PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT
MIDWAY RISING SPORTS ARENA COMPLEX
3220, 3240, 3250, AND 3500 SPORTS ARENA BOULEVARD
SAN DIEGO, CALIFORNIA 92110

Prepared for

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GROUP DELTA

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Midway Rising LLC C/O
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700 Second Street
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Attention: Ryan Herrell, Executive Vice President

SUBJECT: **PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT**
Midway Rising Sports Arena Complex
3220, 3240, 3250, and 3500 Sports Arena Boulevard
San Diego, California 92110

Mr. Herrell:

Group Delta Consultants, Inc. (Group Delta) is submitting this Preliminary Geotechnical Investigation Report to support the preparation of the California Environmental Quality Act documentation and to provide preliminary recommendations for design and construction. Group Delta prepared this report per the referenced proposal (Group Delta, 2022). This report is a final version for the Specific Plan and Tentative Map submittal.

We appreciate the opportunity to support this project. Please contact us with questions or comments, or if you need anything else.

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EXECUTIVE SUMMARY

Group Delta Consultants, Inc. (Group Delta) is submitting this report to support the initial phases of the redevelopment of the 50-acre Pechanga Sports Arena site. Midway Rising LLC proposes a new arena, entertainment plaza, hotel, six blocks of residential housing, and park space. The redevelopment plans to raise portions of the site up to three feet.

Group Delta managed advancing eight borings and eight cone penetration tests to depths ranging from 20 to 120 feet with laboratory testing of soil samples collected from the borings. Group Delta interpreted the field and laboratory data, and then conducted engineering analyses to prepare this report with our findings, conclusions, and recommendations.

Geologically young, loose, and soft soils associated with the changing coastline and the growth of the San Diego River Delta underlie the site. Undocumented fill underlain by paralic estuarine deposits extend from the ground surface to depths ranging from about 100 to 105 feet. Due to an abrupt change in apparent density and soil type at depths of about 60 feet, these deposits are subdivided into *upper* and *lower* paralic estuarine deposits. Sandstone and conglomerate mapped as old paralic deposits occur below the paralic estuarine deposits.

Groundwater depths range from 6 to 16 feet and fluctuate from tidal influences. Underground obstructions consist of the piles supporting the Pechanga Arena, utilities, remnant building foundations, and a historic dump site.

The fill and upper paralic estuarine deposit soils are highly compressible and possess a low shear strength. The observed presence of mica, organics, and/or seashells can adversely influence the geotechnical engineering characteristics of these deposits. The lower paralic estuarine deposit soils are less compressible and gain shear strength with depth. The sandstone and conglomerate old paralic deposits possess a very high shear strength and a very low compressibility.

The primary geologic hazard is liquefaction of the upper paralic estuarine deposit soils during an earthquake. The most likely secondary effect of liquefaction is settlement. Liquefaction requires site response analyses to incorporate the amplification of ground motions into the seismic design of structures. Liquefaction also creates large downdrag loads on piled foundations.

Design and construction of the redevelopment will need to mitigate the potential for soil liquefaction, consider the high compressibility and low shear strength of the underlying soils, and manage a shallow groundwater level. Since the proposed buildings have high structural loads, they will require, individually or combined, ground improvement and/or deep foundations to provide satisfactory long-term performance. New underground utilities and existing underground utilities that will remain will need to consider the settlement caused from fill placement and the settlement caused by liquefaction. This report provides preliminary recommendations for design and construction and discusses geotechnical-related construction considerations known at this time.

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation by Group Delta Consultants, Inc. (Group Delta) for the redevelopment that Midway Rising LLC (Midway Rising) is proposing for the Pechanga Arena site located at 3220, 3240, 3250, and 3500 Sports Arena Boulevard in the City of San Diego, California. Figure 1, Site Location Map, shows the regional location of the project site.

The purposes of this report are to: 1) provide geologic and geotechnical information to support the preparation of the California Environmental Quality Act (CEQA) documentation, 2) provide preliminary geotechnical recommendations for design and construction, and 3) discuss the geotechnical-related construction considerations known at this time. Revisions may be needed for changes to the redevelopment, the detailed design phase, and changes in expected construction processes and/or subsurface conditions exposed during construction.

Group Delta developed the recommendations using information from a previous geotechnical desktop study report (Group Delta, 2023), recent subsurface exploration and laboratory testing, geotechnical engineering interpretation and analyses, and our previous experience.

