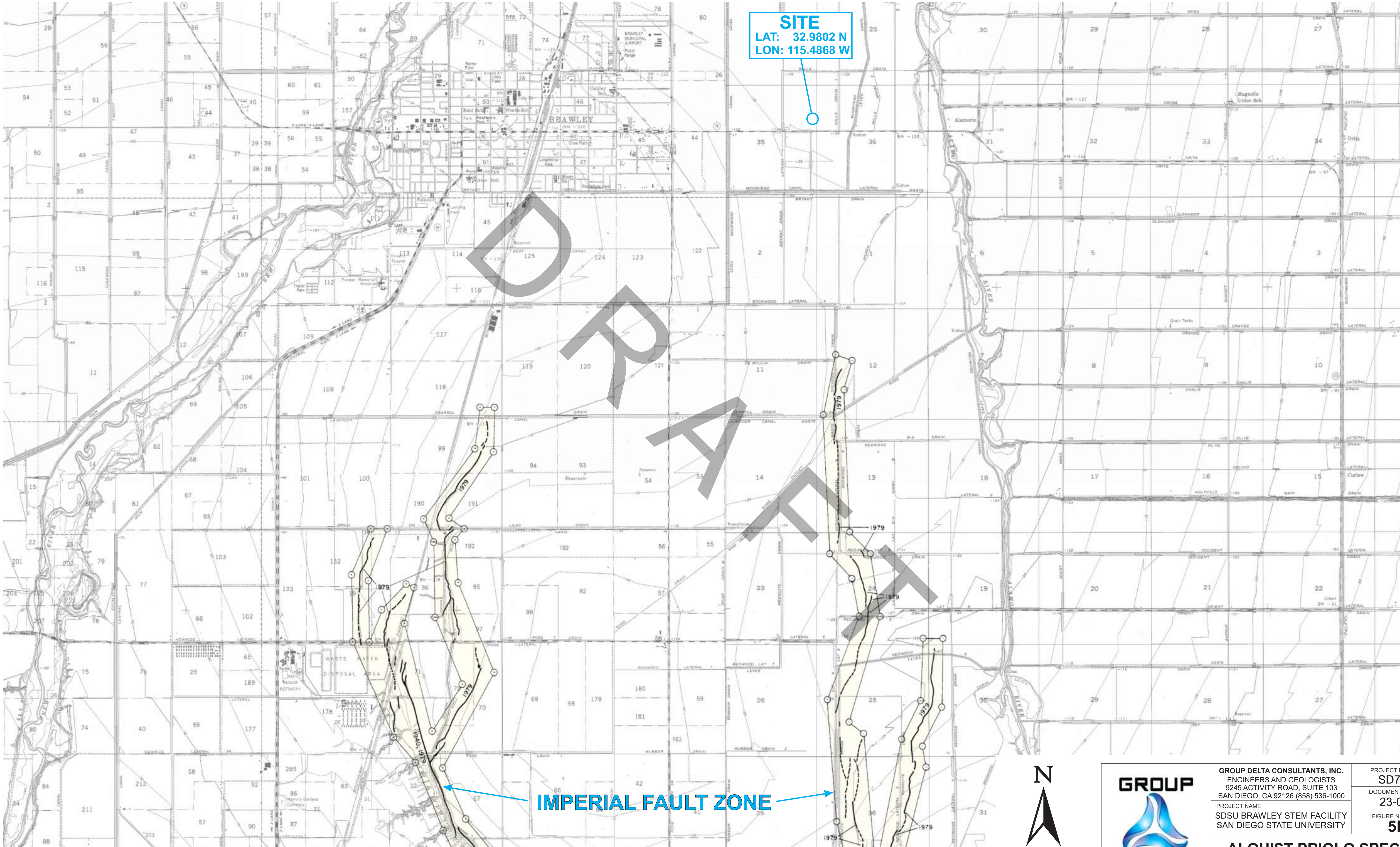
Pre-Quaternary fault (older than 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissance nature, or was not done with the object of dating fault displacements. Faults in this category are not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

N

NO SCALE

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

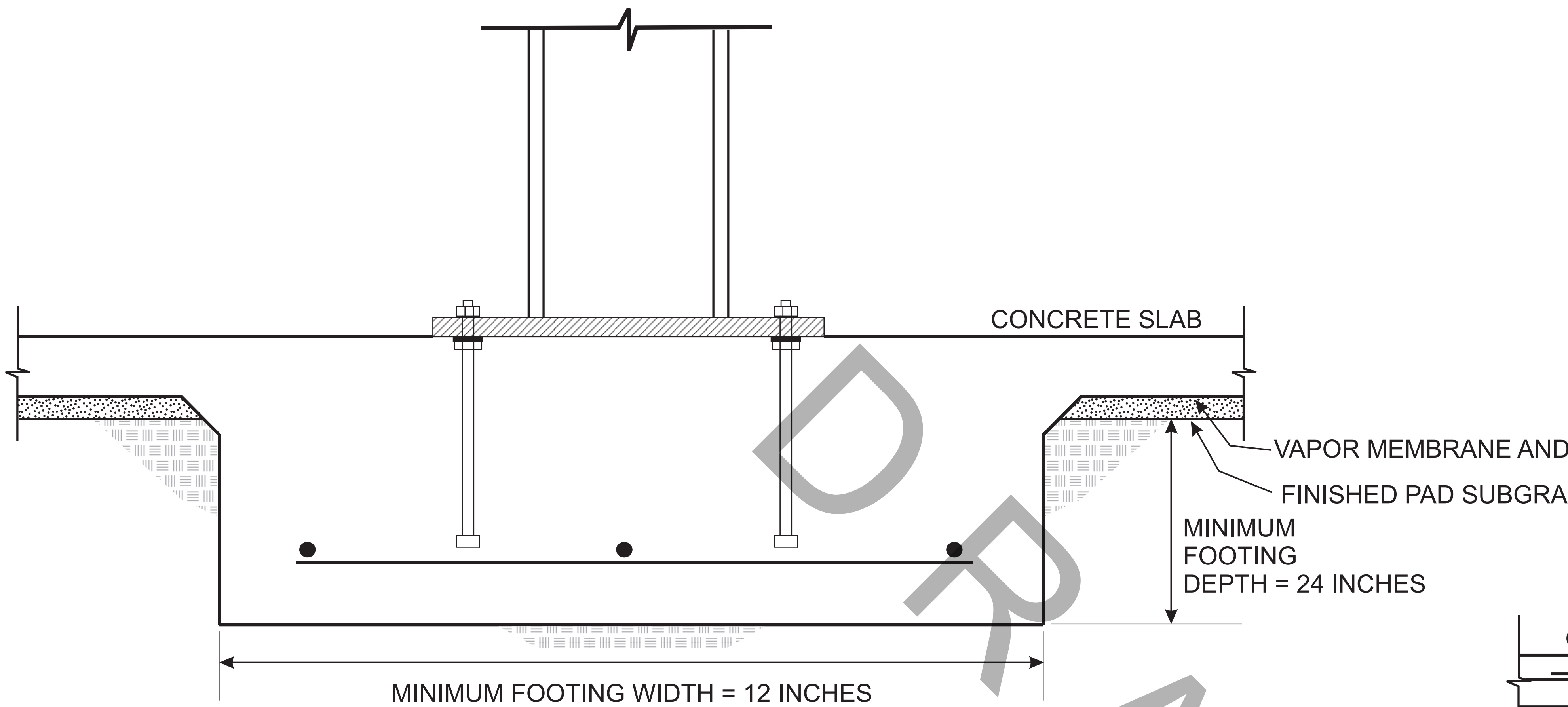
GROUP DELTA	**GROUP DELTA CONSULTANTS, INC.** ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD725A DOCUMENT NUMBER 23-0017
	PROJECT NAME SDSU BRAWLEY STEM FACILITY SAN DIEGO STATE UNIVERSITY	FIGURE NUMBER 5A
	FAULT LOCATIONS	



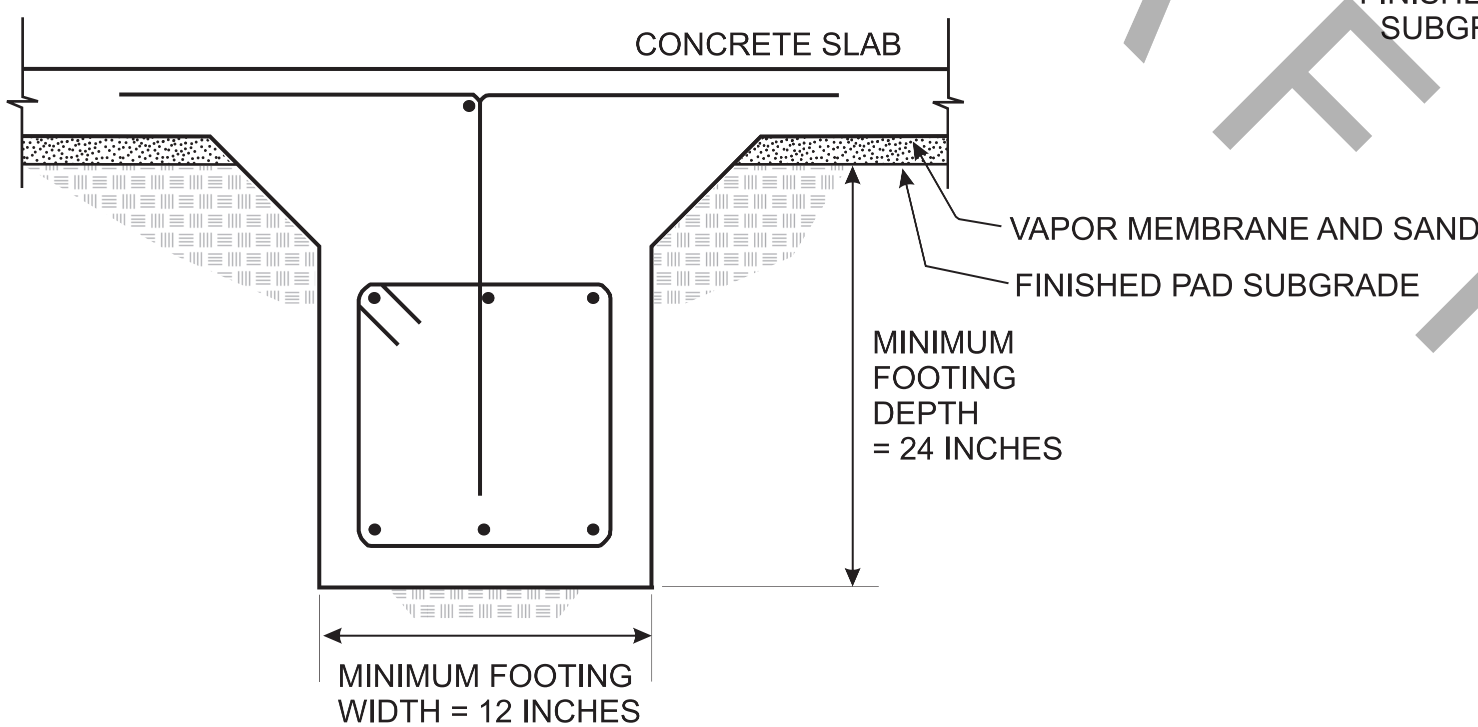
Reference: State of California (1990). *Alquist-Priolo Special Studies Zones, Brawley and Alamario Quadrangles,* Revised Official Map, January 1.

GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD725A
	DOCUMENT NUMBER 23-0017
PROJECT NAME SDSU BRAWLEY STEM FACILITY SAN DIEGO STATE UNIVERSITY	FIGURE NUMBER 5B

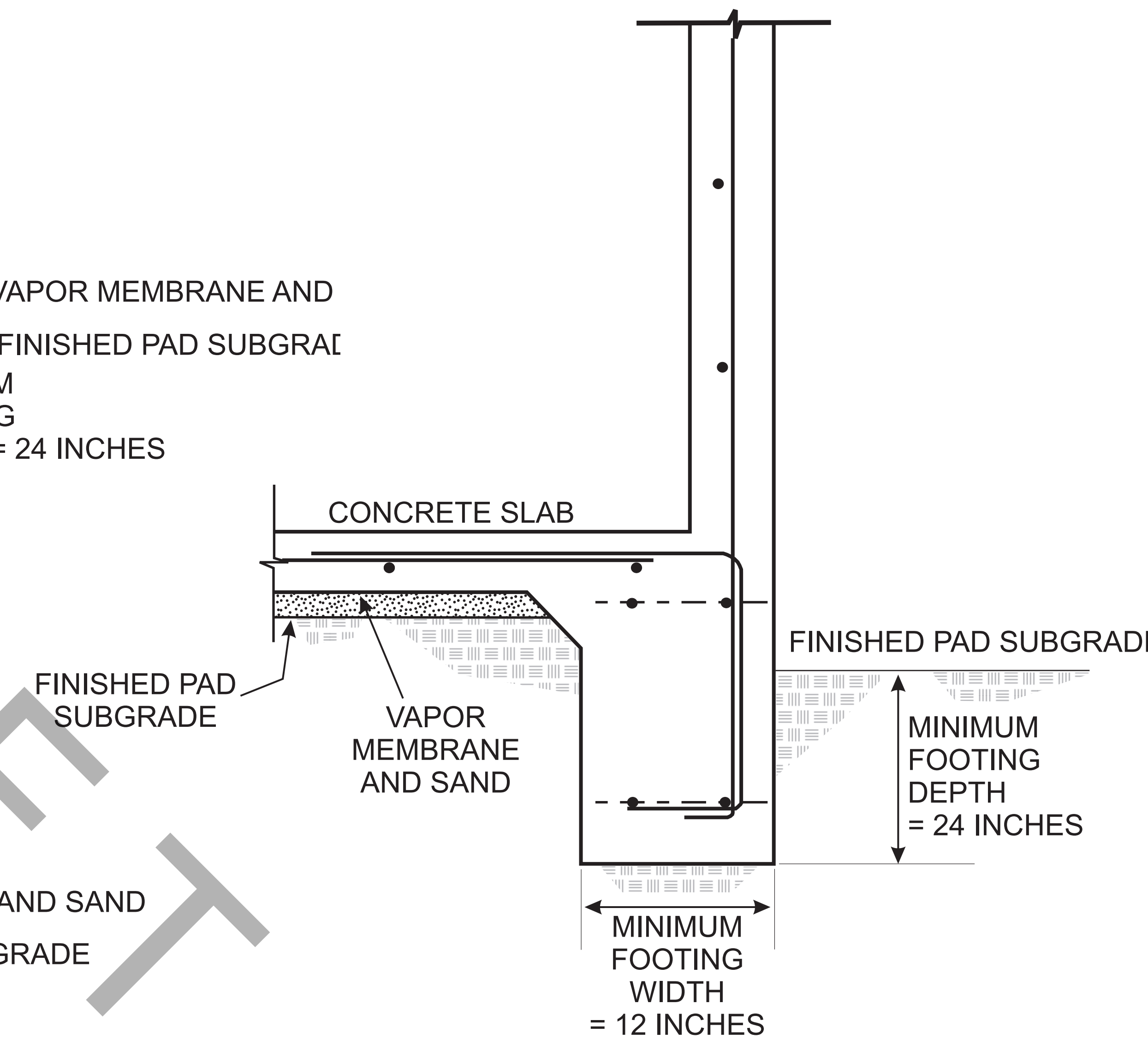
ALQUIST-PRIOLO SPECIAL STUDIES ZONES



SQUARE FOOTING



INTERIOR CONTINUOUS FOOTING

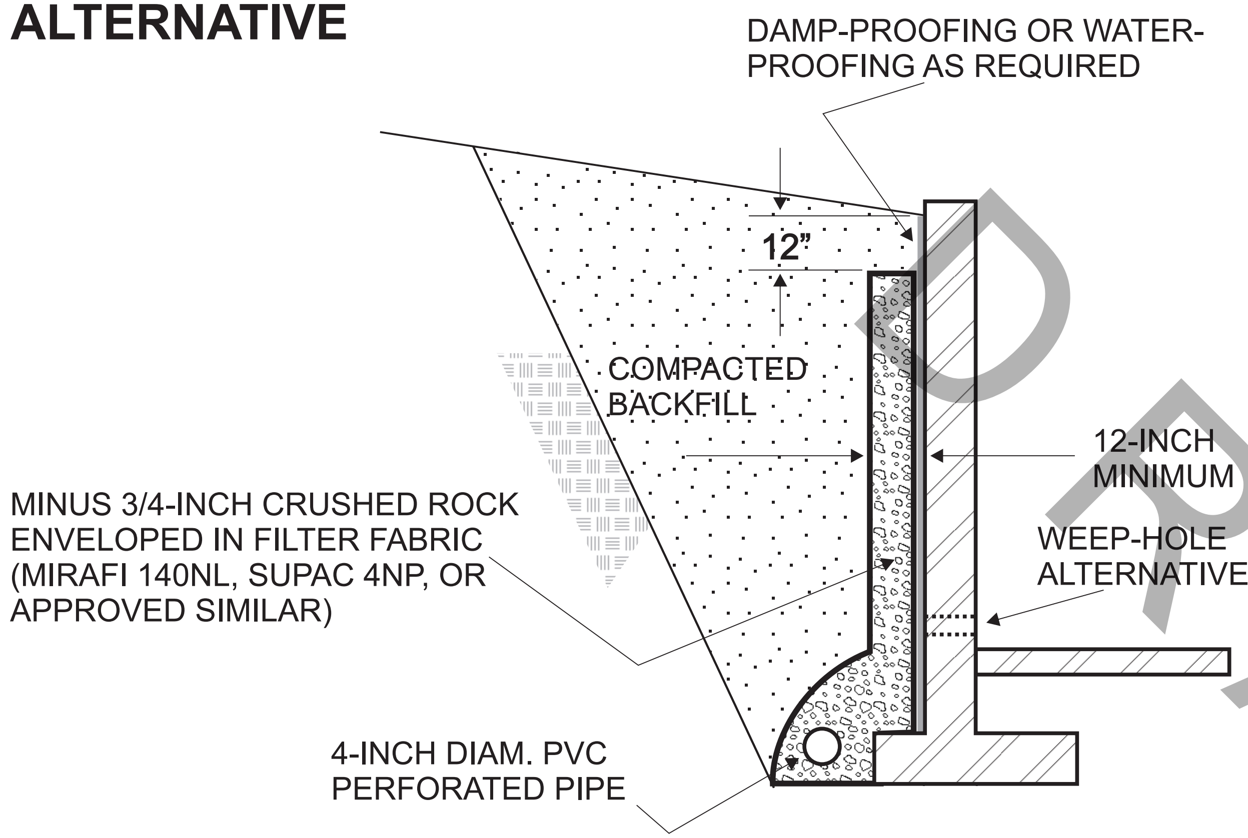


EXTERIOR CONTINUOUS FOOTING

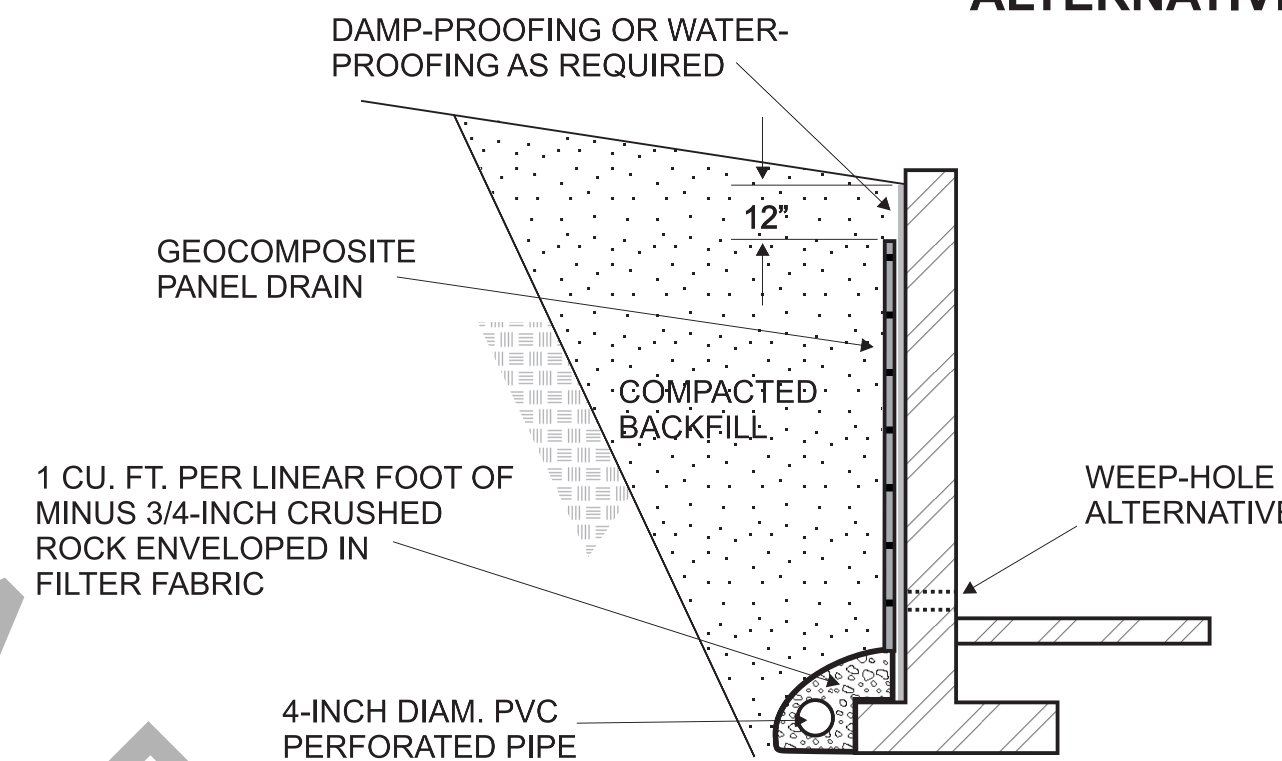
REINFORCEMENT SHOWN FOR ILLUSTRATION PURPOSES ONLY; NO SCALE.

GROUP DELTA	**GROUP DELTA CONSULTANTS, INC.** ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD725A DOCUMENT NUMBER 23-0017
	PROJECT NAME SDSU BRAWLEY STEM FACILITY SAN DIEGO STATE UNIVERSITY	FIGURE NUMBER 6
	SHALLOW FOUNDATIONS	

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.

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PROJECT NUMBER	SD725A
DOCUMENT NUMBER	23-0017
PROJECT NAME	SDSU BRAWLEY STEM FACILITY SAN DIEGO STATE UNIVERSITY
FIGURE NUMBER	7

WALL DRAINAGE DETAILS

APPENDIX A
EXPLORATION RECORDS

APPENDIX A

EXPLORATION RECORDS

Field exploration included a visual reconnaissance of the site, the drilling of three (3) hollow stem and mud rotary exploratory borings and the advancement of five (5) Cone Penetration Test (CPT) soundings on February 17, 2023. The maximum depth of exploration was approximately 85 feet below surrounding grades. A summary of the explorations is included in Table A-1. The approximate exploration locations are shown in Figure 2, Exploration Locations. Logs of the explorations are provided in Figures A-1 through A-10, immediately after the Boring Record Legends.

HOLLOW STEM AND MUD ROTARY BORINGS

The hollow stem and mud rotary exploratory borings were advanced by Tri-County Drilling using a CME 75 truck mounted drill rig. Disturbed samples were collected from the borings using a 2-inch outside diameter unlined Standard Penetration Test (SPT) sampler. Less disturbed samples were collected using a 3-inch outside diameter ring lined sampler (a modified California sampler). Bulk samples were also collected in the upper five feet of the boring. The samples were sealed in plastic bags, labeled, and returned to the laboratory for testing. A summary of the exploratory boring locations, elevations and depths is shown on the following page in Table A-1.

The drive samples were collected from the exploratory borings using an automatic hammer with average Energy Transfer Ratio (ETR) of approximately 82 percent. For each sample, the 6-inch incremental blowcounts were recorded on the logs. The field blow counts (N) were normalized to approximate the standard 60 percent ETR, as shown on the logs (N_{60}). The California ring samples were also corrected for the 3-inch sampler diameter using Burmister's correction factor.

The exploratory borings were logged using the Caltrans Soil and Rock Logging, Classification and Presentation Manual (2010) as a guideline.

CONE PENETRATION TESTS

The CPT soundings were advanced by Kehoe Testing and Engineering in general accordance with ASTM D5778. The CPT soundings were carried out using an integrated electronic cone system manufactured by Vertek. The soundings were advanced using a 30-ton truck-mounted CPT rig. The cone used during the program was a 15 centimeter squared (cm^2) cone and recorded the following parameters at approximately 2.5 centimeter depth intervals:

- Cone Resistance;
- Sleeve Friction;
- Dynamic Pore Pressure;

APPENDIX A

EXPLORATION RECORDS (Continued)

At location CPT-10, shear wave velocity measurements were obtained at five-foot intervals to a depth of approximately 85 feet, where CPT refusal was encountered due to flexure in the rods. The shear wave was generated using an air-actuated hammer placed under the CPT rig at a specified offset distance from the rods. The cone was equipped with a triaxial geophone, which recorded the shear wave signal generated by the air hammer. The above parameters were recorded and viewed in real time using a laptop computer. A summary of the collected shear wave measurements is presented in Figure A-9.

Note: The exploration locations were measured in the field using a Garmin GPSMAP 64st Global Positioning System (GPS) receiver and by visually estimating, pacing or taping distances from nearby landmarks, if available. The surface elevations were estimated using GoogleEarth Pro (Google, Inc., 2023). The locations and elevations provided should not be considered more accurate than is implied by the scale of the map and the accuracy of the equipment used to locate the explorations. The lines designating the interface between differing soil materials on the logs may be abrupt or gradational. Further, soil conditions at locations between the explorations may be substantially different from those at the specific locations we explored. The Boring Records are part of a geotechnical report which must be considered in its entirety.

Table A-1 – Explorations Summary (see Figure 2, Exploration Locations)

Exploration ID	Latitude [°]	Longitude [°]	Top Elevation MSL [1] [FT]	Exploration Depth [FT]	Bottom Elevation MSL [FT]	Figure No.
B-1	32.980090	115.487080	-132	21.5	-154	A-1
B-2	32.980220	115.486660	-134	21.5	-156	A-2
B-3	32.980170	115.486650	-136	51.5	-188	A-3
CPT-6	32.980180	115.487230	-135	50.5	-186	A-4
CPT-7	32.979930	115.486870	-133	50.4	-184	A-5
CPT-8	32.980070	115.486720	-133	50.7	-184	A-6
CPT-9	32.980180	115.486690	-134	51.7	-186	A-7
CPT-10	32.980420	115.486940	-136	85.2	-221	A-8

[1] GoogleEarth Pro (Google, Inc.) was used to estimate the top elevation of each exploration.

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. SD725A

STEM FACILITY
SAN DIEGO STATE UNIVERSITY
BRAWLEY CAMPUS
BRAWLEY, CALIFORNIA

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 2937)
WA	Percent passing the No. 200 Sieve (ASTM D 1140)

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. SD25A

STEM FACILITY
SAN DIEGO STATE UNIVERSITY
BRAWLEY CAMPUS
BRAWLEY, CALIFORNIA

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. SD725A

STEM FACILITY
SAN DIEGO STATE UNIVERSITY
BRAWLEY CAMPUS
BRAWLEY, CALIFORNIA

BORING RECORD LEGEND #3

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
SDSU Brawley STEM Facility	SD725A	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
560 CA-78, Brawley, California	2/17/2023	2/17/2023	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Tri-County Drilling	Hollow Stem Auger	D. Guzman	C. Vonk

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUNDWATER (ft)
CME 75	8	21.5	-132	▼ NM / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 82%, N_{60} = 1.36*N_{SPT} = 0.91*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	PID READINGS (ppm)	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
													FILL: Fat CLAY (CH); moderate yellowish brown (10YR 5/4); moist; mostly fines; trace fine sand; high plasticity.
													LACUSTRINE DEPOSITS: Fat CLAY (CH); moderate yellowish brown (10YR 5/4); moist; mostly fines; trace fine sand; high plasticity.
	-135		B1					29.9		CR EI			
5			S2	4 5 8	13	18							Very stiff.
	-140		R3	4 5 9	14	13							Stiff clay; thinly interbedded with fine sand. PP = 1.25 tsf.
10			SH4							WA PI UC C			Lean CLAY (CL); stiff; moderate yellowish brown (10YR 5/4); wet; mostly fines; medium plasticity. (100% fines)
	-145												Fat CLAY (CH); medium stiff; medium yellowish brown (10YR 5/4); wet; mostly fines; trace fine sand; high plasticity. PP = 1.0 tsf.
15			S5	2 3 3	6	8							
	-150												SILTY SAND (SM); medium dense; dark yellowish orange (10YR 6/6); wet; mostly fine sand; some fines; nonplastic.
20			R6	8 12 11	23	21							
	-155												Total Depth = 21.5 feet (Target depth reached). Groundwater not measured - borehole caved shortly after drilling. Boring backfilled on 2/17/2023 shortly after drilling with bentonite chips and soil cuttings. This Boring Record is part of a geotechnical report which must be considered in its entirety. The exploration elevation was estimated using GoogleEarth Pro.

DRAFT

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San Diego, California 92126

FIGURE A-1

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
SDSU Brawley STEM Facility	SD725A	**B-2**

SITE LOCATION	START	FINISH	SHEET NO.
560 CA-78, Brawley, California	2/17/2023	2/17/2023	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Tri-County Drilling	Hollow Stem Auger	D. Guzman	C. Vonk

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUNDWATER (ft)
CME 75	8	21.5	-134	▼ 10.2 / *-144.2*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 82%, N_{60} = 1.36*N_{SPT} = 0.91*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	PID READINGS (ppm)	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	-135												**FILL:** Fat CLAY (CH); moderate yellowish brown (10YR 5/4); moist; mostly fines; trace fine sand; high plasticity. (98% Fines; 2% Sand)
			B1					22.6		PA PI EI			**LACUSTRINE DEPOSITS:** Fat CLAY (CH); moderate yellowish brown (10YR 5/4); moist; mostly fines; trace sand; high plasticity.
5	-140		S2	3 5 6	11	15		27.9		PA PI			Very stiff; scattered caliche nodules. (100% Fines) PP = 3.25 tsf.
			R3	5 4 8	12	11		25.9	94	PA PI			Dark yellowish orange (10YR 6/6); moist to wet. (100% Fines) PP = 2.5 tsf.
10	-145		S4	2 3 5	8	11						▼	Moderate yellowish brown (10YR 5/4); wet. PP = 3.75 tsf.
													Lean CLAY (CL); stiff; dark yellowish orange (10YR 6/6); wet; mostly fines; medium plasticity. (100% Fines)
15	-150		SH5							WA PI UC C			
													SILTY SAND (SM); dense; moderate yellowish brown (10YR 5/4); wet; some fines; mostly fine sand; nonplastic. (55% Sand; 45% Fines)
20	-155		R6	4 15 22	37	34		23.4	103	PA			
													Total Depth = 21.5 feet. Groundwater measured at a depth of 10.2 feet approx. 3 hours after completion of drilling. Boring backfilled on 2/17/2023 shortly after drilling with bentonite chips and soil cuttings. This Boring Record is part of a geotechnical report which must be considered in its entirety. The exploration elevation was estimated using GoogleEarth Pro.

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GDC_LOG_BORING_MMX_SOIL_SD SD725A LOGS.GPJ GDCLOG.GDT 3/22/23

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
SDSU Brawley STEM Facility	SD725A	**B-3**

SITE LOCATION	START	FINISH	SHEET NO.
560 CA-78, Brawley, California	2/17/2023	2/17/2023	1 of 3

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Tri-County Drilling	Hollow Stem Auger / Rotary Wash	D. Guzman	C. Vonk

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUNDWATER (ft)
CME 75	8 / 4	51.5	-136	▼ NM / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 82%, N_{60} = 1.36*N_{SPT} = 0.91*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	PID READINGS (ppm)	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
													FILL: Fat CLAY (CH); moderate yellowish brown (10YR 5/4); moist; mostly fines; trace fine sand; high plasticity.
													LACUSTRINE DEPOSITS: Fat CLAY (CH): moderate yellowish brown (10YR 5/4); moist; mostly fines; trace fine sand; high plasticity.
			B1					27.8		CR EI			
	-140												
5			R2	4 6 10	16	15		30.4	92				Very stiff. PP = 3.0 tsf.
	-145												
10			S3	1 2 3	5	7							Stiff; wet. PP = 1.75 tsf.
	-150												
15			R4	2 4 5	9	8							Medium stiff; medium to high plasticity.
	-155												
20			S5	5 8 13	21	29							SILTY SAND (SM); medium dense; moderate yellowish brown (10YR 5/4); wet; mostly fine sand; some fines; nonplastic.
	-160												

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BORING RECORD

PROJECT NAME	SDSU Brawley STEM Facility
PROJECT NUMBER	SD725A
BORING	**B-3**
SITE LOCATION	560 CA-78, Brawley, California
START	2/17/2023
FINISH	2/17/2023
SHEET NO.	2 of 3
DRILLING COMPANY	Tri-County Drilling
DRILLING METHOD	Hollow Stem Auger / Rotary Wash
LOGGED BY	D. Guzman
CHECKED BY	C. Vonk
DRILLING EQUIPMENT	CME 75
BORING DIA. (in)	8 / 4
TOTAL DEPTH (ft)	51.5
GROUND ELEV (ft)	-136
DEPTH/*ELEV.* GROUNDWATER (ft)	NM / *na*
SAMPLING METHOD	Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES	ETR ~ 82%, $N_{60} = 1.36*N_{SPT} = 0.91*N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	PID READINGS (ppm)	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			R6	6 15 16	31	28		23.7	102	PA			**LACUSTRINE DEPOSITS: (continued)** SILTY SAND (SM); medium dense; moderate yellowish brown (10YR 5/4); wet; mostly fine sand; some fines; nonplastic. (64% Sand; 36% Fines)
	-165												Fat CLAY (CH); stiff; moderate yellowish brown (10YR 5/4); wet; mostly fines; trace fine sand; high plasticity. (99% Fines)
30			SH7							WA PI C			
	-170												
35			S8	5 5 8	13	18							PP = 1.25 tsf.
	-175												Medium to high plasticity. PP = 1.75 tsf.
40			R9-1 R9-2	8 11 12	23	21		26.5	98	WA			SILT (ML); stiff; moderate yellowish brown (10YR 5/4); wet; mostly fines; some fine sand; low plasticity. (99% Fines)
	-180												Lean CLAY (CL); stiff; moderate yellowish brown (10YR 5/4); wet; mostly fines; trace fine sand; medium plasticity. (100% Fines)
45			S10	2 3 5	8	11				WA			
	-185												(See description on following page)

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

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
SDSU Brawley STEM Facility	SD725A	B-3

SITE LOCATION	START	FINISH	SHEET NO.
560 CA-78, Brawley, California	2/17/2023	2/17/2023	3 of 3

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Tri-County Drilling	Hollow Stem Auger / Rotary Wash	D. Guzman	C. Vonk

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUNDWATER (ft)
CME 75	8 / 4	51.5	-136	▼ NM / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 82%, $N_{60} = 1.36*N_{SPT} = 0.91*N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	PID READINGS (ppm)	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			R11	6 9 21	30	27		26.7	98				**<u>LACUSTRINE DEPOSITS: (continued)</u>** SILTY CLAY (CL-ML); stiff; moderate yellowish brown (10YR 5/4); mostly fines; little fine sand; low to medium plasticity.
	-190												Total Depth = 51.5 feet (Target depth reached). Groundwater not measured due to use of mud rotary drilling method. Boring backfilled on 2/17/2023 shortly after drilling with bentonite chips and cement grout. This Boring Record is part of a geotechnical report which must be considered in its entirety. The exploration elevation was estimated using GoogleEarth Pro.
55													
	-195												
60													
	-200												
65													
	-205												
70													
	-210												

GDC_LOG_BORING_MMX_SOIL_SD SD725A LOGS.GPJ GDCLOG.GDT 3/22/23

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San Diego, California 92126

FIGURE
A-3 c

Group Delta Consultants, Inc.
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San Diego, California 92126
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Project: **Project No. SD760, SDSU Brawley STEM Facility**
Location: **560 CA-78, Brawley, California**

CPT-6
Total depth: 50.54 ft

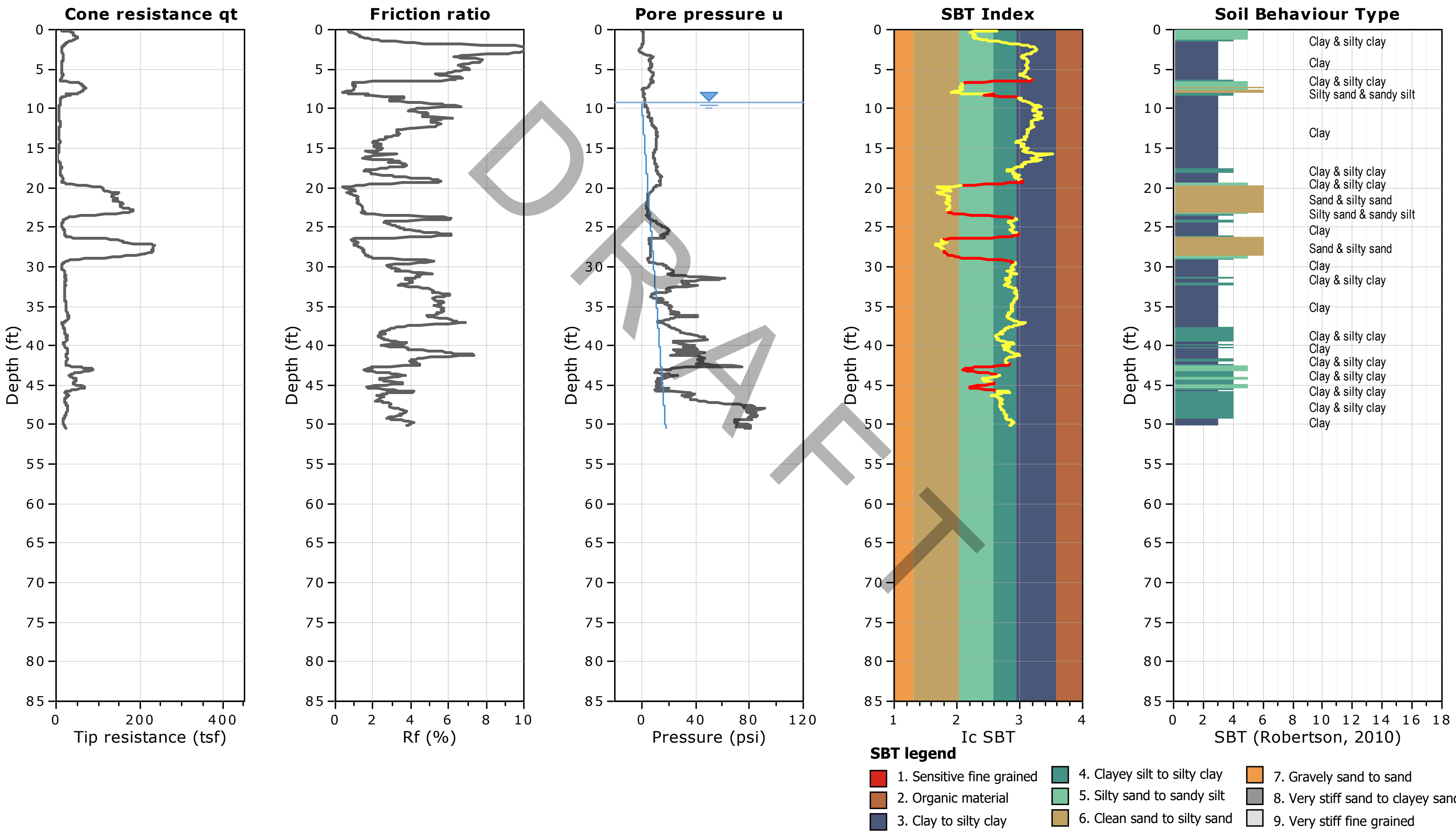

FIGURE A-4

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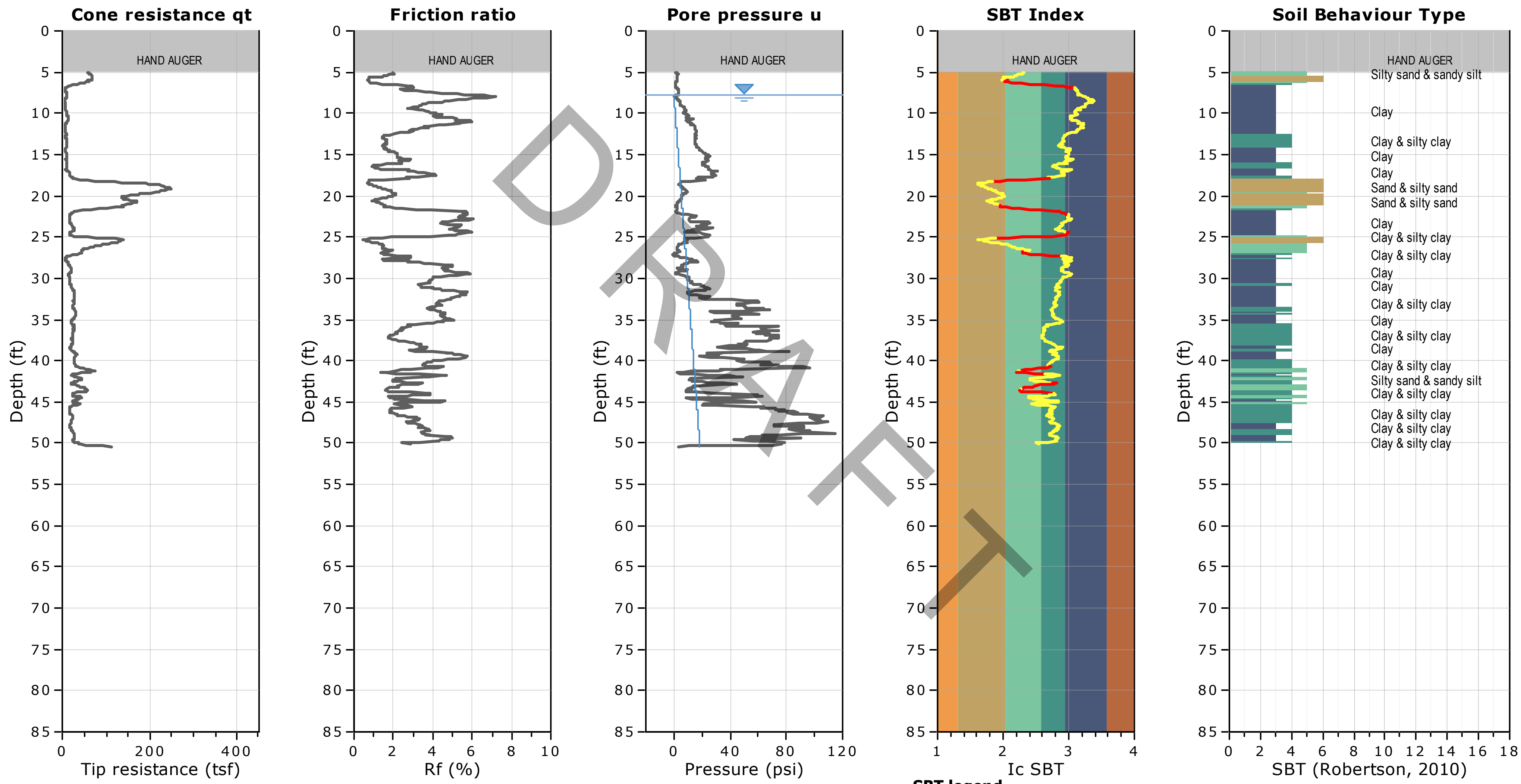
San Diego, California 92126
www.GroupDelta.com

Project: **Project No. SD760, SDSU Brawley STEM Facility**
Location: **560 CA-78, Brawley, California**

CPT-7
Total depth: 50.41 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

FIGURE A-5

Group Delta Consultants, Inc.
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San Diego, California 92126
www.GroupDelta.com

Project: Project No. SD760, SDSU Brawley STEM Facility
Location: 560 CA-78, Brawley, California