1.1 Scope of Services

Group Delta prepared this report per the referenced proposal (Group Delta, 2022). We provided the following scope of services.

- A field investigation consisting of eight borings and eight cone penetrometer tests to depths ranging from 20 to 120 feet. Figure 2 shows the approximate locations of these explorations. Appendix A provides relevant information.
- Geotechnical laboratory testing of soil samples collected from the borings. Appendix B provides the test results.
- Interpretation of the field and laboratory data and engineering analyses. Appendix C provides additional information.
- Preparation of this report with our findings, conclusions, and recommendations.

1.2 Site Description

The 50-acre site is located north of Sports Arena Boulevard and south of Kurtz Street in the Midway District of the City of San Diego. The existing Pechanga Arena and surrounding surface parking occupies most of the site. Low rise retail and commercial buildings occupy the eastern and western portions of the site. Interstate 8 and the San Diego River levees are north of the site. The sides of the levee channel are armored with riprap with fill embankments ranging from 16 to 18 feet high (Group Delta, 2015).

The elevation of the site ranges from about 7.5 feet to 15 feet, NGVD 29 (Project Design Consultants, 2023). The highest elevations surround the existing Pechanga Arena. The lowest elevations are in the northwest area of the site.

1.3 Project Description

Midway Rising proposes to redevelop the site with a new arena, entertainment plaza, hotel, and six blocks of residential housing. The blocks of housing will be residential over parking, residential over retail and parking, and residential over retail that will surround parking. The redevelopment will include several types of parks. We have based our understanding of the redevelopment on the Midway Rising Specific Plan (City Thinkers, Safdie Rabines, OJB and PDC/Bowman; 2023), the Tentative Map for Midway Rising (PDC, 2023), and the information described below.

The redevelopment earthwork proposes a minimum building pad elevation of 10 feet, NGVD 29 to accommodate flooding (Project Design Consultants, 2023). Project Design Consultants estimate this earthwork could require 20,000 to 30,000 cubic yards of fill to raise portions of the site up to three feet to achieve the proposed building pad elevation.

The residential housing may be 8 to 12 story structures consisting of five stories of wood framing over three stories of cast-in-place reinforced concrete with column loads of 750 kilopounds (kips) or twelve stories of post-tensioned concrete with column loads of 1,700 kips (KPFF, 2023). We understand from preliminary communication with Walter P. Moore the new sports arena could have column loads ranging from 100 to 1,000 kips. Basements and below grade parking are not proposed as part of the development.

1.4 Prior Geotechnical Studies

Several geotechnical engineering investigations have been completed at the site and nearby. Group Delta reviewed these studies and prepared a Geotechnical Desktop Study Report (Group Delta, 2023). We have incorporated relevant information into the findings presented in this report.

2.0 FIELD AND LABORATORY INVESTIGATION

The field investigation included a site reconnaissance and eight hollow stem/mud rotary borings and eight Cone Penetration Tests (CPT) to depths ranging from 20 to 120 feet. These explorations were completed during February and March 2023. Figure 2, Exploration Locations shows their approximate locations. Figure 2 also shows the locations of cross-sections A-A’ and B-B’, Figures 3A and 3B, that depict an interpretation of the subsurface conditions. Appendix A provides the exploration records and other relevant information. The scope of the field investigation complies with the recommendation for subsurface exploration provided in the Additional Geotechnical Engineering Services of the referenced Geotechnical Desktop Study Report (Group Delta, 2023).

Soil samples were collected from the borings for laboratory testing. The testing program included sieve analyses and plasticity index testing to classify the soil using the Unified Soil Classification

System and to provide data to evaluate the potential for liquefaction. Other index-type tests were completed to evaluate the soil expansion potential and corrosivity. Consolidation tests were conducted to help evaluate static settlement. Direct shear and unconfined compressive strength tests were completed to evaluate soil shear strength. The Exploration Records in Appendix A and Appendix B provide the laboratory test results.

3.0 GEOLOGY AND SUBSURFACE CONDITIONS

Geologically young, loose, and soft soils associated with the changing coastline and the growth of the San Diego River Delta underlie the site. These soils occur as fill from land reclamation and as alluvial/estuarine sediments deposited from the ancient San Diego River Delta. Old paralic deposits comprising sandstone and conglomerate underlie these soils (Kennedy and Tan, 2008). Figure 4, Geologic Map, shows the mapped limits of these geologic units relative to the site.