CPT-8
Total depth: 50.72 ft

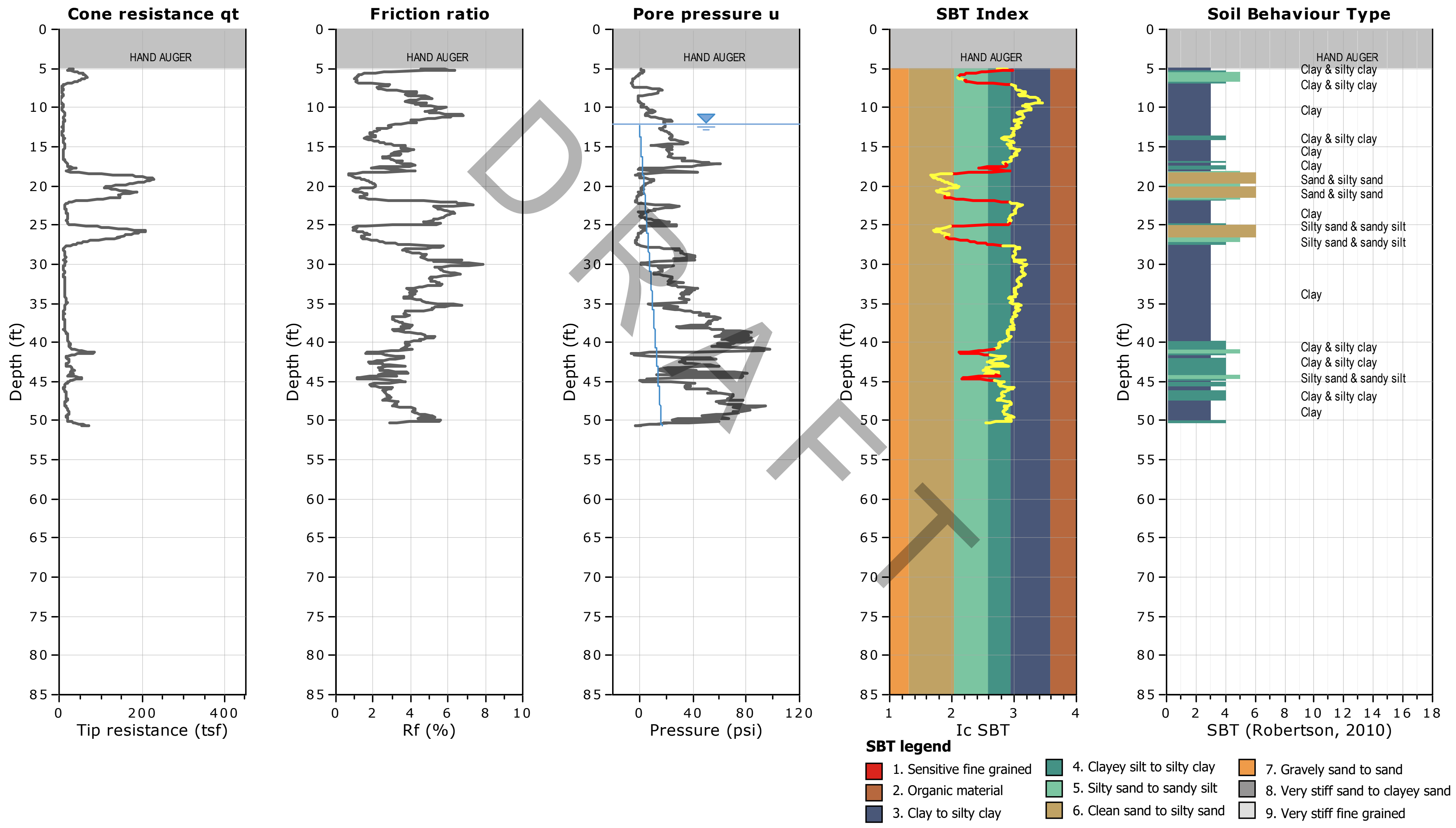

FIGURE A-6

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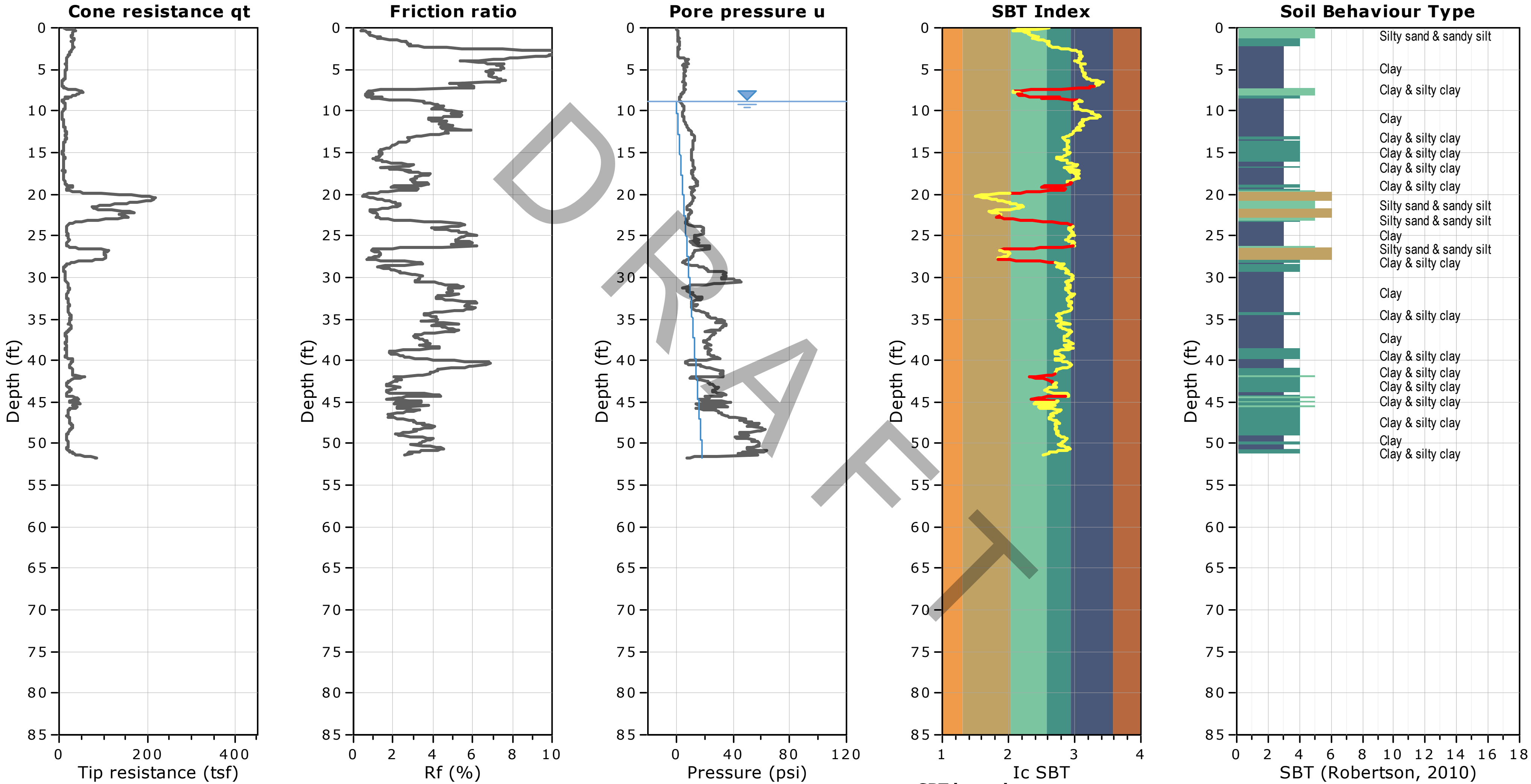
www.GroupDelta.com

Project: Project No. SD760, SDSU Brawley STEM Facility

Location: 560 CA-78, Brawley, California

CPT-9

Total depth: 51.72 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

FIGURE A-7

Group Delta Consultants, Inc.
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San Diego, California 92126
www.GroupDelta.com

Project: **Project No. SD760, SDSU Brawley STEM Facility**
Location: **560 CA-78, Brawley, California**

CPT-10
Total depth: 85.24 ft

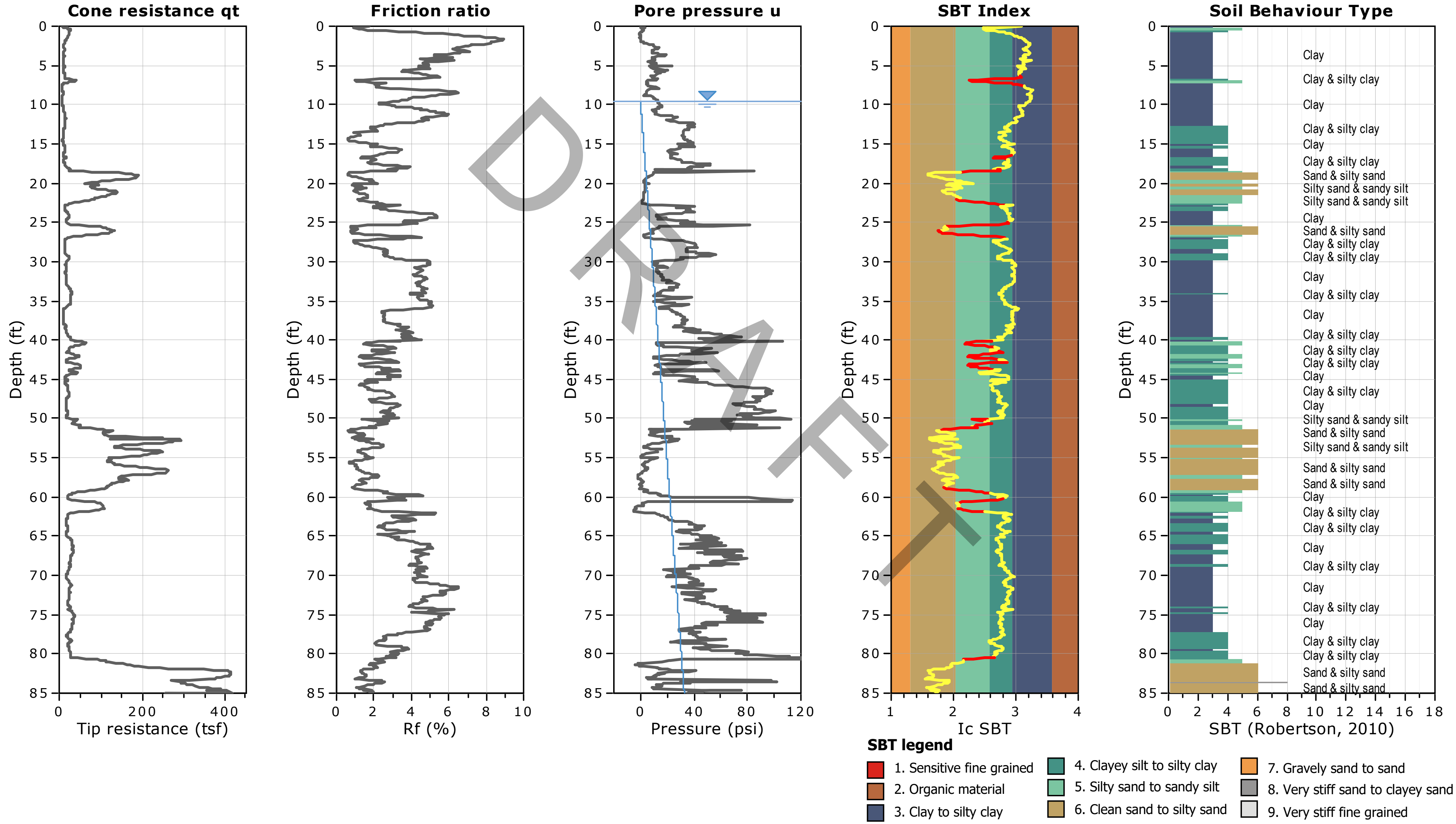


FIGURE A-8

Project No. SD760, SDSU Brawley STEM Facility
560 CA-78, Brawley, California

CPT Shear Wave Velocity Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-10	5.05	4.05	4.52	10.70	422	
	10.07	9.07	9.29	20.64	450	480
	15.03	14.03	14.17	34.92	406	342
	20.01	19.01	19.11	44.30	431	527
	25.03	24.03	24.11	51.60	467	685
	29.99	28.99	29.06	61.32	474	509
	35.04	34.04	34.10	72.88	468	436
	40.06	39.06	39.11	81.24	481	600
	45.01	44.01	44.06	88.20	499	710
	50.00	49.00	49.04	96.66	507	589
	55.02	54.02	54.06	103.30	523	755
	60.01	59.01	59.04	109.00	542	875
	65.06	64.06	64.09	116.80	549	647
	70.01	69.01	69.04	124.20	556	669
	75.03	74.03	74.06	131.24	564	713
	80.02	79.02	79.05	138.32	571	705
	85.01	84.01	84.03	142.60	589	1166

Shear Wave Source Offset - 2 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

FIGURE A-9

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 tests 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 D6913 and D1140 and were used to supplement visual classifications. The test results are summarized on the Boring Records in Appendix A and are presented in detail in Figures B-1 through B-2.

Atterberg Limits: ASTM D4318 was used to determine the liquid and plastic limits, and plasticity index of selected soil samples. The test results are presented with the associated gradation analyses in Figures B-1.1 through B-1.5 and are also summarized in Figure B-3.

Expansion Index: The expansion potential of selected soil samples was estimated in general accordance with ASTM D4829. The test results are summarized in Figure B-4. Figure B-4 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-5.

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-5, 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. The extracted solution was then tested for water soluble chloride using a calibrated ion specific electronic probe in general accordance with ASTM D512. The test results are also shown in Figure B-5.

APPENDIX B

LABORATORY TESTING (Continued)

Unconfined Compressive Strength: The undrained shear strength of two selected soil samples were assessed using unconfined compression testing performed in general accordance with ASTM D2166. The test results are presented in Figure B-6.1 and B-6.2. The Pocket Penetration tests conducted on clayey samples during the field investigation are shown in the Boring Records in Appendix A.

Consolidation: The one-dimensional consolidation properties of the selected samples were evaluated in general accordance with ASTM D2435. The samples were inundated with water under a nominal seating load, allowed to swell, and then subjected to controlled stress increments while restrained laterally and drained axially. The test results are presented in Figure B-7.1 through B-7.3.

COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

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

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

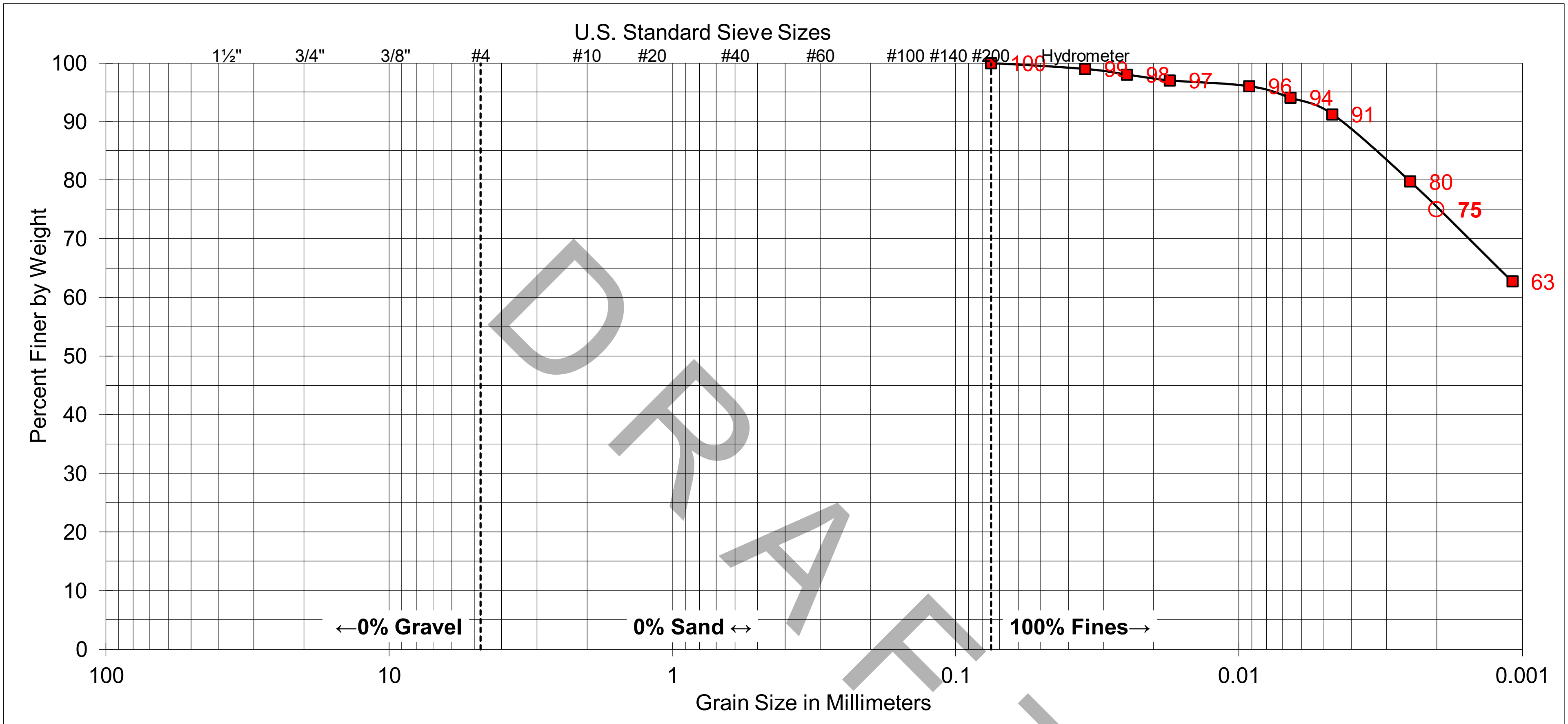
ATTERBERG LIMITS	
LIQUID LIMIT:	67
PLASTIC LIMIT:	23
PLASTICITY INDEX:	44

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD725A

FIGURE B-1.1



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
EXPLORATION ID:	B-2
SAMPLE DEPTH:	5' - 6.5'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

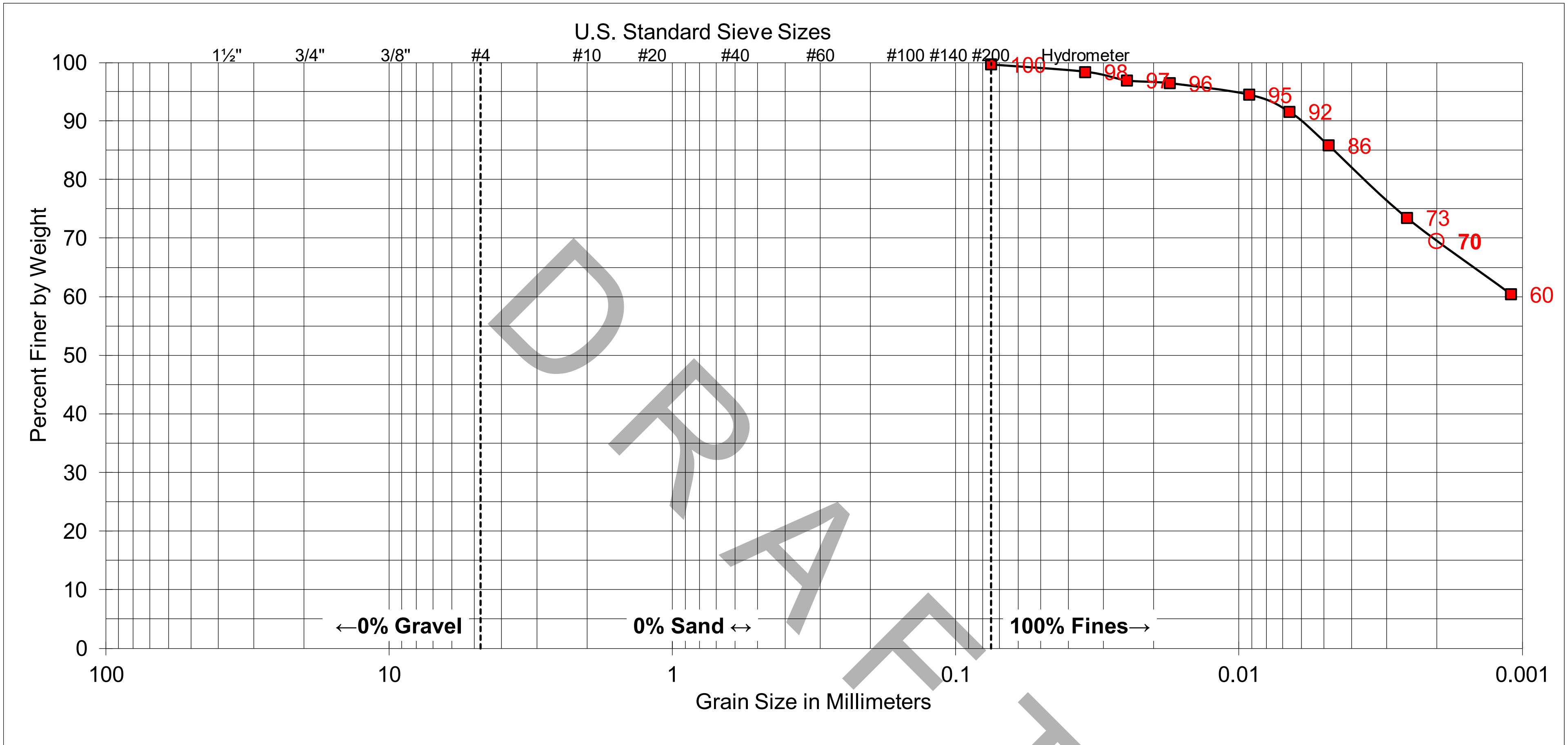
ATTERBERG LIMITS	
LIQUID LIMIT:	78
PLASTIC LIMIT:	26
PLASTICITY INDEX:	52

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD725A

FIGURE B-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	B-2
SAMPLE DEPTH:	7.5' - 9'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

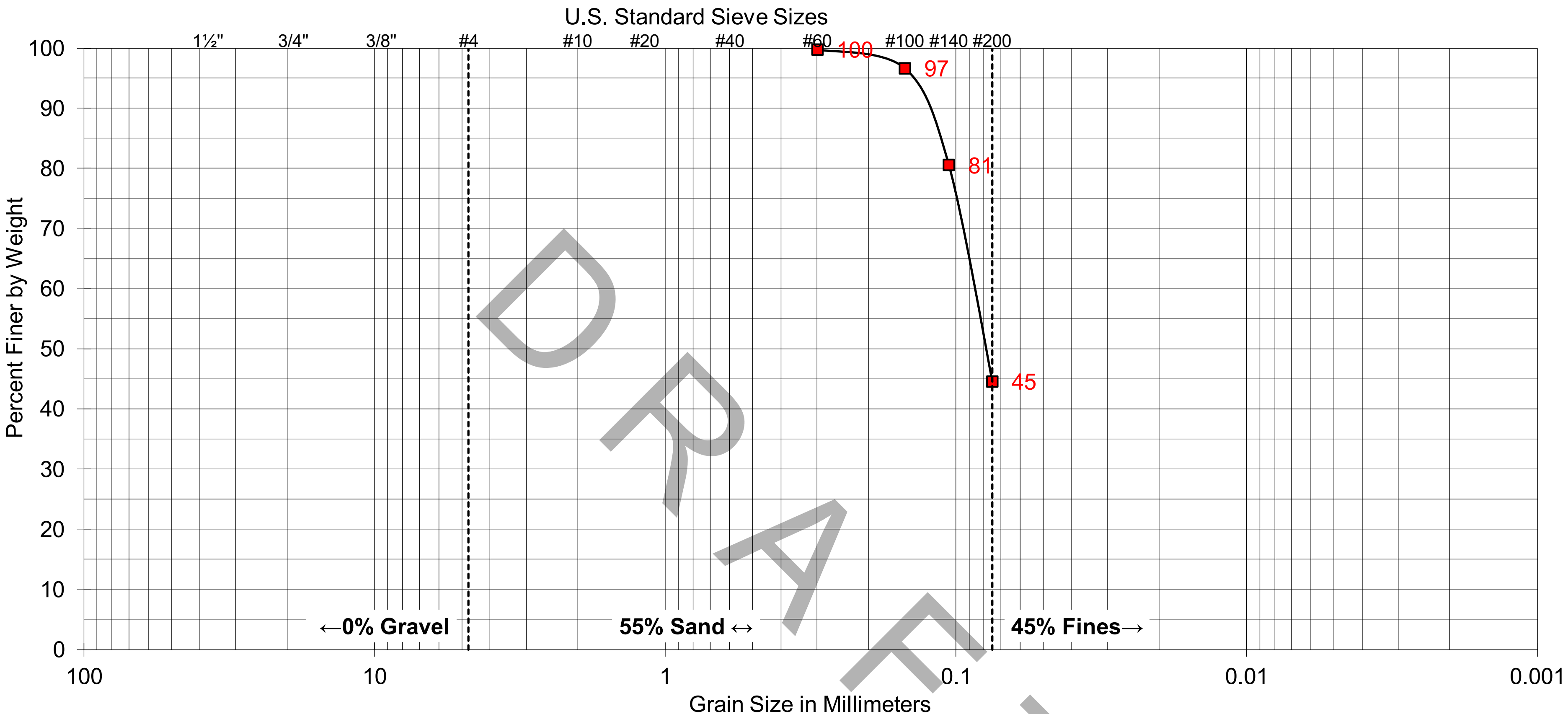
ATTERBERG LIMITS	
LIQUID LIMIT:	72
PLASTIC LIMIT:	26
PLASTICITY INDEX:	46

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD725A

FIGURE B-1.3



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	B-2
SAMPLE DEPTH:	20' - 21.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