Prior subsurface explorations conducted at the site and nearby (Group Delta, 1999, 2019, and 2020) and the current subsurface explorations encountered undocumented fill[1] over paralic estuarine deposits. Some of these explorations encountered old paralic deposits below these soils. The following paragraphs describe these materials. Cross-sections A-A’ and B-B’, Figures 3A and 3B, depict an interpretation of the subsurface conditions.

3.1 Undocumented Fill

Undocumented fill soils (fill) were observed in all the exploratory borings. The soils were interpreted to range from 7 to 13 feet in thickness. The fill soils were observed to consist of clayey sand (Unified Soil Classification System - SC) and silty sand (SM) and poorly graded sand (SP). Gravel and cobbles, and construction debris were frequently observed in the upper portions of the fill. The apparent density based on drive sampler resistance was very loose to medium dense.

3.2 Paralic Estuarine Deposits

Paralic estuarine deposits were observed below the fill to elevations ranging from 3.0 to -1.0 feet NGVD 29. The soils were interpreted to extend to depths of about 100 to 105 feet. Due to an abrupt change in apparent density/consistency and soil type, these deposits were subdivided into two units referred to as the *upper* and *lower* paralic estuarine deposits described below.

3.2.1 Upper Paralic Estuarine Deposits

Upper paralic estuarine deposits were interpreted to extend to elevations ranging from about -40 to -50 feet NGVD 29, resulting in a thickness ranging from about 40 to 55 feet. These soils were observed to mostly consist of silty sand (SM), sand (SP-SM), and non-plastic sandy silts (ML) that mostly occur in 5-foot thick or less layers. An approximately 10-foot-thick layer of fat clay (CH) was

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[1] **Undocumented fill** is soil that has been placed and compacted with no documentation of observation and compaction testing by a geotechnical engineer.

observed from elevations -27 to -37 feet NGVD 29 within the western portion of the site.
The upper paralic deposit soils were typically observed to be dark gray to grayish black and to have mica and seashells. The soils often had a light organic odor. The apparent density and consistency based on drive sampler resistance was very loose to medium dense, and soft to stiff.

3.2.2 Lower Paralic Estuarine Deposits

Lower paralic estuarine deposits were interpreted to extend to elevations ranging from about -89 to -97 feet NGVD 29, resulting in a thickness below the upper paralic deposits ranging from about 40 to 55 feet. These soils were observed to consist mostly of silty sand (SM), sand (SP-SM, SP), and sandy silt (ML). The apparent density based on drive sampler resistance was medium dense to very dense. These soils were typically observed to be medium to dark grey and to have some mica.

3.3 Old Paralic Deposits

Old paralic deposits were observed below the paralic estuarine deposits to the maximum depth of exploration of 120 feet. When disturbed by drilling, the old paralic deposits were observed to consist of poorly graded sand with gravel (SP) and poorly graded gravel with sand (GP). The explorations terminated in a layer of gravel and cobbles that was initially encountered at elevations ranging from -89 to -97 feet NGVD 29. The relative density based on drive sampler resistance was very dense.

3.4 Groundwater

Groundwater levels are closely related to the water surface elevation within the San Diego River and subject to tidal influences. Groundwater was measured in the various subsurface explorations at depths ranging from 6 to 16 feet that correspond to elevations of 3.0 to -4.0 feet NGVD 29. The most direct measurement of groundwater occurred in a temporary monitoring well installed in Boring A-23-013, where groundwater was measured at a depth of 7 feet that corresponds to an elevation of approximately 2 feet NGVD 29. Appendix A provides a summary of the groundwater measurements.

Groundwater levels will fluctuate from tidal influences. Daily tidal fluctuations recently measured at the nearby Quivira Basin recording station ranged from about 0.0 to 8.0 feet NGVD 29 (NOAA, 2023). The porosity of the soil should attenuate tidal fluctuations.

3.5 Underground Obstructions

In addition to the piles supporting the Pechanga Arena (Group Delta, 2023), underground utilities, remnant building foundations, and the historic dump site (Group Delta, 1999), there may be other types of underground obstructions. Typical environmental assessments, along with surface geophysical studies, potholing, and research by cultural resources specialists, such as an architectural historian, should help to locate obstructions prior to construction.

4.0 GEOLOGIC HAZARDS

The primary geologic hazard at the site requiring mitigation is liquefaction. The City of San Diego Seismic Safety Element map indicates the site lies within the “Liquefaction, High Potential – shallow groundwater, major drainages, hydraulic fills” geologic hazard category. Figure 5 reproduces this map with an outline of the site. Listed below are the geologic hazards that could affect the project followed by discussions that elaborate on these hazards.

- Strong Ground Motion
- Earthquake Surface Fault Rupture
- Liquefaction and Secondary Effects
- Seismic Compaction
- Tsunamis

4.1 Strong Ground Motion

The site could be subjected to moderate to strong ground motion from a nearby or more distant, large magnitude earthquake occurring during the expected life span of the project. Numerous regional and local faults can produce large earthquakes with magnitudes (M) 6.0 or greater. Figure 6, Regional Faults and Earthquakes Map, presents the locations of these faults and the historical earthquake epicenters recorded on them. This hazard is managed by structural design using the latest edition of the California Building Code. This report provides preliminary recommendations.

4.2 Earthquake Surface Fault Rupture Hazard

The potential for surface fault rupture is very low. No active or potentially active faults project towards the site. Surface fault rupture is displacement on a fault that occurs at the ground surface because of tectonic forces. Based on the findings from this geotechnical investigation, prior geotechnical investigations in the area, and the City of San Diego and the State of California geologic hazard mapping, the site is not underlain by an active or a potentially active fault, per the City of San Diego definitions of fault activity[2] in their Guidelines for Geotechnical Reports (City of San Diego, 2018). We have based this assessment on the following specific information.

- The California Geological Survey (CGS, 2021) maps the trace of the active Rose Canyon Fault Zone (RCFZ) approximately 4,000 feet east of the western perimeter of the site. The RCFZ is a complex system of northwest-trending, right-lateral strike-slip, steeply dipping, parallel to subparallel faults. Figure 7, Earthquake Zones of Required Investigation, La Jolla outlines the site on the CGS map of the same title relative to the RCFZ. Figure 5, San Diego Seismic Element also shows the location of the RCFZ relative to the site.

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[2] **Active Faults** – this class of fault has had demonstrable surface displacement during Holocene time (past 11,700 years). **Potentially Active Faults** - faults with Quaternary (2.6 million years ago) displacement, but Holocene surface displacement is indeterminate. **Inactive Faults** – pre-Quaternary faults.

- The City of San Diego Seismic Safety Element maps the trace of the Point Loma Fault approximately 1,800 feet southwest of the southwest corner of the site. This map also indicates the trace of a short unnamed fault is located approximately 1,100 feet southwest of the southwest corner of the site. The City of San Diego Seismic Safety Element map indicates these faults are “Potentially Active, Inactive, Presumed Inactive or Activity Unknown.” Figure 5, San Diego Seismic Safety Element shows the locations of these faults.

4.3 Liquefaction and Secondary Effects

Liquefiable soils underlie the site. 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 soil pore water pressure from strong ground motion during an earthquake. The secondary effects of liquefaction are sand boils, settlement, lateral spreading, and overall instability and/or permanent horizontal deformations within sloping ground. Of these, settlement should be the most likely to occur given the site surface and subsurface conditions. Liquefaction-induced settlement can cause adverse vertical deformation of the ground surface and the soils supporting shallow foundations, and it can create large downdrag loads on piles.

4.3.1.1 Liquefaction

An assessment of the potential for liquefaction triggering and an estimate of the liquefaction-induced settlement interprets the following:

- Potentially liquefiable soils occur at the design groundwater level (+3 feet NGVD 29) and extends to about 60 feet below existing grades (-50 feet NGVD 29). The liquefiable soils are predominantly silty sand (USCS Symbol SM), sand (SP-SM), and non-plastic sandy silts (ML). In the upper 40 feet below existing grades (-30 feet NGVD 29), liquefiable materials generally occur as a thick, continuous layer that is occasionally interrupted by thin layers of non-liquefiable materials less than about three feet in thickness. Below a depth of 40 feet, liquefiable materials occur in relatively thin layers (about 5-foot thick or less) that are separated by non-liquefiable materials that range from about two to ten feet in thickness.
- Estimated settlements range from 7 to 10 inches in our calculations. Differential settlement over the common 30- to 40-foot column spacing is typically estimated to be one-half to two-thirds of the total settlement. Actual settlements realized in the field following a seismic event can vary significantly from calculations. Accordingly, design total and differential liquefaction induced settlements are provided in a table in Appendix C to account for the potential variability of actual liquefaction induced settlements compared to those that were calculated as a part of this evaluation.