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

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD725A

FIGURE B-1.4

COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	B-3
SAMPLE DEPTH:	25 - 26.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

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

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD725A

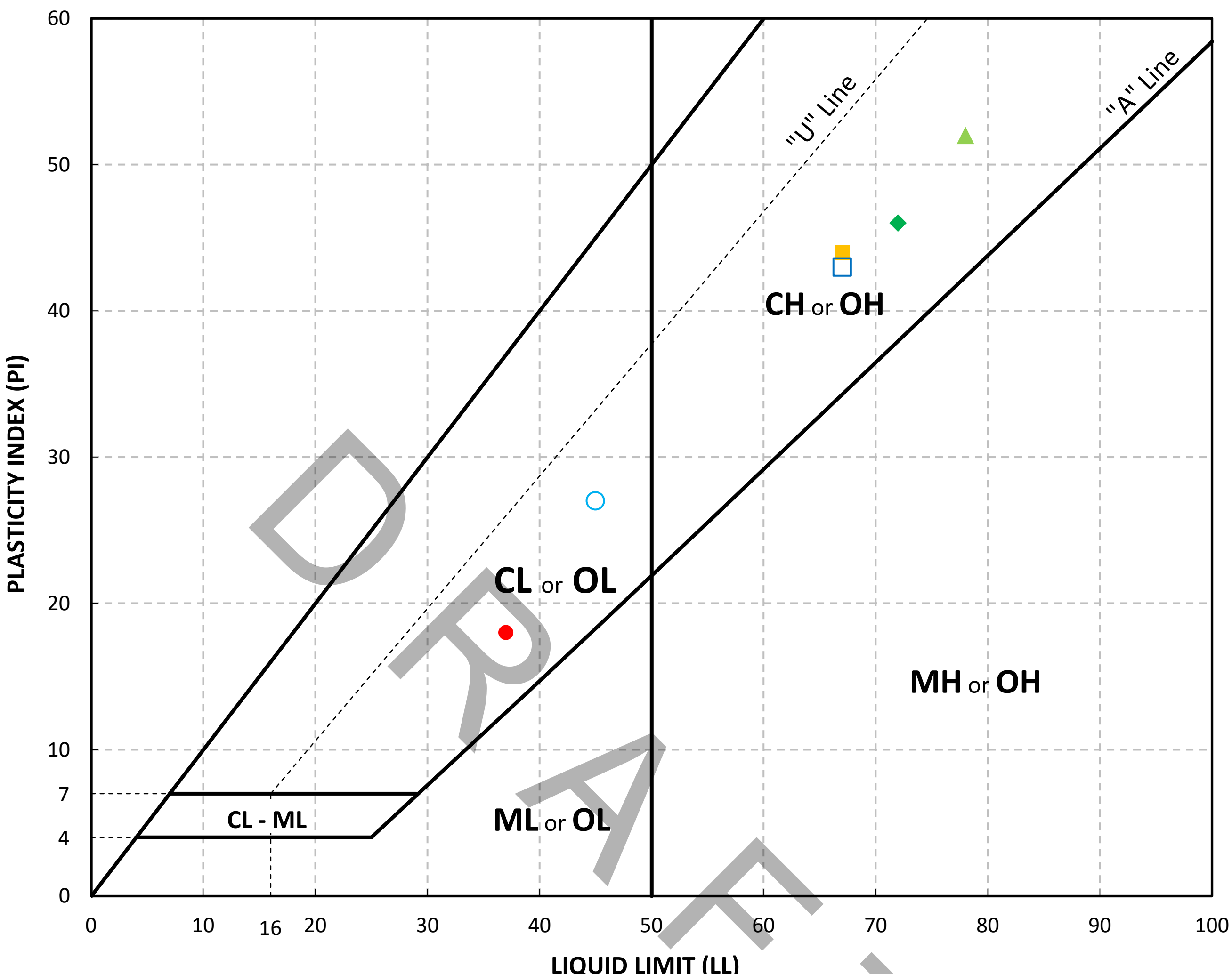
FIGURE B-1.5

PERCENT PASSING THE NO. 200 SIEVE

(ASTM D1140)

SAMPLE ID	DESCRIPTION	PERCENT PASSING THE NO. 200 (%)
B-1 @ 10’ – 12’	Lean CLAY (CL)	100
B-2 @ 15’ – 17’	Lean CLAY (CH)	100
B-3 @ 30’ – 32’	Fat CLAY (CL)	99
B-3 @ 41’ – 41.5’	SILT (ML)	99
B-3 @ 45’ – 46.5’	Lean CLAY (CL)	100

ATTERBERG LIMITS
(ASTM D4318)



SYMBOL	BORING NO.	SAMPLE NO.	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL DESCRIPTION (USCS)
●	B-1	10' - 12'	37	19	18	Lean CLAY (CL)
■	B-2	0' - 5'	67	23	44	Fat CLAY (CH)
▲	B-2	5' - 6.5'	78	26	52	Fat CLAY (CH)
◆	B-2	7.5' - 9'	72	26	46	Fat CLAY (CH)
○	B-2	15' - 17'	45	18	27	Lean CLAY (CL)
□	B-3	30' - 32'	67	24	43	Fat CLAY (CH)

Notes: (1) Unified Soil Classification System (USCS) per ASTM D2487
(2) NP = Non-Plastic per ASTM D4318

GROUP DELTA

LABORATORY TEST RESULTS

Project No. SD725A

FIGURE B-3

EXPANSION TEST RESULTS
(ASTM D4829)

SAMPLE ID	DESCRIPTION	EXPANSION INDEX
B-1 @ 0' – 5'	Fat CLAY (CH)	116
B-2 @ 0' – 5'	Fat CLAY (CH)	92
B-3 @ 0' – 5'	Fat CLAY (CH)	132

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 ID	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
B-1 @ 0' – 5'	7.83	262	0.98	0.11
B-3 @ 0' – 5'	7.82	295	1.43	0.06

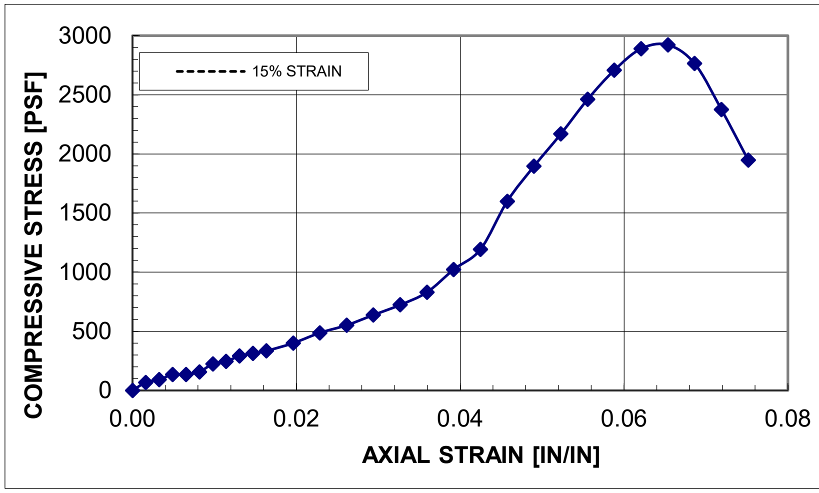
SULFATE CONTENT [%]	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	-
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

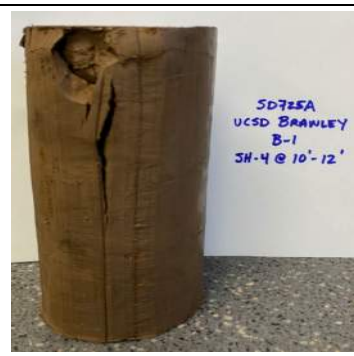
SOIL RESISTIVITY [OHM-CM]	GENERAL DEGREE OF CORROSIVITY TO FERROUS METALS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

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

PROJECT:	**SDSU Brawley STEM Facility**	TEST METHOD:	**ASTM D2166**
SAMPLE I.D.:	**B-1 @ 10' - 12'**	TESTED BY:	**J. Krehbiel**
DESCRIPTION:	**Lean CLAY (CL)**	DATE:	**3/2/23**

TYPE OF SAMPLE	Shelby	
WET WT. OF SAMPLE	1232.66	[g]
INITIAL DIAM.	2.845	[in]
INITIAL HEIGHT	6.121	[in]
INITIAL AREA	6.357	[in^2]
INITIAL VOLUME	38.91	[in^3]
WET DENSITY	120.7	[pcf]
DRY WT. OF SAMPLE	967.75	[g]
WEIGHT OF WATER	264.9	[g]
INITIAL TOTAL MOISTURE	27.4	[%]
DRY DENSITY	94.7	[pcf]
L-D RATIO	2.2:1	
STRAIN RATE	1.00	[%/min]
STRAIN AT FAILURE	6.53	[%]
STRAIN AT FAILURE	0.400	[in]
15% STRAIN	0.918	[in]
FAILURE CRITERIA:	Yield	
COMP. STRENGTH:	2922	[psf]
SHEAR STRENGTH:	1461	[psf]
SPEC. GRAVITY (Assumed)	2.79	
SATURATION:	91	[%]
FAILURE MODE:	Semi-Plastic	





SPECIMEN AFTER FAILURE

Elapsed Time [min]	**Axial Load** [lb]	**Strain Dial** [in]	**Total Deformation** [in]	**Axial Strain** [in/in]	**Corrected Area** [in^2]	**Stress** [psf]
0.0	0.0	1.000	0.000	0.000	6.36	0
0.1	3.0	0.990	0.010	0.002	6.37	68
0.3	4.0	0.980	0.020	0.003	6.38	90
0.7	6.0	0.960	0.040	0.007	6.40	135
0.8	7.0	0.950	0.050	0.008	6.41	157
1.0	10.0	0.940	0.060	0.010	6.42	224
1.4	14.0	0.910	0.090	0.015	6.45	312
1.6	15.0	0.900	0.100	0.016	6.46	334
1.9	18.0	0.880	0.120	0.020	6.48	400
2.3	22.0	0.860	0.140	0.023	6.51	487
2.6	25.0	0.840	0.160	0.026	6.53	551
2.9	29.0	0.820	0.180	0.029	6.55	638
3.2	33.0	0.800	0.200	0.033	6.57	723
3.5	38.0	0.780	0.220	0.036	6.59	830
3.8	47.0	0.760	0.240	0.039	6.62	1023
4.2	55.0	0.740	0.260	0.042	6.64	1193
4.6	74.0	0.720	0.280	0.046	6.66	1600
4.9	88.0	0.700	0.300	0.049	6.68	1896
5.1	101.0	0.680	0.320	0.052	6.71	2168
5.5	115.0	0.660	0.340	0.056	6.73	2460
5.8	127.0	0.640	0.360	0.059	6.75	2708
6.2	136.0	0.620	0.380	0.062	6.78	2889
6.5	138.0	0.600	0.400	0.065	6.80	2922
6.8	131.0	0.580	0.420	0.069	6.83	2764
7.1	113.0	0.560	0.440	0.072	6.85	2376
7.4	93.0	0.540	0.460	0.075	6.87	1948

GROUP DELTA

UNCONFINED COMPRESSIVE STRENGTH

Project No. SD725A

FIGURE B-6.1

PROJECT: **SDSU Brawley STEM Facility**

SAMPLE I.D.: **B-2 @ 15' - 17'**

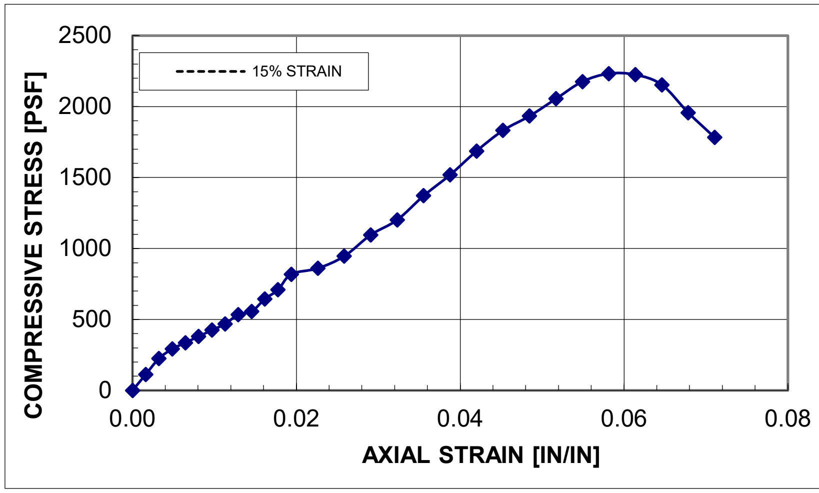
DESCRIPTION: **Lean CLAY (CL)**

TEST METHOD: **ASTM D2166**

TESTED BY: **J. Krehbiel**

DATE: **3/2/23**

TYPE OF SAMPLE	Shelby	
WET WT. OF SAMPLE	1248.68	[g]
INITIAL DIAM.	2.85	[in]
INITIAL HEIGHT	6.191	[in]
INITIAL AREA	6.379	[in^2]
INITIAL VOLUME	39.49	[in^3]
WET DENSITY	120.4	[pcf]
DRY WT. OF SAMPLE	998.57	[g]
WEIGHT OF WATER	250.1	[g]
INITIAL TOTAL MOISTURE	25.0	[%]
DRY DENSITY	96.3	[pcf]
L-D RATIO	2.2:1	
STRAIN RATE	0.99	[%/min]
STRAIN AT FAILURE	5.81	[%]
STRAIN AT FAILURE	0.360	[in]
15% STRAIN	0.929	[in]
FAILURE CRITERIA:	Yield	
COMP. STRENGTH:	2232	[psf]
SHEAR STRENGTH:	1116	[psf]
SPEC. GRAVITY (Assumed)	2.8	
SATURATION:	86	[%]
FAILURE MODE:	semi-plastic	



SPECIMEN AFTER FAILURE

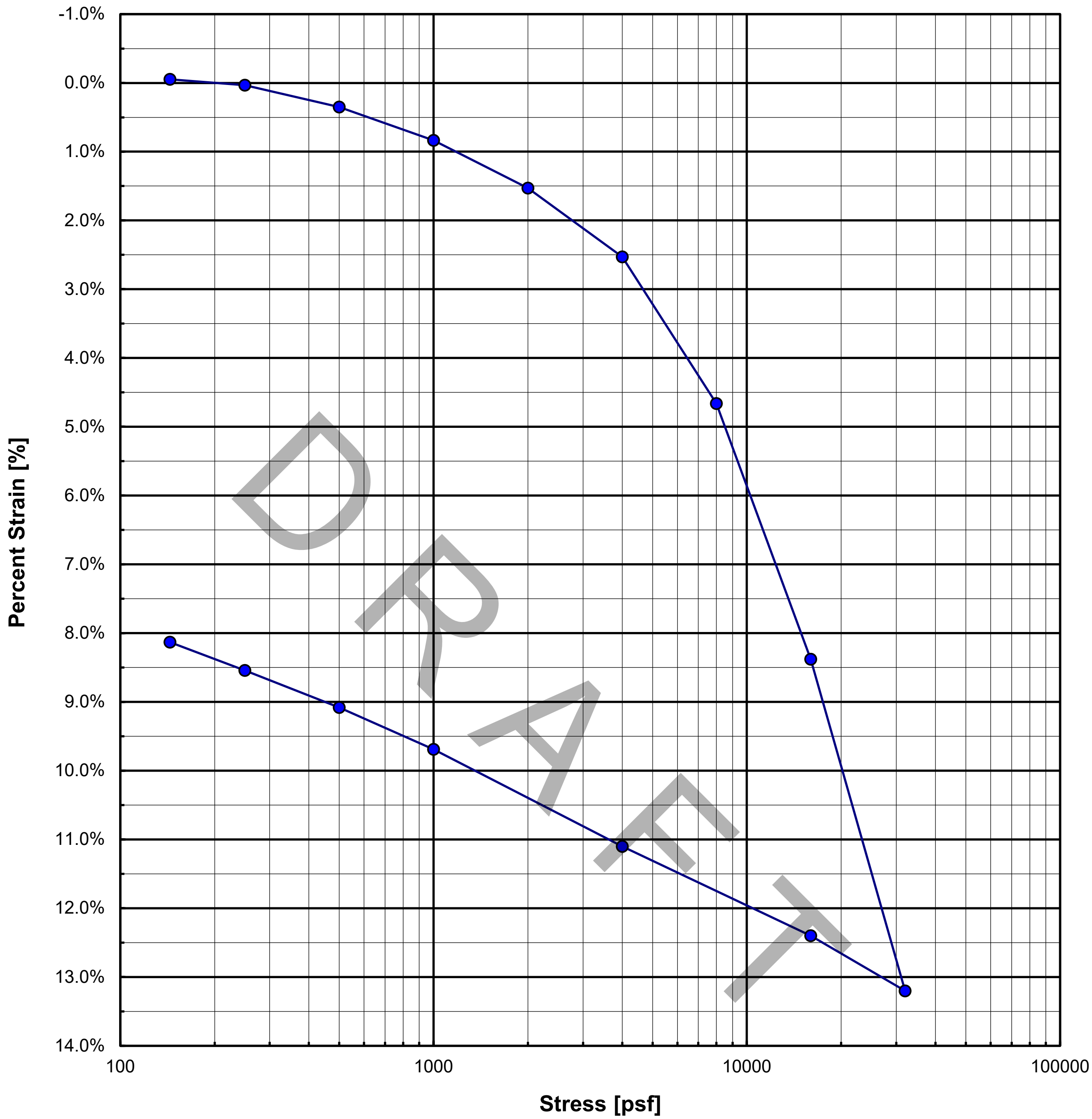
Elapsed Time [min]	Axial Load [lb]	Strain Dial [in]	Total Deformation [in]	Axial Strain [in/in]	Corrected Area [in^2]	Stress [psf]
0.0	0.0	1.000	0.000	0.000	6.38	0.0
0.2	5.0	0.990	0.010	0.002	6.39	112.7
0.3	10.0	0.980	0.020	0.003	6.40	225.0
0.7	15.0	0.960	0.040	0.006	6.42	336.4
0.8	17.0	0.950	0.050	0.008	6.43	380.6
1.0	19.0	0.940	0.060	0.010	6.44	424.7
1.5	25.0	0.910	0.090	0.015	6.47	556.1
1.7	29.0	0.900	0.100	0.016	6.48	644.0
1.8	32.0	0.890	0.110	0.018	6.49	709.5
2.1	37.0	0.880	0.120	0.019	6.51	819.0
2.3	39.0	0.860	0.140	0.023	6.53	860.4
2.6	43.0	0.840	0.160	0.026	6.55	945.5
2.9	50.0	0.820	0.180	0.029	6.57	1095.8
3.2	55.0	0.800	0.200	0.032	6.59	1201.4
3.6	63.0	0.780	0.220	0.036	6.61	1371.5
3.9	70.0	0.760	0.240	0.039	6.64	1518.8
4.3	78.0	0.740	0.260	0.042	6.66	1686.7
4.6	85.0	0.720	0.280	0.045	6.68	1831.9
4.9	90.0	0.700	0.300	0.048	6.70	1933.1
5.2	96.0	0.680	0.320	0.052	6.73	2055.0
5.6	102.0	0.660	0.340	0.055	6.75	2176.0
5.9	105.0	0.640	0.360	0.058	6.77	2232.3
6.2	105.0	0.620	0.380	0.061	6.80	2224.7
6.5	102.0	0.600	0.400	0.065	6.82	2153.7
6.8	93.0	0.580	0.420	0.068	6.84	1956.8
7.1	85.0	0.560	0.440	0.071	6.87	1782.3

GROUP DELTA

UNCONFINED COMPRESSIVE STRENGTH

Project No. SD725A

FIGURE B-6.2



SAMPLE ID: B-1 @ 10' - 12' **SOIL TYPE: Lean CLAY (CL)**

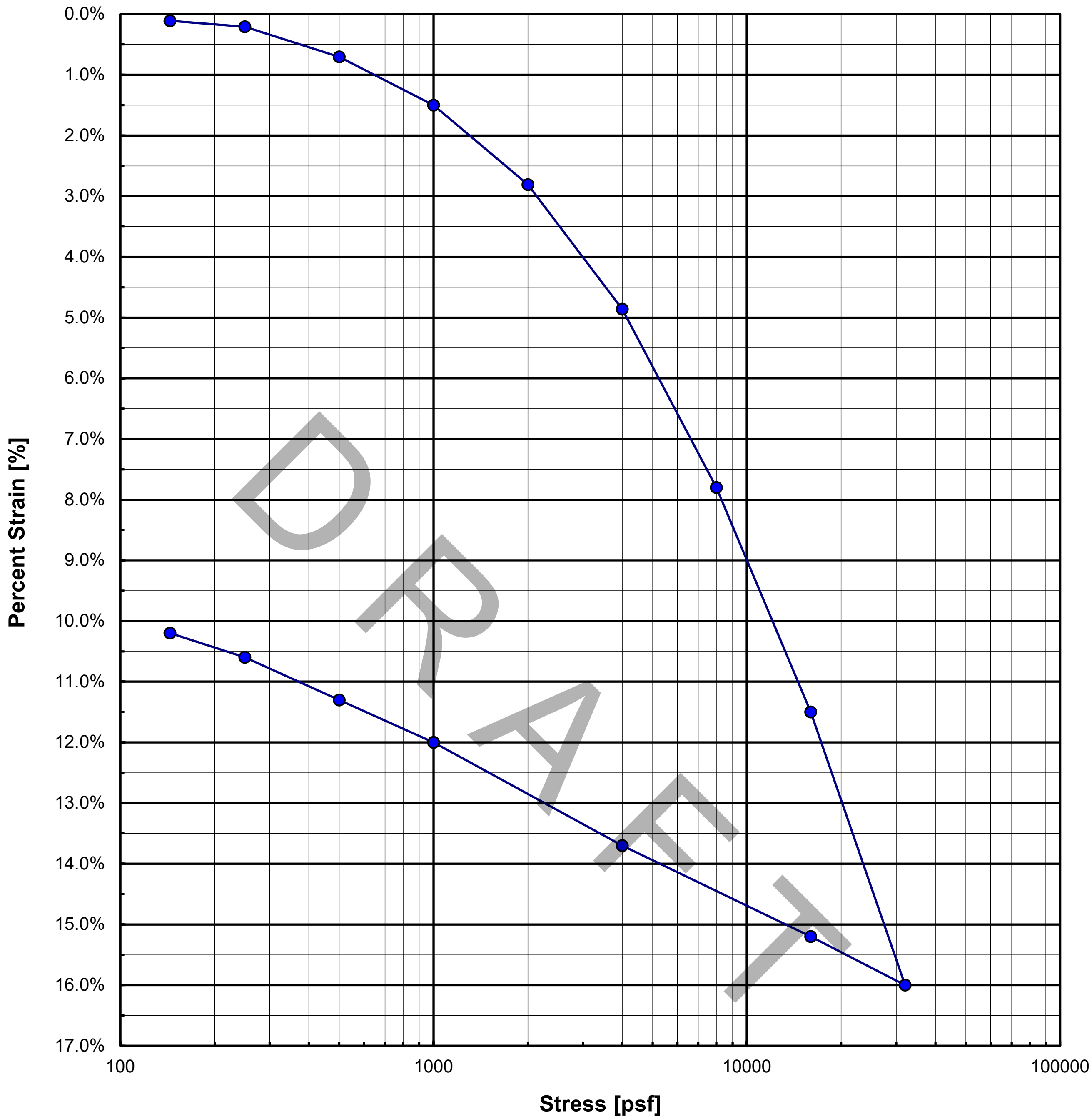
INITIAL	FINAL	
1.0000	0.9187	SAMPLE HEIGHT [IN]
95.6	104.1	DRY DENSITY [PCF]
2.79	2.79	SPECIFIC GRAVITY (ASSUMED)
0.82	0.67	VOID RATIO (e)
27.6	24.1	WATER CONTENT [%]
93.6	100.0	DEGREE OF SATURATION [%]