Appendix C summarizes the methods used to assess liquefaction triggering and estimate liquefaction-induced settlement and provides a summary of the results of the analyses.

4.3.1.2 Lateral Spreading

The potential for lateral spreading should be low because an unprotected face does not exist along the San Diego River near the site since there is a flood control levee maintained by the City of San Diego (PDC, 2023). The sides of the levee channel are armored with riprap (Group Delta, 2015). Lateral spreading is the relatively rapid, fluid-like movement that can cause large horizontal deformations within the gently sloping ground near the shoreline with an unprotected face.

4.4 Seismic Compaction

The potential for seismic compaction-induced settlement should be low. Soils prone to seismic compaction should be removed by typical site preparation earthwork. Seismic compaction is the densification from strong ground motion of loose granular soil that exist above groundwater.

4.5 Tsunamis

The potential for large waves from a tsunami to affect the site should be low. The State of California Tsunami Inundation Map (California Emergency Management Agency, 2009) indicates the site does not lie within a tsunami inundation area. Tsunamis are sea waves created by the sudden uplift of the sea floor during an earthquake. Figure 8, Tsunami Inundation Map, reproduces this map with the outline of the site shown.

The California Tsunami Inundation map indicates the existing San Diego River levees north of the site would channel a tsunami up the San Diego River channel beyond the project site. Group Delta summarized a prior geotechnical evaluation of these levees near the West Mission Bay Bridge (Group Delta, 2023).

5.0 GEOTECHNICAL ENGINEERING CHARACTERISTICS

The primary geotechnical engineering characteristics that will influence design and construction are the high compressibility and the low shear strength of the fill and upper paralic estuarine deposits. These soils extend to depths ranging from about 50 to 60 feet. The lower paralic estuarine deposits below these soils gain shear strength and become less compressible. Sandstones and conglomerate old paralic deposits underlay these materials at depths ranging from about 100 to 105 feet. The geotechnical engineering characteristics of these materials should be similar to very dense sands.

The presence of mica, organics, and/or seashells observed in the estuary environment of the site can influence the geotechnical engineering characteristics of the fill and upper paralic estuarine deposits. In particular, the presence of mica flakes in sands has been shown to reduce shear strength and alter volume change characteristics (Hight, 2002; Mundegar, 1998).

5.1 Compressible Soils

The loads imposed on the existing fill and upper paralic deposits soils from placing additional fill and using shallow foundations could generate adverse static settlement. Static settlement is the combination of short-term elastic and long-term consolidation vertical deformations. Coarse grained soils, such as sand, should settle elastically with the application of load. Fine-grained soils, such as clay, should continue to settle after the load is fully applied. Provided below are preliminary estimates:

- The total static settlement from the placement of about 3 feet of fill is estimated to range from about 1.5 to 2.5 inches. The duration for this settlement to be substantially complete is estimated to range from 2 months, to up to 12 months, after the completion of the fill placement. This substantial variability stems from a thick fat clay layer that underlies the western portion of the site, which is estimated to take significantly longer to reach substantial completion than the eastern portion of the site. A test fill as described in the *Construction Considerations* section of the report should be considered in this area.
- The total static settlement from a 10- by 10-foot spread footing designed using an allowable bearing pressure of 1,000 pounds per square foot (psf) is estimated to be one inch or less. Differential settlement could range from one-half to two-thirds of the estimated total settlement over a typical horizontal column spacing of 30 to 40 feet. The duration for this settlement to be substantially complete is estimated to be one month or less from the initial loading.

Appendix C summarizes the methods used to estimate settlement and provides a summary of results of the analyses.

5.2 Soil Shear Strength

Direct measurement and typical geotechnical correlations indicate the fill and upper paralic deposits possess relatively low shear strength. This low shear strength precludes using shallow foundations except where a structure can be economically designed using a relatively low allowable bearing pressure and it can accommodate the settlement estimated from static loads and liquefaction per ASCE 7-16. Appendix C provides plots of soil shear strength versus elevation.

5.3 Expansive Soils

Expansive soils are clays that are prone to shrinking or swelling with decreases or increases in moisture content. Near surface soil samples exhibited a “very low” to ‘low” potential for expansion when tested per ASTM D4829. Construction may encounter expansive soils in the fill due to the uncontrolled method of placement. Appendix B provides the laboratory test data.