GROUP DELTA

CONSOLIDATION RESULTS

Project No. SD725A

FIGURE B-7.1



SAMPLE ID: B-2 @ 15' - 17' **SOIL TYPE: Lean CLAY (CL)**

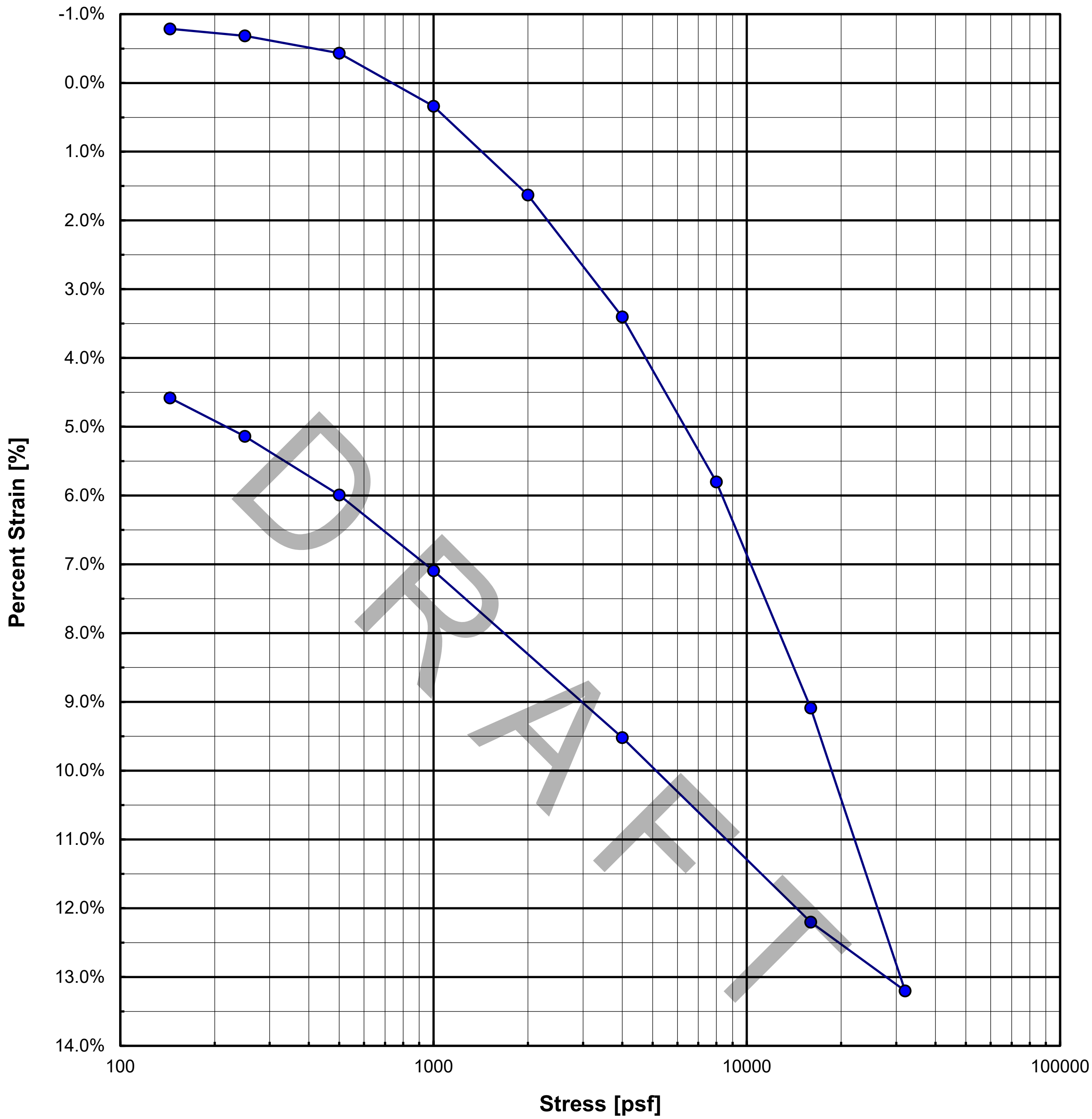
INITIAL	FINAL	
1.0000	0.8980	SAMPLE HEIGHT [IN]
94.0	104.7	DRY DENSITY [PCF]
2.80	2.80	SPECIFIC GRAVITY (ASSUMED)
0.86	0.67	VOID RATIO (e)
28.2	23.9	WATER CONTENT [%]
92.2	100.0	DEGREE OF SATURATION [%]

GROUP DELTA

CONSOLIDATION RESULTS

Project No. SD725A

FIGURE B-7.2



SAMPLE ID: B-3 @ 30' - 32' **SOIL TYPE: Fat CLAY (CH)**

INITIAL	FINAL	
1.0000	0.9542	SAMPLE HEIGHT [IN]
93.3	97.7	DRY DENSITY [PCF]
2.85	2.85	SPECIFIC GRAVITY (ASSUMED)
0.92	0.82	VOID RATIO (e)
29.7	28.8	WATER CONTENT [%]
91.5	100.0	DEGREE OF SATURATION [%]

GROUP DELTA

CONSOLIDATION RESULTS

Project No. SD725A

FIGURE B-7.3

APPENDIX C
DATA FROM PRIOR GEOTECHNICAL STUDY (GROUP DELTA, 2022)

APPENDIX C

DATA FROM PRIOR GEOTECHNICAL STUDY (GROUP DELTA, 2022)

Subsurface data from the project site and the surrounding area was compiled from Group Delta's prior geotechnical investigation to the southwest of the SDSU Brawley STEM Facility project (Group Delta, 2022). The locations of the exploration records included in Appendix C are shown on Figure 2, Exploration Locations.

FIELD EXPLORATION

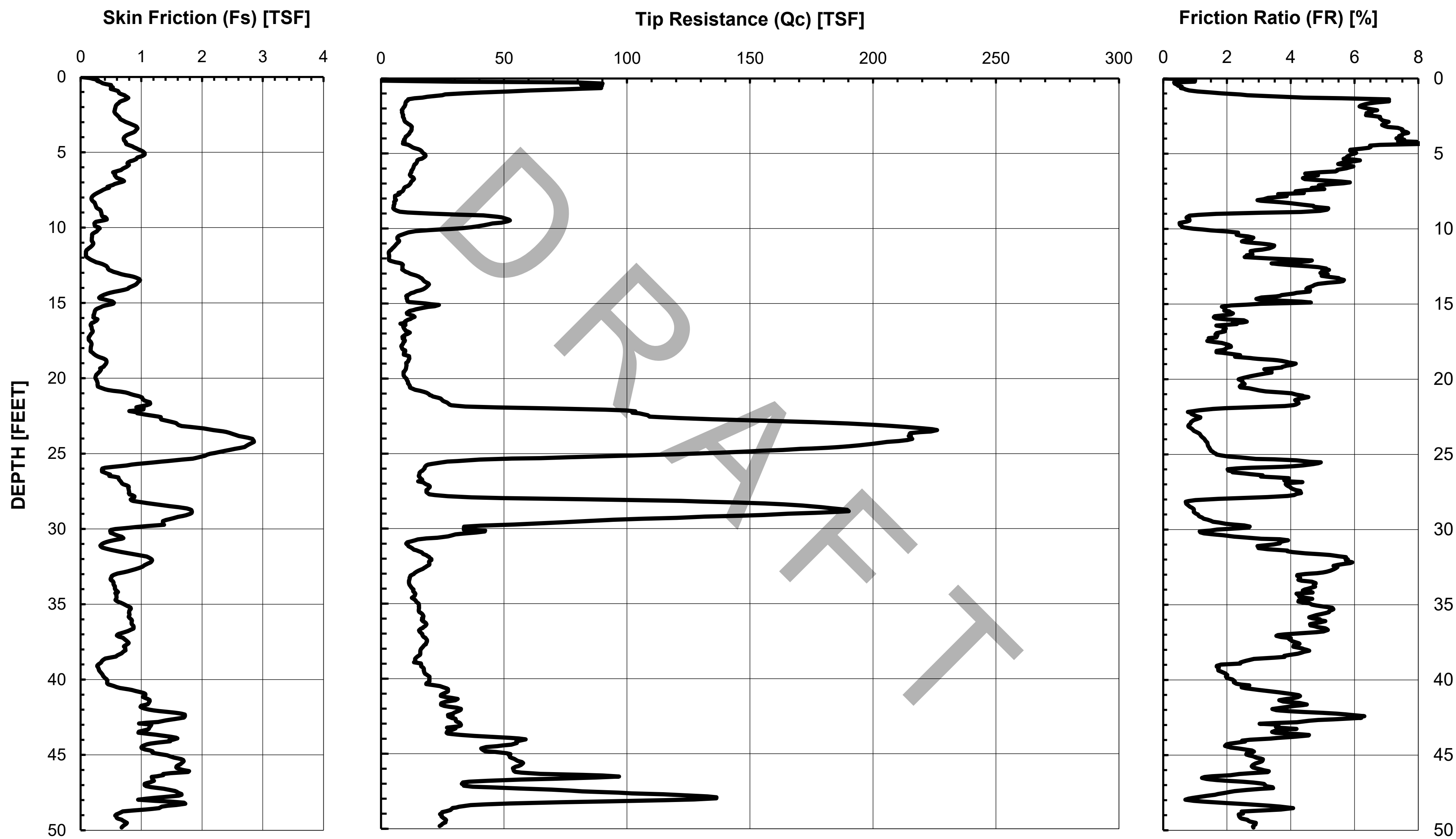
FIELD EXPLORATION

The field exploration included a visual and geologic reconnaissance of the site, and the advancement of five cone penetration test soundings (CPT) on March 22nd, 2022. Bulk soil samples were collected from the upper 4-feet of existing soil at each CPT sounding location. The CPT soundings were advanced by Kehoe Testing and Engineering to a maximum depth of about 88½ feet below surrounding grades. The approximate CPT locations are shown on the Exploration Plan, Figure 3. The CPT soundings and interpreted soil profiles are shown in Figures A-1 through A-5.

The CPT soundings were advanced using a 30-ton rig with a 15 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). Figure A for each CPT sounding presents the raw data. The CPT data may also be used to estimate soil parameters such as undrained shear strength, as shown Figure B for each CPT sounding. The interpretations are based on the normalized cone resistance and friction ratio (Robertson, 2010).

At the location of CPT-5, shear wave velocity measurements were also taken at 5-foot depth intervals using an air actuated hammer located inside the front jack of the rig. The raw interval shear wave data is attached immediately after the interpreted soil profile for CPT-5 at the end of Appendix A. The average shear wave velocity measured within the upper 88½ feet (Vs_d) at the location of CPT-5 was 585 ft/s or 178 m/s. Based on a commonly used extrapolation method, the average shear wave velocity in the upper 100 feet of the soil profile (Vs_{30}) is estimated at 610 ft/s or 186 m/s (Boore, 2004). This corresponds to a 2019 California Building Code (CBC) seismic Site Class D (Stiff Soil) with respect to seismic design of the planned short-period structure at this site.

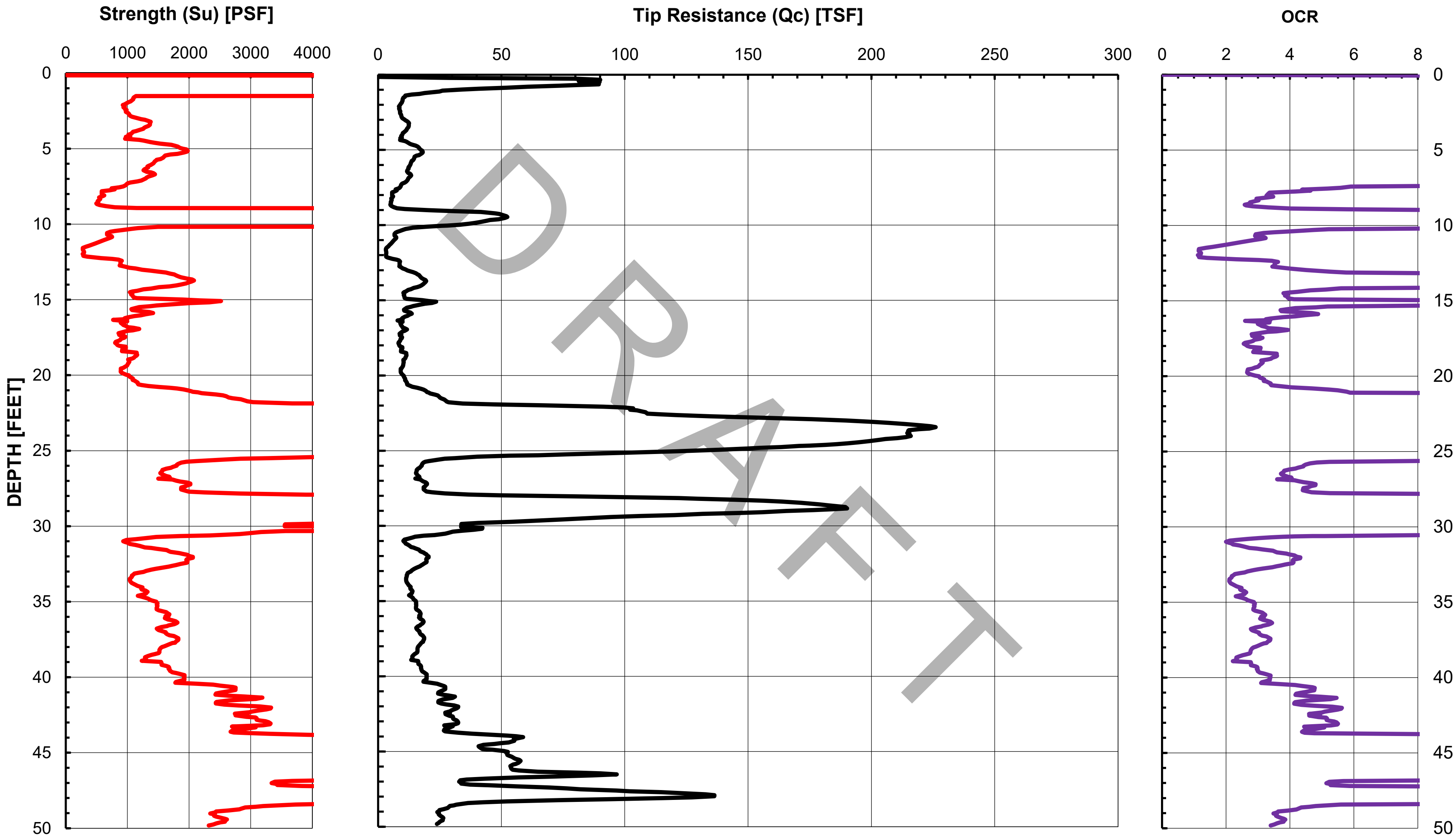
The CPT locations were determined by visually estimating, pacing and taping distances from landmarks shown on the Exploration Plans. The locations shown should not be considered more accurate than is implied by the method of measurement used and the scale of the map. The lines designating the interface 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.



GROUP DELTA

CONE PENETOMETER DATA (CPT-1)

Document No. 22-0028
Project No. SD725
FIGURE A-1a



GROUP DELTA

ESTIMATED STRENGTH AND OCR (CPT-1)

Document No. 22-0028

Project No. SD725

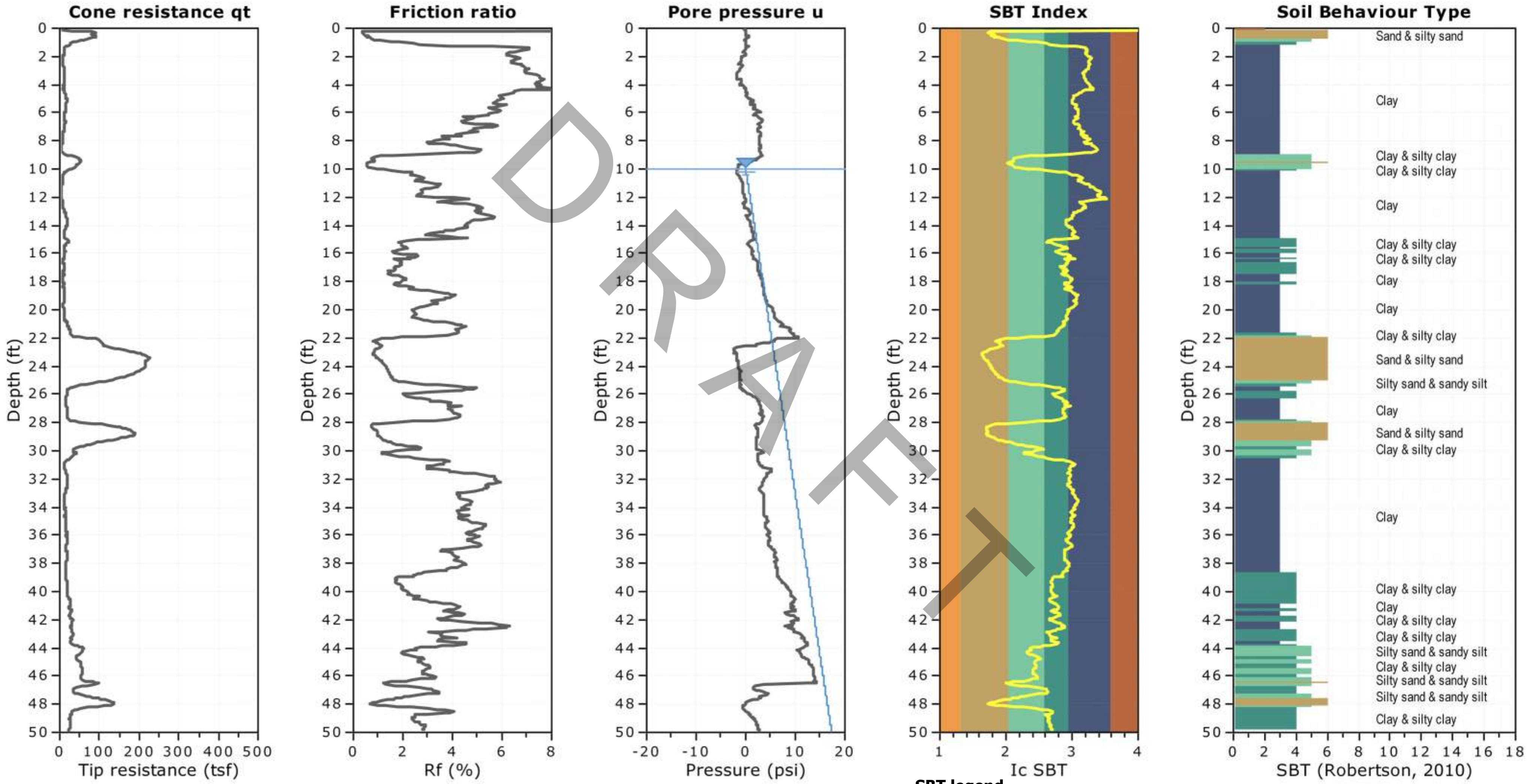
FIGURE A-1b

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

CPT-1

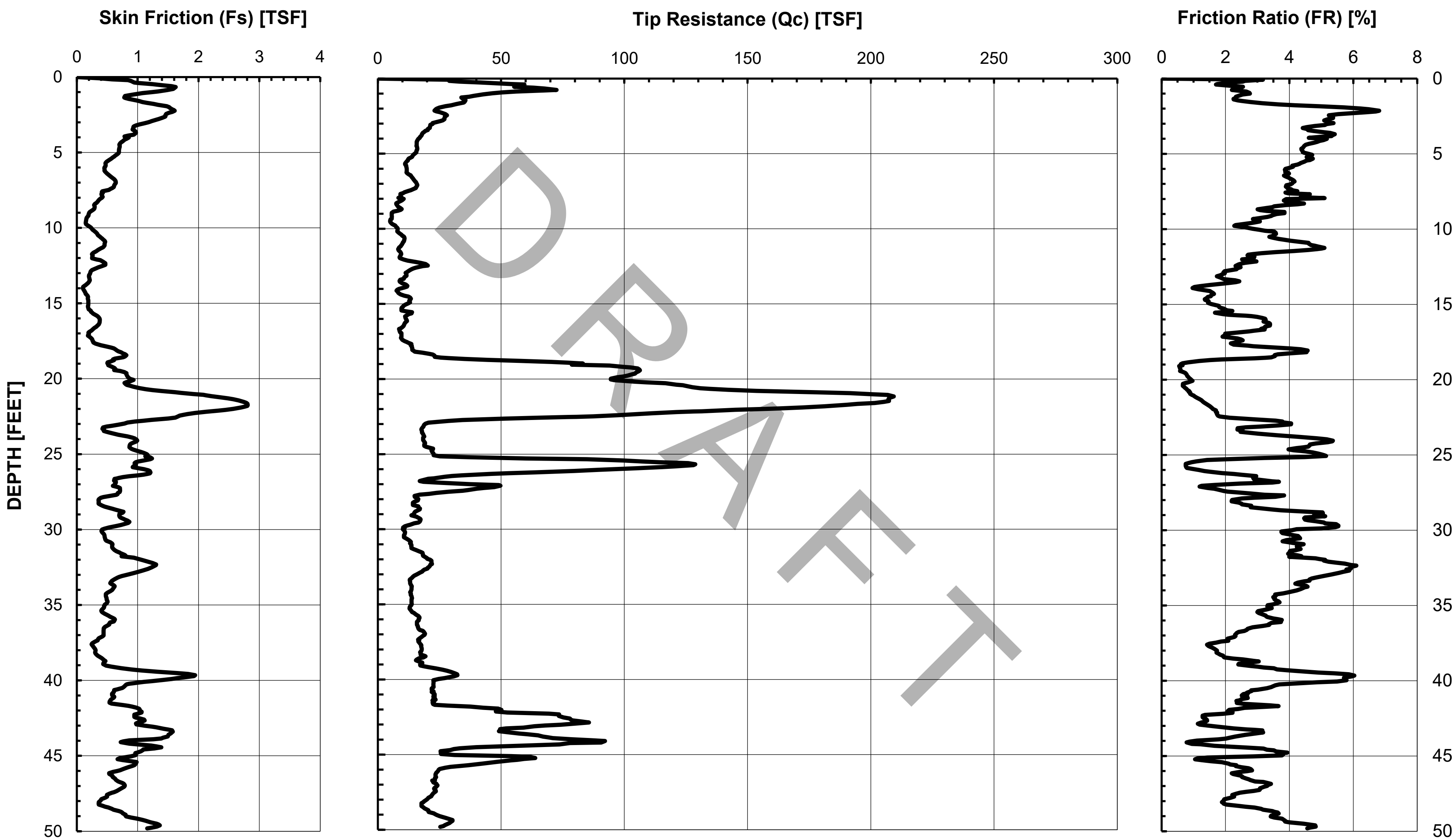
Project: **Brawley Science Center Addition**
Location: **San Diego State University, Brawley Campus**

Total depth: 50.21 ft, Date: 3/22/2022
Surface Elevation: -131.00 ft



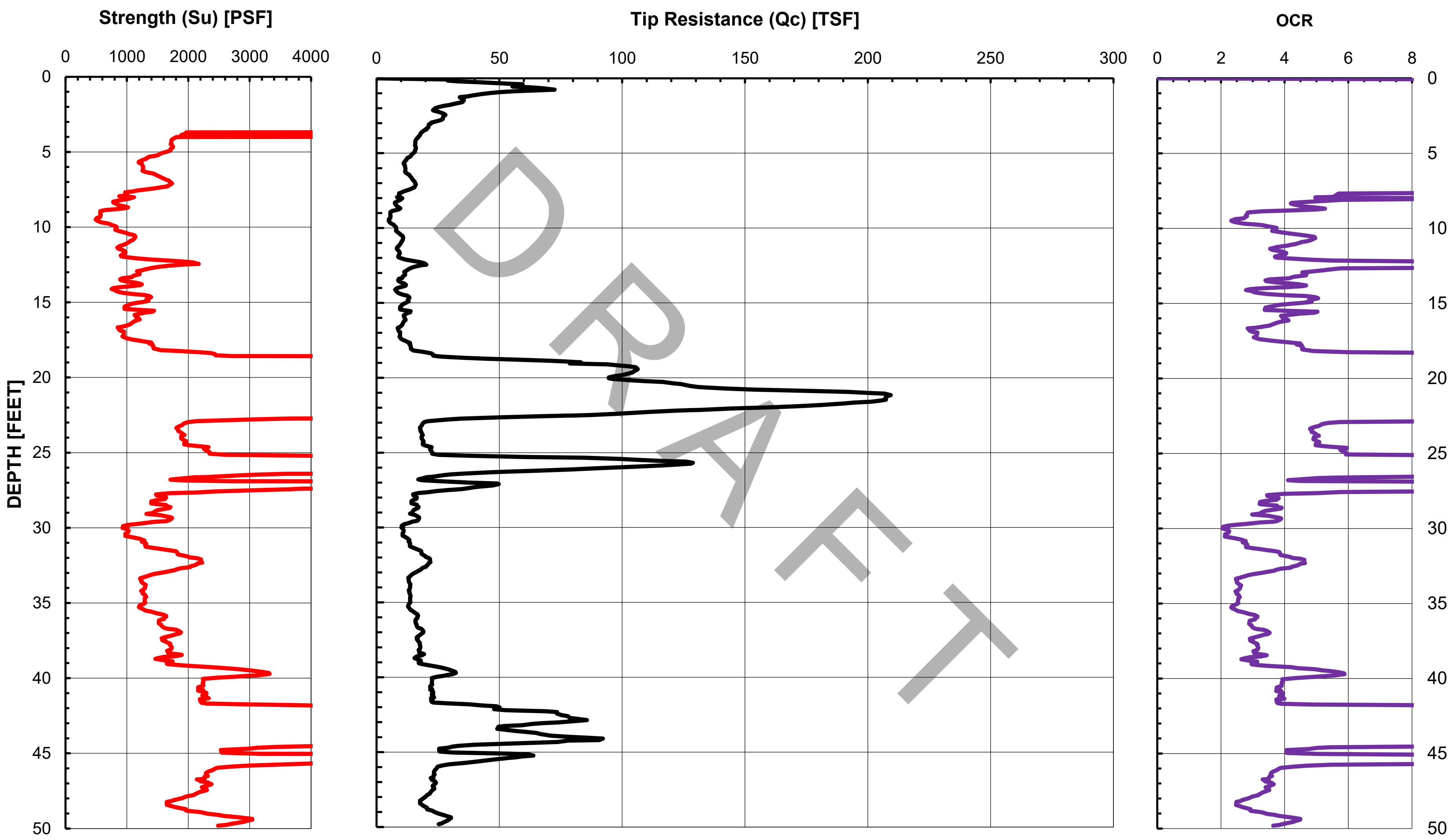
SBT legend

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



CONE PENETOMETER DATA (CPT-2)

Document No. 22-0028
Project No. SD725
FIGURE A-2a



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ESTIMATED STRENGTH AND OCR (CPT-2)

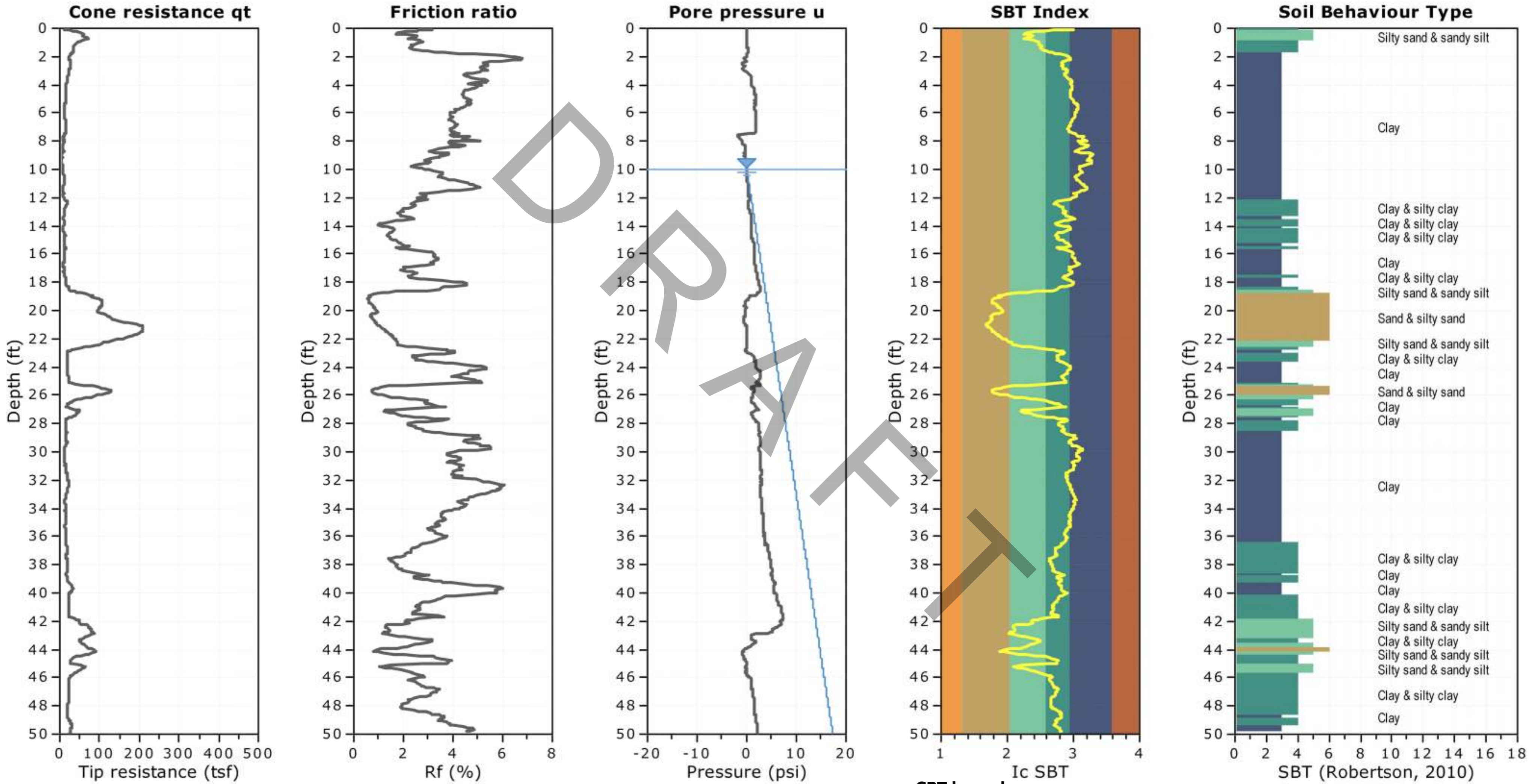
Document No. 22-0028
Project No. SD725
FIGURE A-2b

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

CPT-2

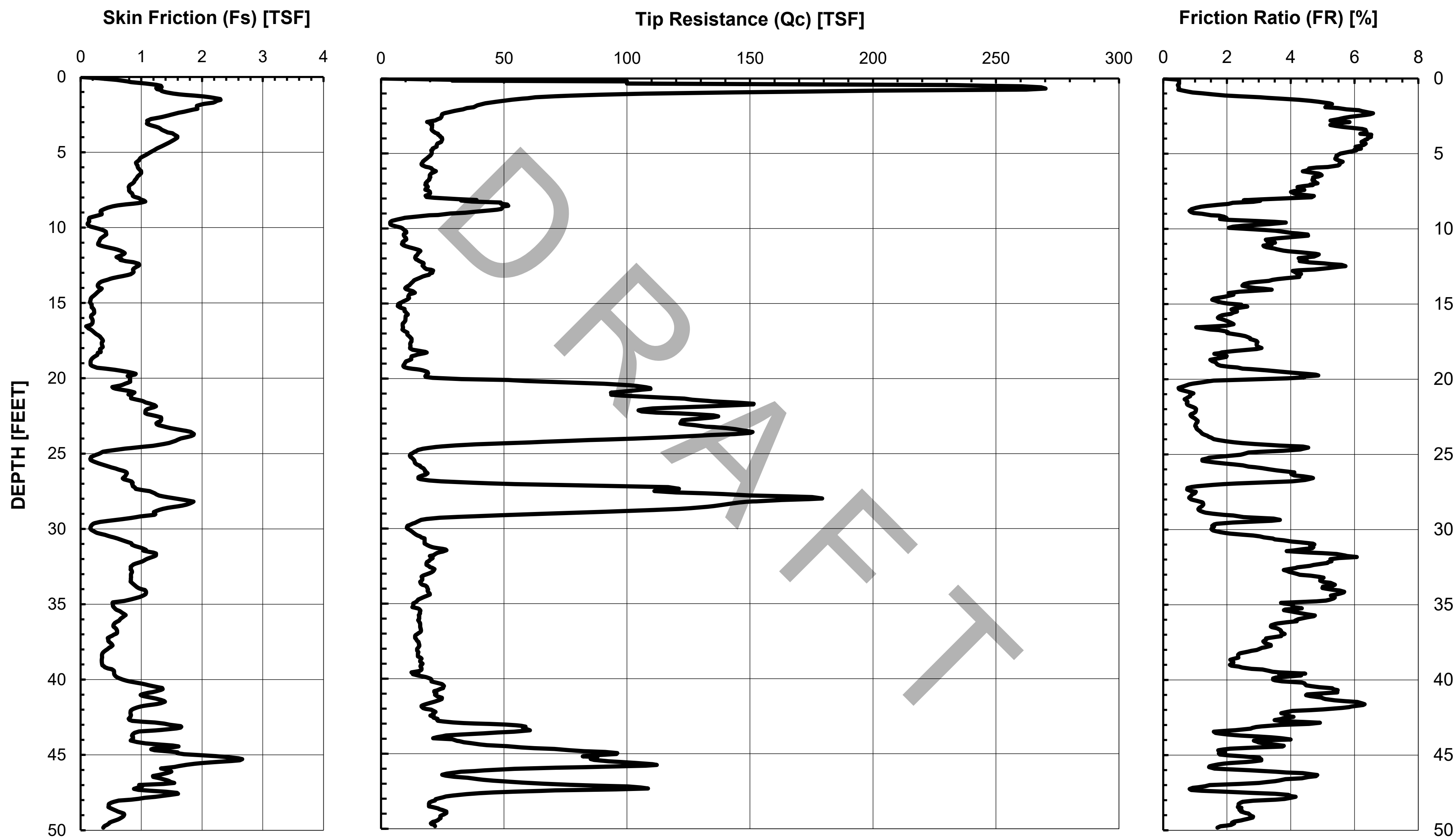
Project: **Brawley Science Center Addition**
Location: **San Diego State University, Brawley Campus**

Total depth: 50.21 ft, Date: 3/22/2022
Surface Elevation: -134.00 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



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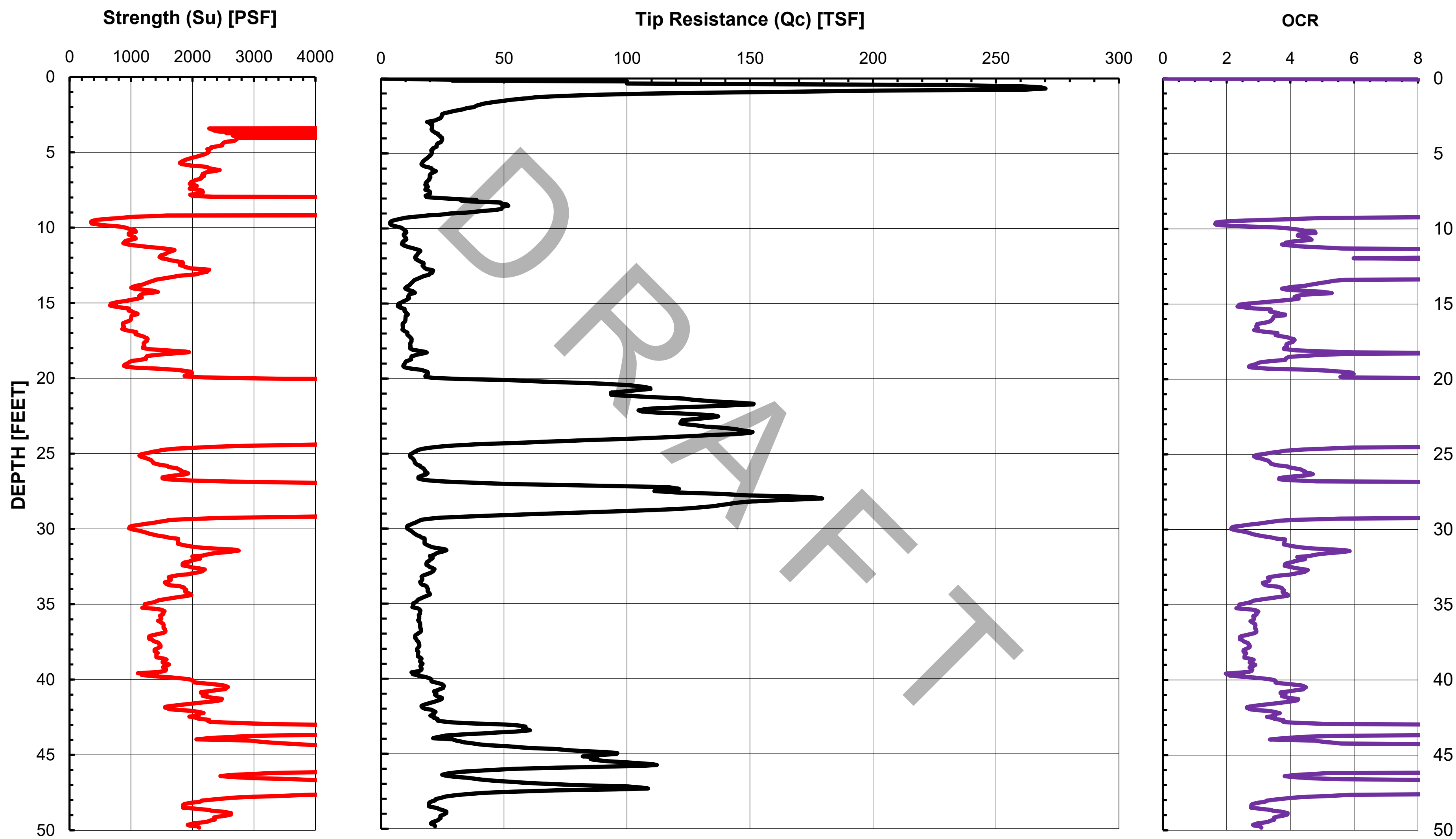
CONE PENETOMETER DATA (CPT-3)

Document No. 22-0028
Project No. SD725
FIGURE A-3a



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ESTIMATED STRENGTH AND OCR (CPT-3)

Document No. 22-0028
Project No. SD725
FIGURE A-3b

Group Delta Consultants
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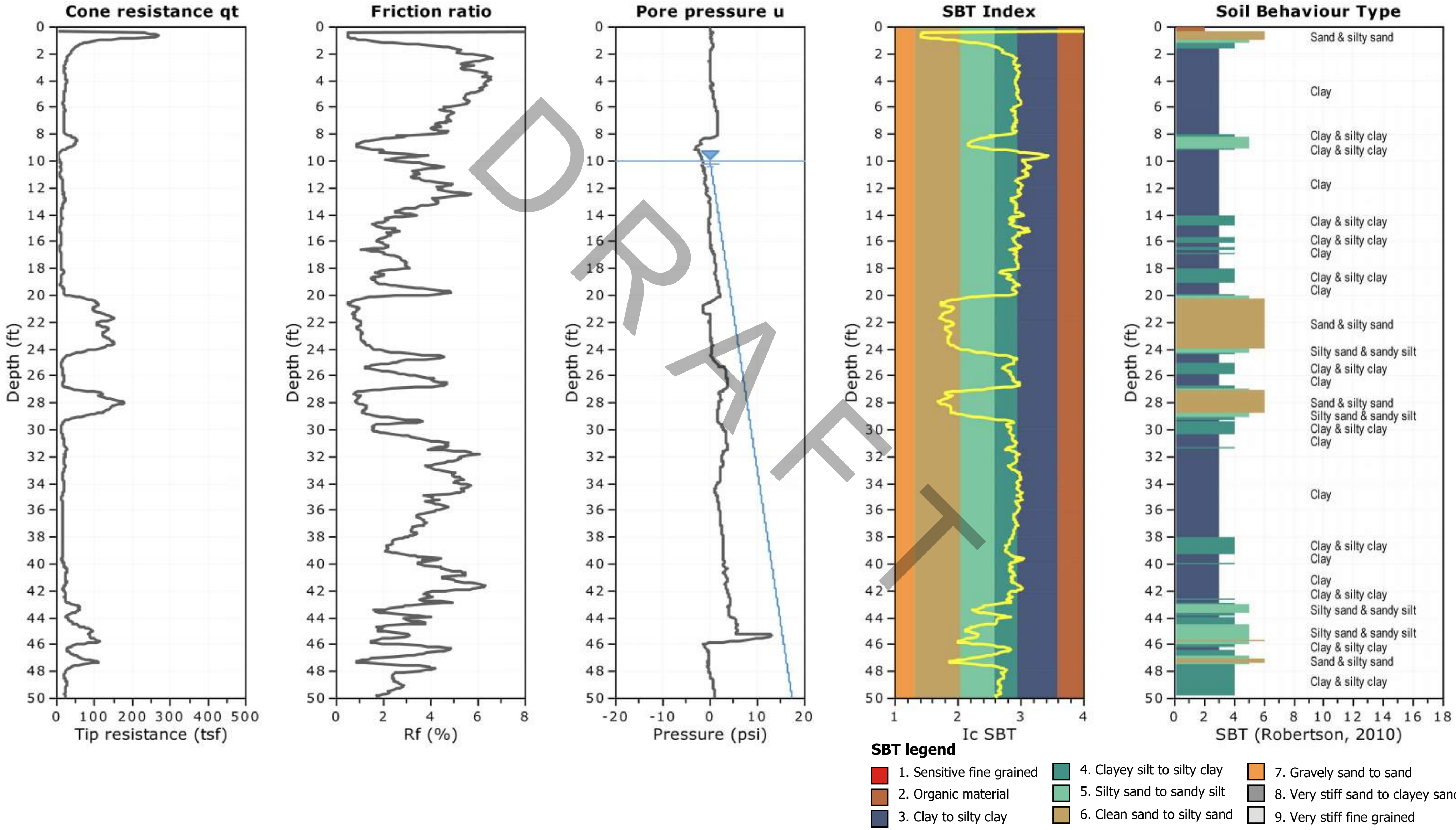
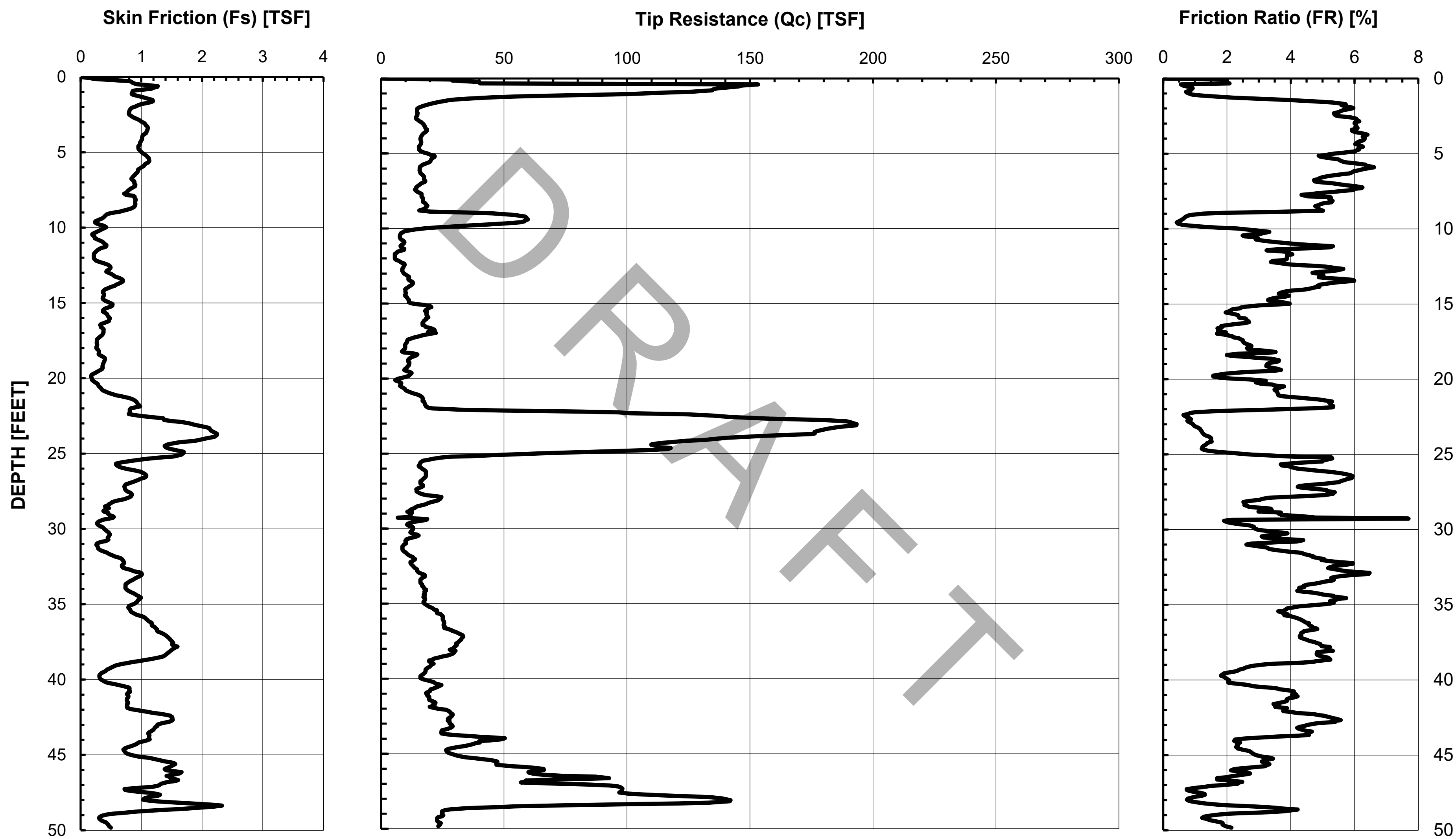
CPT-3

Project: **Brawley Science Center Addition**
Location: **San Diego State University, Brawley Campus**

Total depth: 50.28 ft, Date: 3/22/2022
Surface Elevation: -132.00 ft

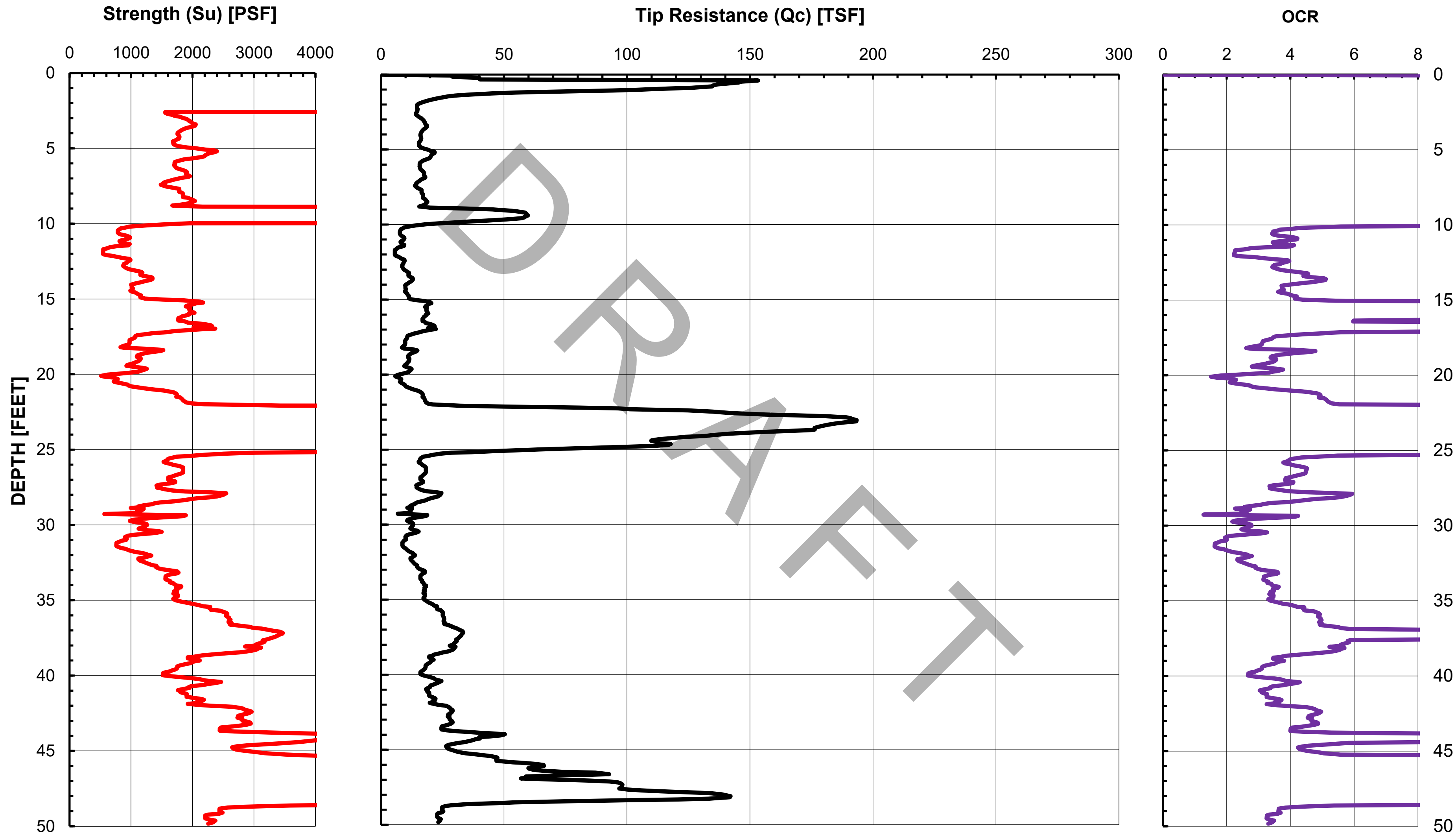




GROUP DELTA

CONE PENETOMETER DATA (CPT-4)

Document No. 22-0028
Project No. SD725
FIGURE A-4a



GROUP DELTA

ESTIMATED STRENGTH AND OCR (CPT-4)

Document No. 22-0028
Project No. SD725
FIGURE A-3b

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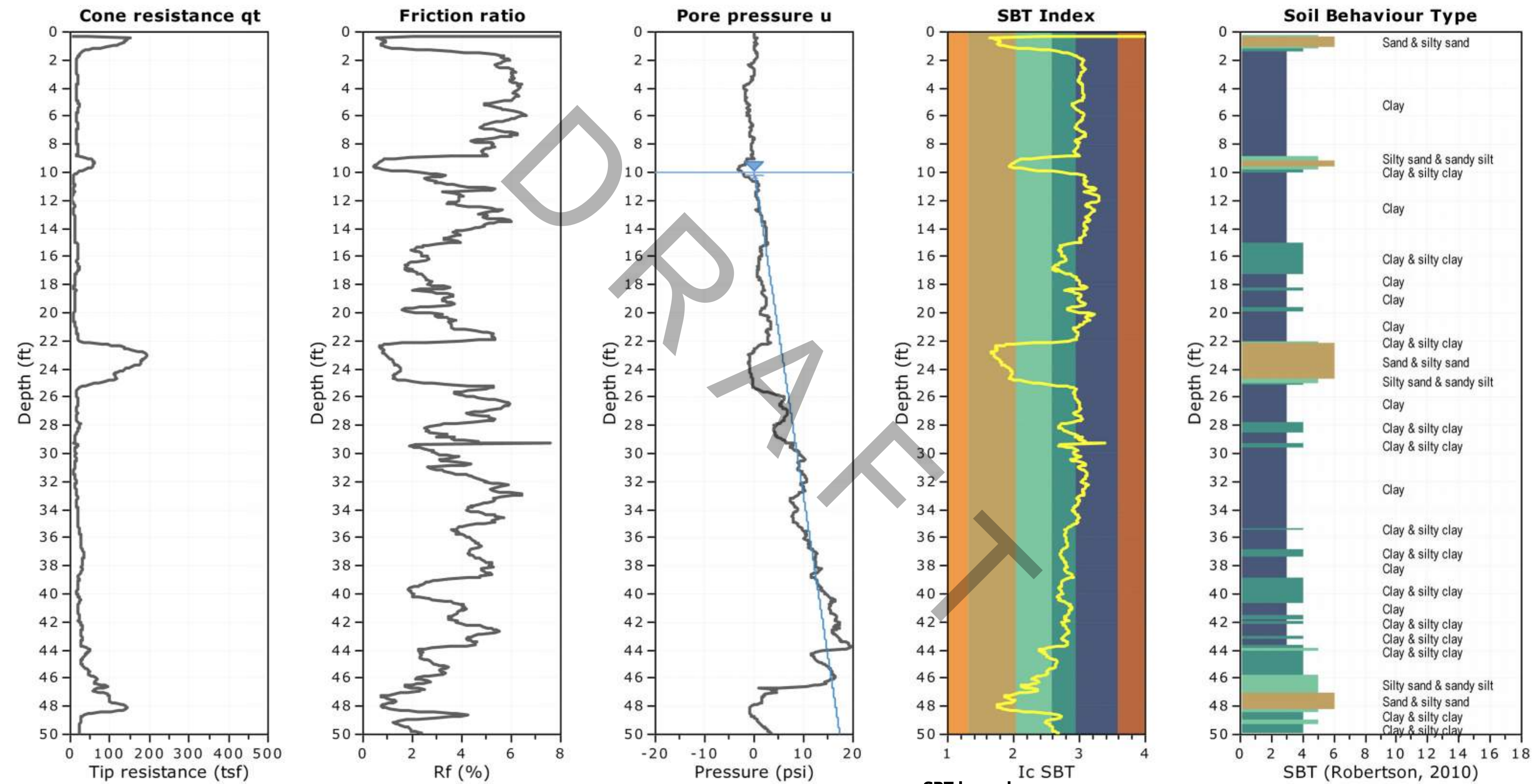
CPT-4

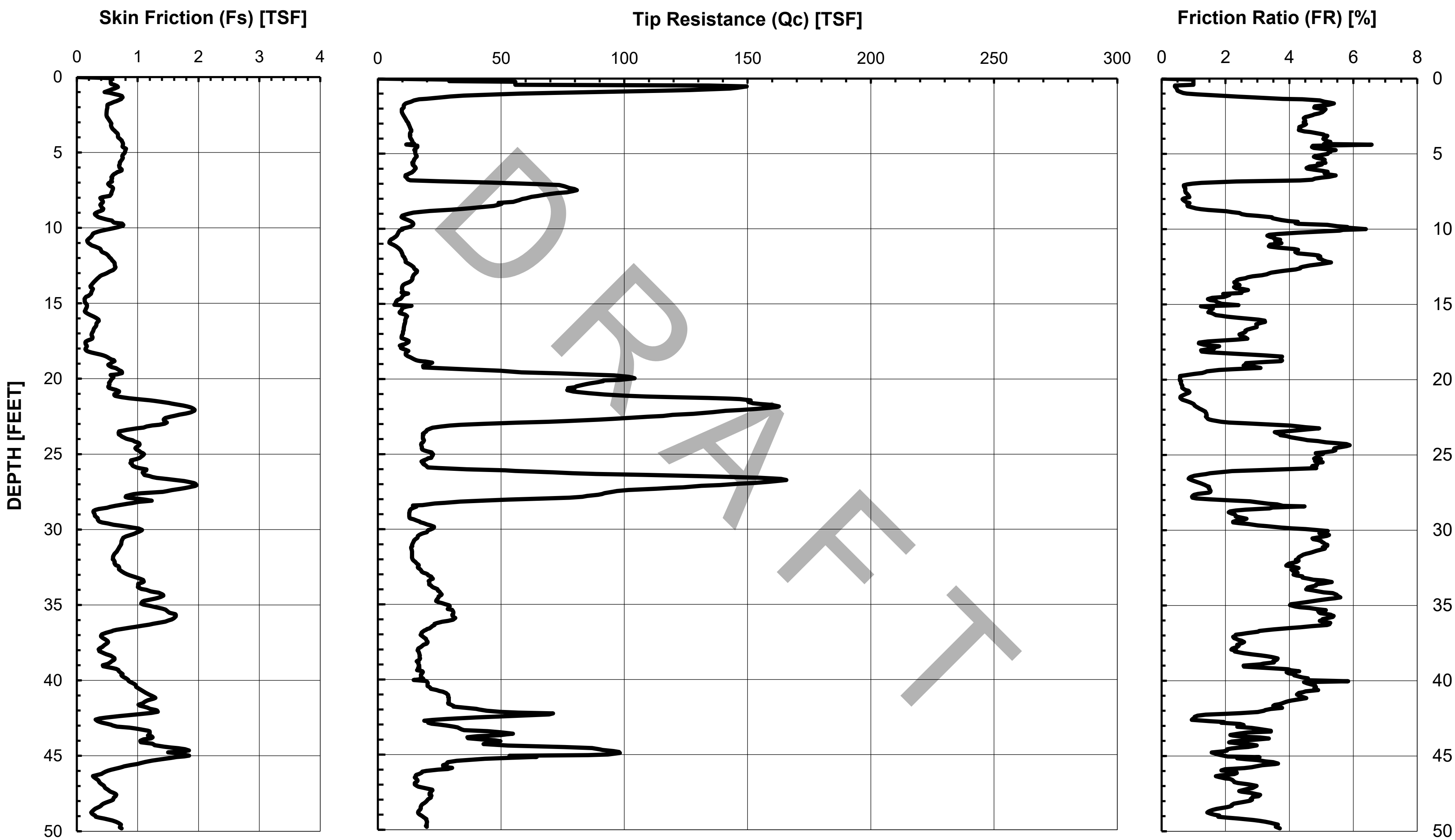
Project: **Brawley Science Center Addition**
Location: **San Diego State University, Brawley Campus**

Total depth: 50.33 ft, Date: 3/22/2022
Surface Elevation: -132.00 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



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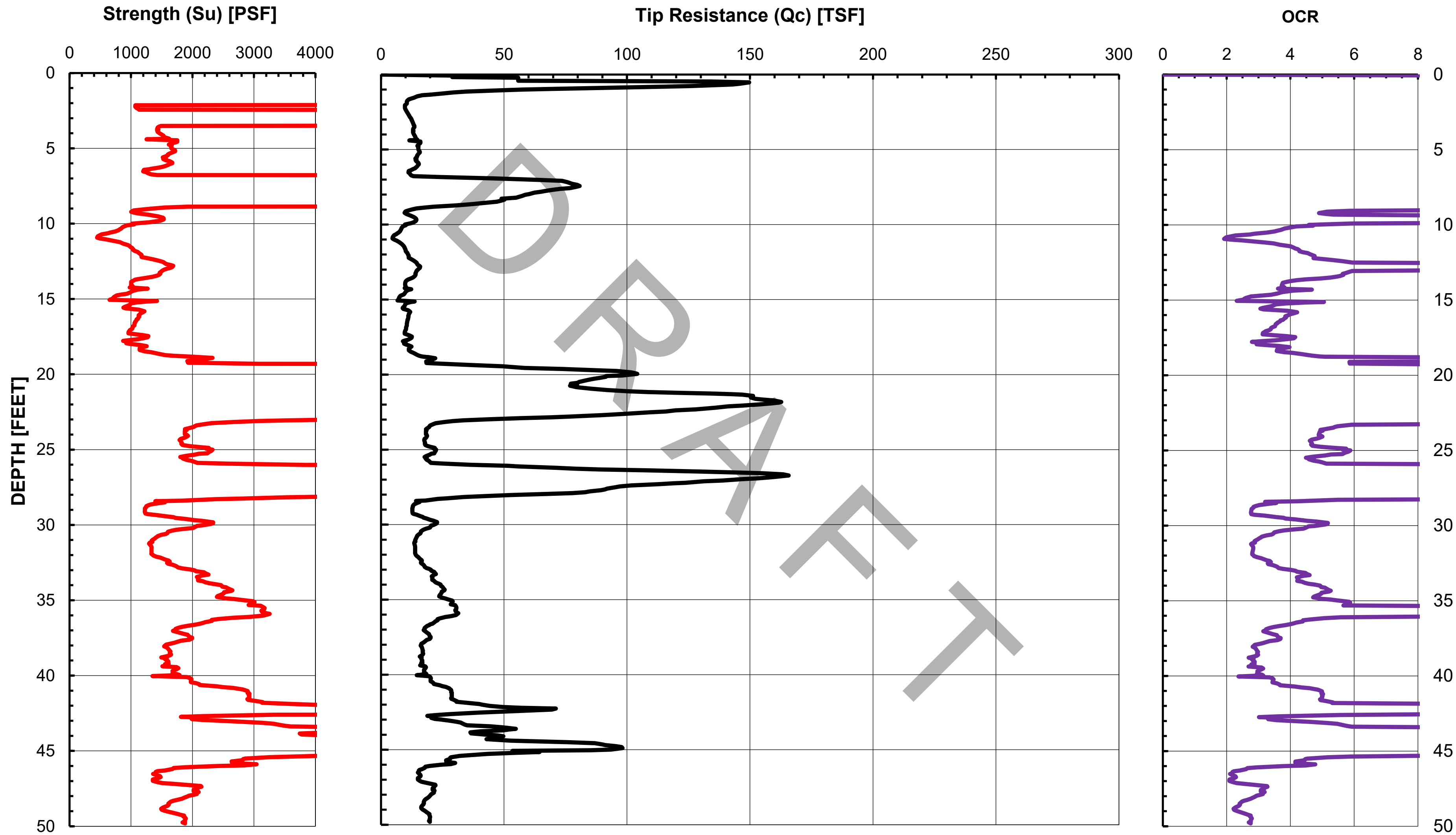
CONE PENETOMETER DATA (CPT-5)

Document No. 22-0028
Project No. SD725
FIGURE A-5a



GROUP DELTA

ESTIMATED STRENGTH AND OCR (CPT-5)

Document No. 22-0028
Project No. SD725
FIGURE A-3b

Group Delta Consultants
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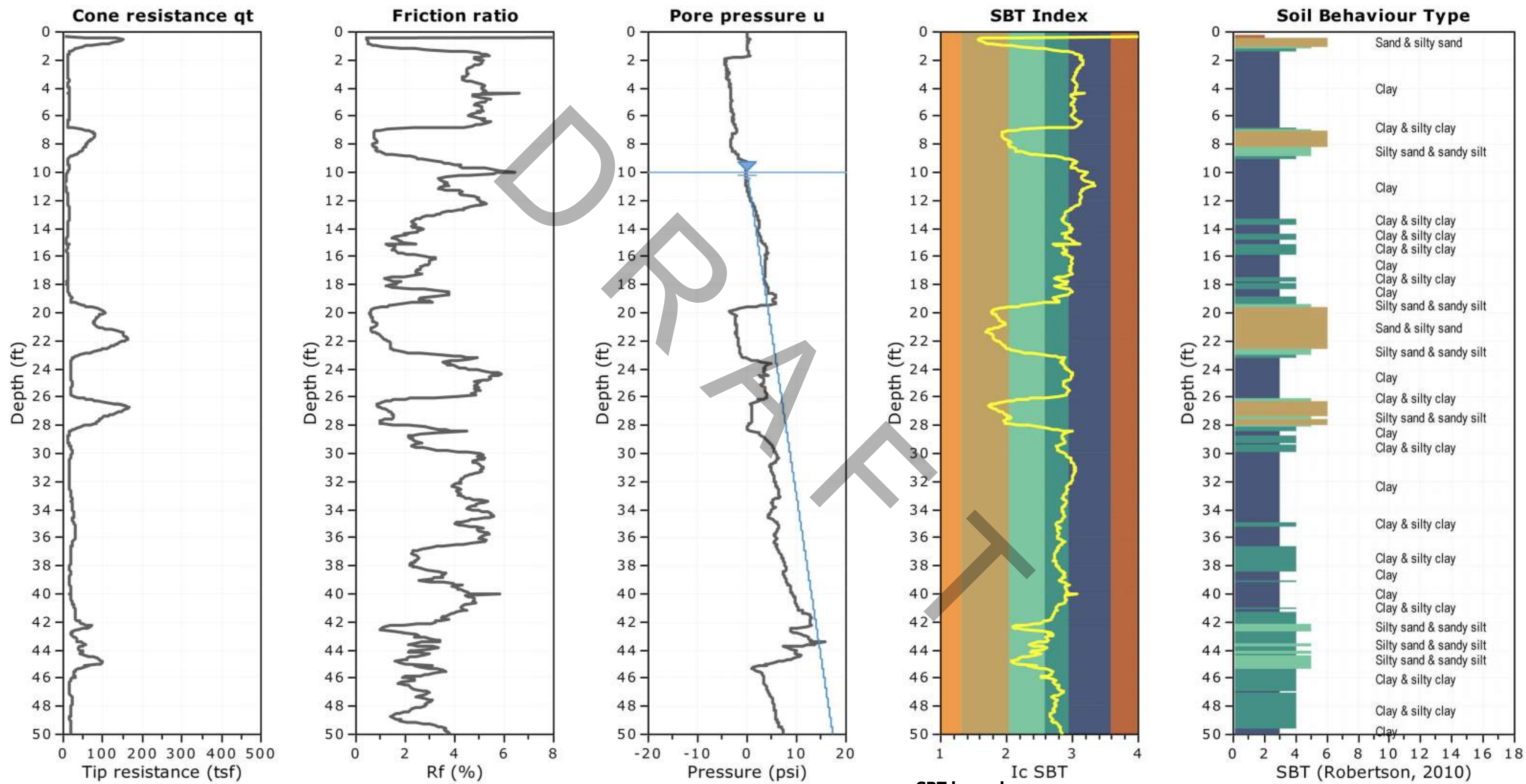
CPT-5

Project: **Brawley Science Center Addition**
Location: **San Diego State University, Brawley Campus**

Total depth: 88.66 ft, Date: 3/22/2022
Surface Elevation: -132.00 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 Consultants
9245 Activity Road, Suite 103
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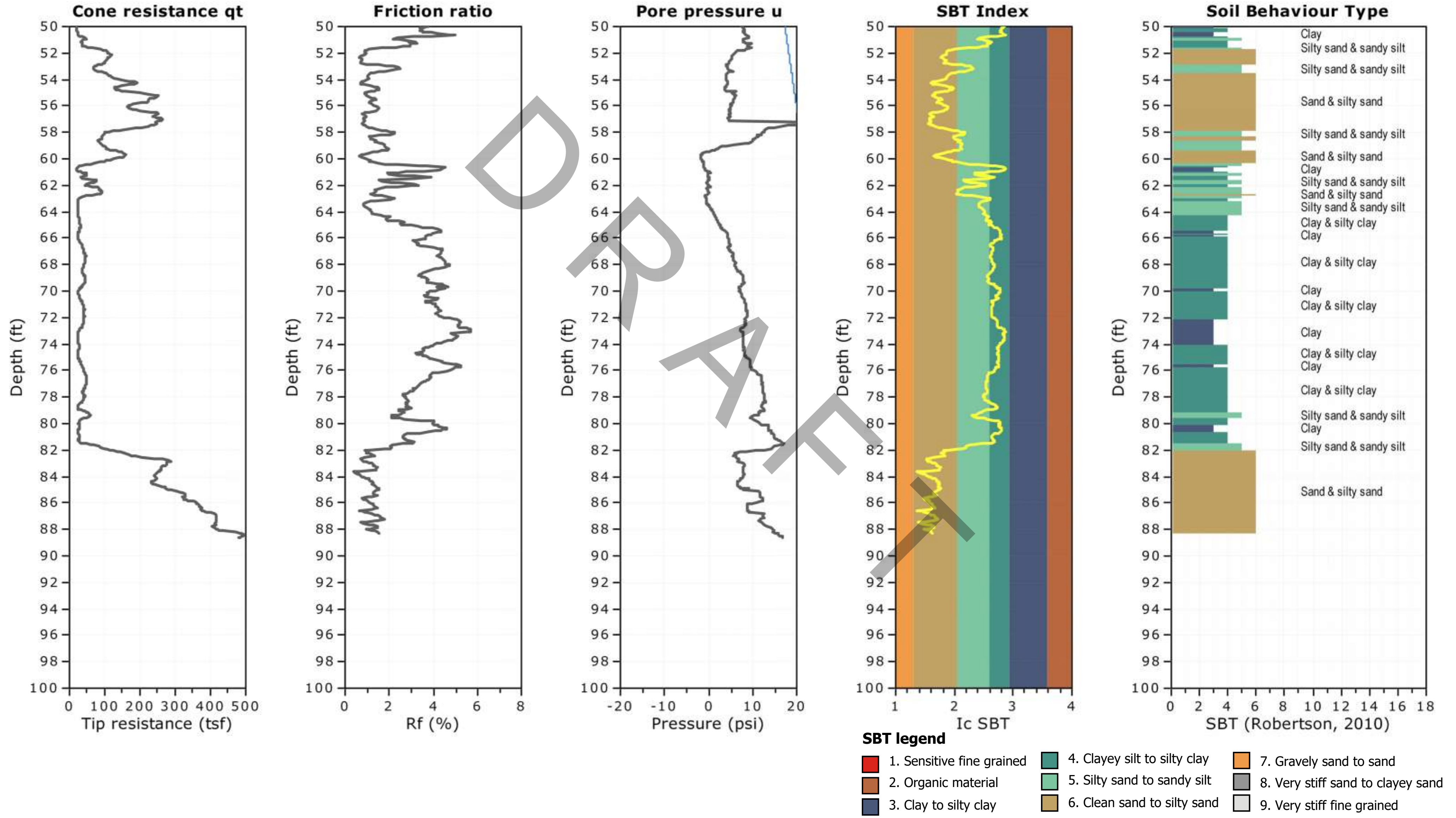
CPT-5

Project: **Brawley Science Center Addition**
Location: **San Diego State University, Brawley Campus**

Total depth: 88.66 ft, Date: 3/22/2022
Surface Elevation: -132.00 ft



Group Delta Consultants
SDSU Brawley Campus Science Building Addition
Brawley, CA

CPT Shear Wave Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity rom Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-5	5.02	4.02	4.49	8.54	525.77	
	10.01	9.01	9.23	19.24	479.69	443
	15.09	14.09	14.23	33.16	429.17	359
	20.05	19.05	19.15	44.50	430.44	434
	25.00	24.00	24.08	52.78	456.29	595
	30.05	29.05	29.12	62.38	466.80	525
	35.01	34.01	34.07	73.12	465.93	461
	40.03	39.03	39.08	81.68	478.47	586
	45.08	44.08	44.13	89.40	493.57	653
	50.03	49.03	49.07	96.24	509.88	723
	55.09	54.09	54.13	103.84	521.25	665
	60.07	59.07	59.10	109.64	539.07	858
	65.32	64.32	64.35	117.76	546.46	646
	70.08	69.08	69.11	123.96	557.51	767
	75.00	74.00	74.03	133.20	555.76	532
	80.05	79.05	79.08	139.94	565.07	749
	85.01	84.01	84.03	145.80	576.36	846
	88.62	87.62	87.64	149.04	588.05	1114

Shear Wave Source Offset - 2.00 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

LABORATORY TESTING

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 test description 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 accordance with ASTM D422 and were used to supplement the visual soil classifications. The test results and associated soil classifications are summarized in Figures B-1.1 through B-1.3.

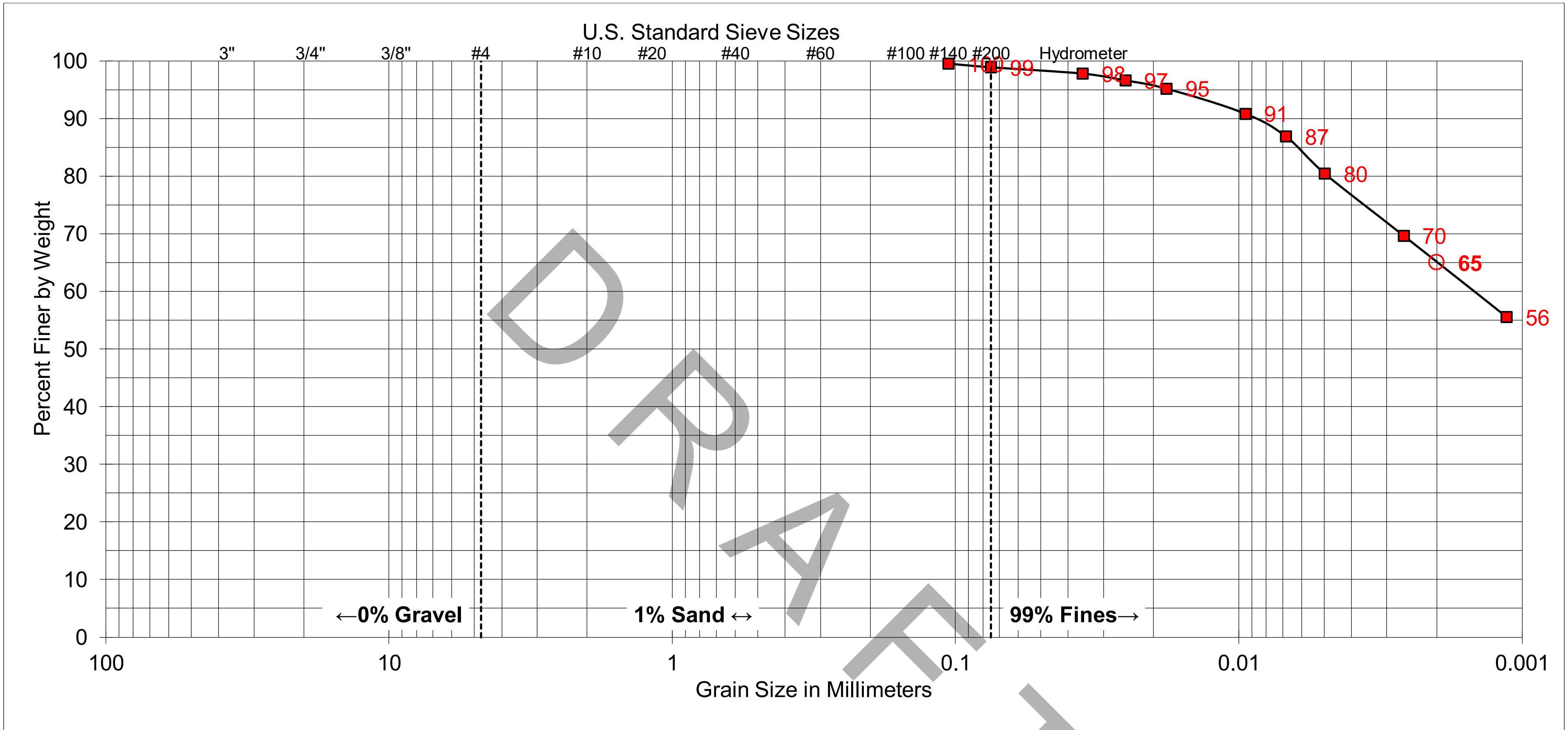
Atterberg Limits: ASTM D4318 was used to determine the liquid and plastic limits, and plasticity index of a selected clayey soil sample. The results are shown in Figure B-1.1.

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

Corrosivity Suite: To assess the potential for reactivity with buried metals, a soil sample was tested for pH and minimum saturated resistivity per Caltrans test method 643. To assess the potential for reactivity with concrete, the sample was tested for water soluble sulfate content per ASTM D516. The water-soluble chloride content was estimated using a calibrated ion specific electronic probe. The soluble sulfate and chloride was extracted from the soil under vacuum using a 10:1 (water to dry soil) dilution ratio. The corrosivity test results are summarized in Figure B-3.

Maximum Density/Optimum Moisture: The maximum density and optimum moisture content of a soil sample were determined per ASTM D1557. The results are shown in Figure B-4.

Direct Shear: The shear strength of a selected samples of the on-site soil was assessed using remolded shear testing performed in general accordance with ASTM D3080. The sample was remolded to about 90 percent relative compaction at near optimum moisture content, saturated, and then tested. The test results are presented in Figures B-5.



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	CPT-2
SAMPLE DEPTH:	1' - 4'

UNIFIED SOIL CLASSIFICATION: CH

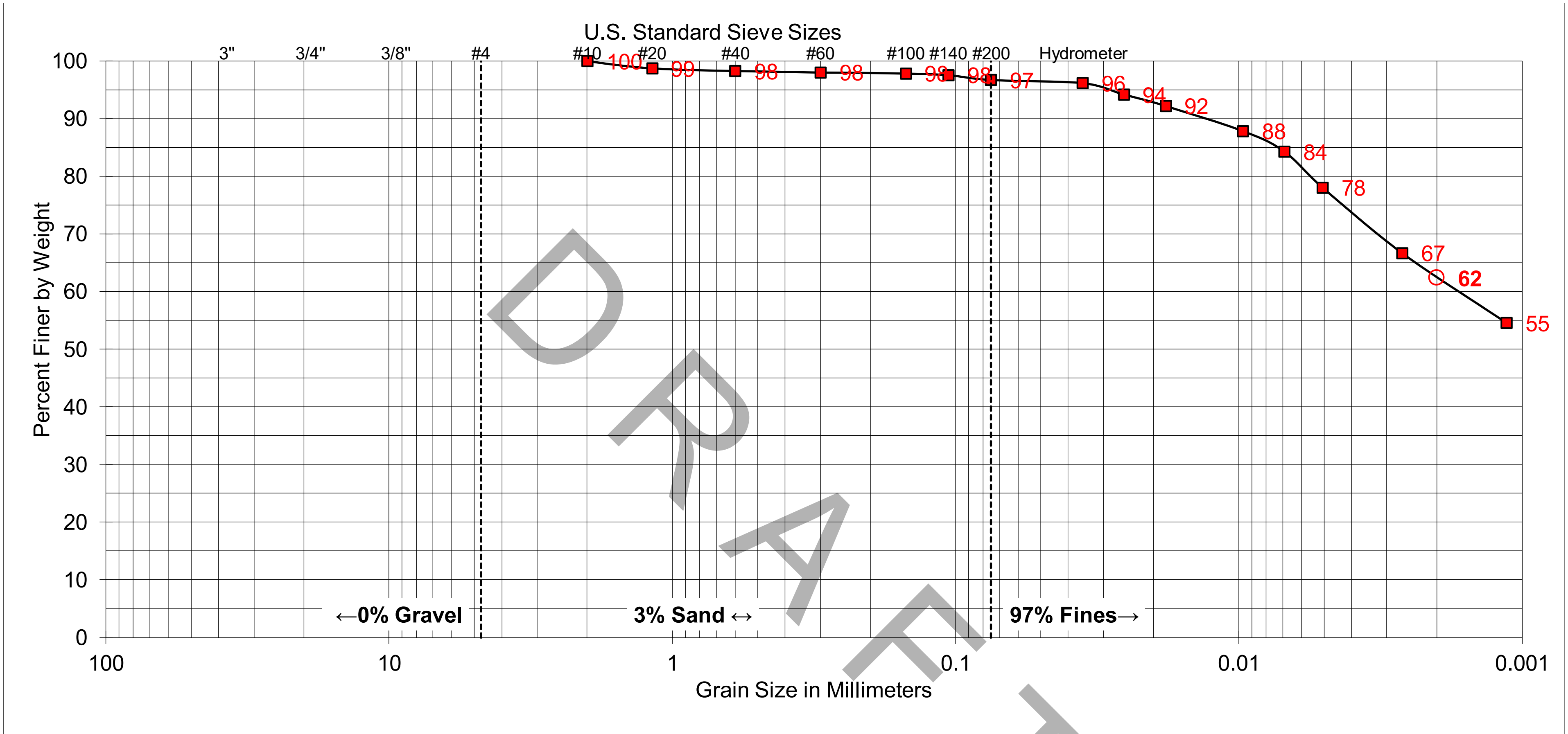
DESCRIPTION: FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	70
PLASTIC LIMIT:	23
PLASTICITY INDEX:	47

GROUP DELTA

SOIL CLASSIFICATION

Document No. 22-0028
Project No. SD725
FIGURE B-1.1



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	CPT-3
SAMPLE DEPTH:	1' - 4'

UNIFIED SOIL CLASSIFICATION: CH

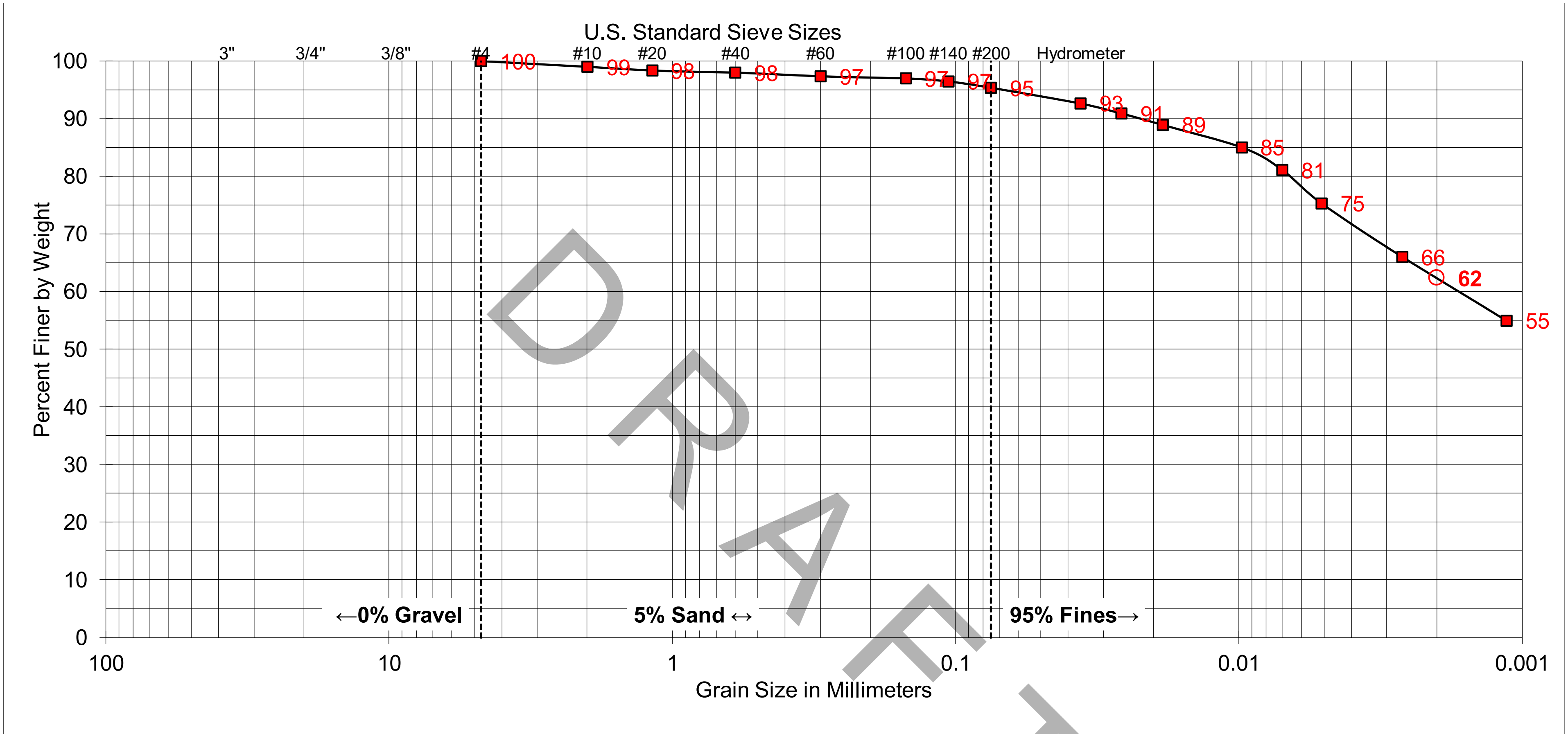
DESCRIPTION: FAT CLAY

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

GROUP DELTA

SOIL CLASSIFICATION

Document No. 22-0028
Project No. SD725
FIGURE B-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	CPT-4
SAMPLE DEPTH:	1' - 4'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

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

GROUP DELTA

SOIL CLASSIFICATION

Document No. 22-0028

Project No. SD725

FIGURE B-1.3

EXPANSION TEST RESULTS
(ASTM D4829)

SAMPLE ID	DESCRIPTION	EXPANSION INDEX
CPT-2 @ 1' – 4'	**Lacustrine Deposits**: Dark brown fat clay (CH).	127
CPT-3 @ 1' – 4'	**Lacustrine Deposits**: Dark brown fat clay (CH).	125
CPT-4 @ 1' – 4'	**Lacustrine Deposits**: Dark brown fat clay (CH).	100

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

SOLUBLE SULFATE TEST RESULTS
(ASTM D516)

SAMPLE ID	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
CPT-3 @ 1' - 4'	7.3	270	1.65 %	0.07

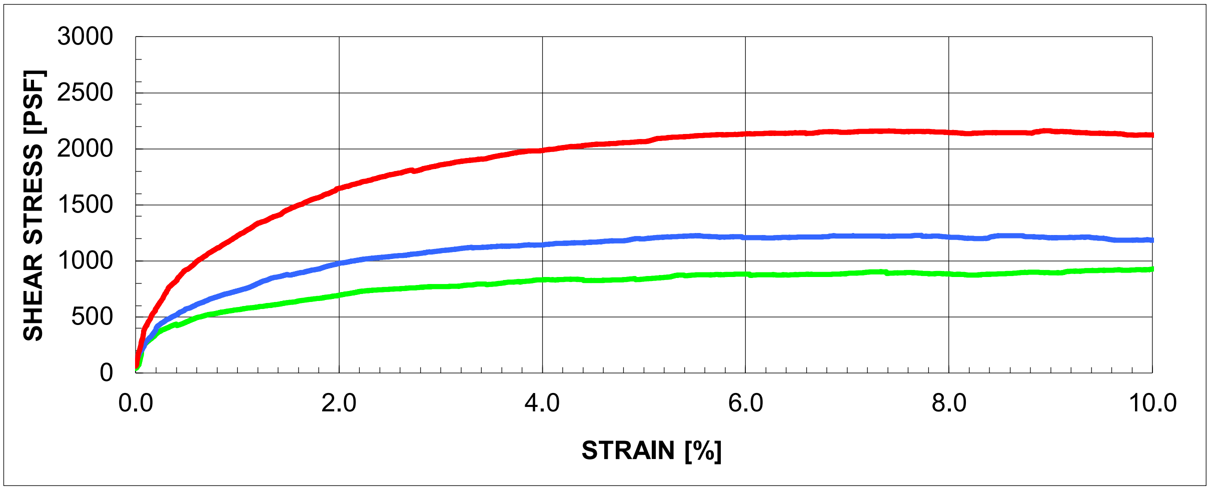
SULFATE CONTENT [%]	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	-
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

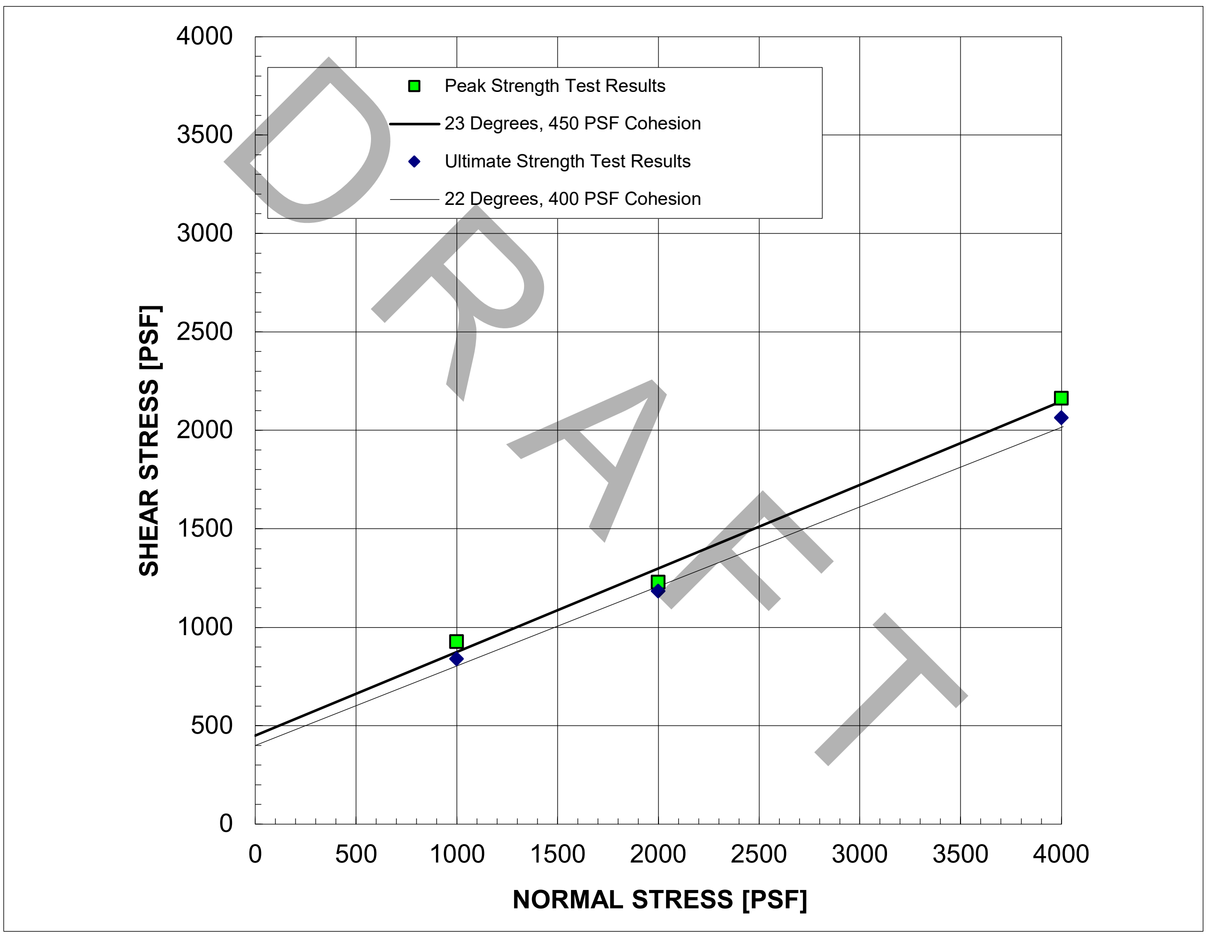
SOIL RESISTIVITY [OHM-CM]	GENERAL DEGREE OF CORROSIVITY TO FERROUS METALS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

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

MAXIMUM DENSITY & OPTIMUM MOISTURE
(ASTM D1557)

SAMPLE ID	DESCRIPTION	MAXIMUM DENSITY [lb/ft^3]	OPTIMUM MOISTURE [%]
CPT-3 @ 1' – 4'	**Lacustrine Deposits**: Dark brown fat clay (CH).	112.9	13.5





SAMPLE: CPT-4 @ 1' - 4'

FILL: Brown fat clay (CL).
(Remolded to ~90% RC @ Optimum).

	PEAK	ULTIMATE
ϕ'	23 °	22 °
C'	450 PSF	400 PSF

STRAIN RATE: 0.0002 IN/MIN

(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	100.7 PCF	100.7 PCF
w_c	14.5 %	24.9 %

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DIRECT SHEAR TEST RESULTS

Document No. 22-0028
Project No. SD725
FIGURE B-5