5.4 Corrosive Soils

A screening level qualitative assessment of the general degree of corrosivity to underground ferrous metals and concrete using the results of laboratory tests on soil samples indicates the potential for increased corrosivity below groundwater because of the influence of seawater. Findings from the pH, resistivity, sulfate, and chloride laboratory test results are summarized below. Appendix B provides the test results.

GENERAL DEGREE OF CORROSIVITY

Soil Condition	pH	Resistivity	Chloride	Sulfate
Above Groundwater [1]	Negligible	Moderate	Negligible	Negligible
Below Groundwater	Negligible	Severe	Severe	Severe

1. May vary considerably due to the uncontrolled nature of the placement of fill.

The above assessment refers to commonly published guidance such as Caltrans (2022) and NACE International (1989). A corrosion consultant should be contacted for specific recommendations.

6.0 CONCLUSIONS

The project site is geotechnically suitable for the proposed redevelopment. The proposed redevelopment should not adversely affect adjacent properties and right-of-way. These conclusions consider that design and construction will need to mitigate the potential for soil liquefaction, consider the high compressibility and low shear strength of the underlying soils, and manage a shallow groundwater level. The primary geotechnical conclusions are provided below.

- Undocumented fill that is underlain by upper paralic estuarine deposits extend from the ground surface to a depth of about 60 feet. These soils were observed to consist mostly of silty sand (USCS Symbol SM), sand (SP-SM), and non-plastic sandy silt (ML). An approximately 10-foot-thick layer of fat clay (CH) was observed at depths of about 40 to 50 feet within the western portion of the site.
- The fill and upper paralic estuarine deposit soils are highly compressible and possess a low shear strength. The observed presence of mica, organics, and/or seashells can also influence the geotechnical engineering characteristics of these deposits. These soils are liquefiable. The liquefaction-induced settlement was estimated to be from 7 to 10 inches.
- Below the upper paralic estuarine deposits are soils referred to as lower paralic estuarine deposits that extend from the ground surface to depths of about 100 to 105 feet. These soils were observed to consist of silty sand (SM), sand (SP-SM, SP) and sandy silt (ML).
- The apparent density of the lower paralic estuarine deposits soils increases and therefore they become less compressible, gain shear strength, and are not considered liquefiable.
- Sandstone and conglomerate old paralic deposits occur below the paralic estuarine deposits. The disturbed old paralic deposits were observed to consist of poorly graded sand with gravel (SP) and poorly graded gravel with sand (GP). The apparent density of these material is very dense. They have very high shear strength and very low compressibility.
- Observed groundwater levels range from 6 to 16 feet below the existing ground surface. Groundwater levels fluctuate from tidal influences.
- Underground obstructions consist of the piles supporting the Pechanga Area (Group Delta, 2023), utilities, remnant building foundations, and a historic dump site (Group Delta, 1999).
- The buildings proposed for the redevelopment have high structural loads that will require, individually or combined, ground improvement and/or deep foundations to provide satisfactory long-term performance. Settlement of the fill placed to raise the site will influence design and construction of the infrastructure, such as underground utilities.

The remainder of this report presents recommendations for civil and structural design and earthwork construction. These recommendations are based on empirical and analytical methods typical of the standards of practice in southern California and San Diego area construction methods. They consider our current understanding of the project design. Revisions may be needed for changes to the redevelopment, the detailed design phase, and changes in expected construction processes and/or subsurface conditions exposed during construction. If these recommendations do not address a specific feature Group Delta can prepare revisions.

7.0 GROUND IMPROVEMENT AND EARTHWORK CONSTRUCTION RECOMENDATIONS

7.1 Ground Improvement

Considering prior projects nearby, Vibro-Replacement Stone Columns and Deep Soil Mixing should be the most likely types of ground improvement to allow for conventional shallow foundations (Group Delta, 2023). The purposes of ground improvement are to:

- Mitigate soil liquefaction and secondary effects, such as settlement and pile downdrag.
- Increase the Site Class for seismic design to reduce seismic demands on the structures.
- Increase the allowable bearing pressure and reduce the static settlement.

Geotechnical and Structural Engineers should develop performance objectives for the ground improvement. A Ground Improvement Specialty Contractor should select the type of ground improvement and design it to meet the performance objectives considering the soil and groundwater conditions and the potential for soil liquefaction. A pilot study is often an early construction activity to confirm the final design. The Geotechnical Engineer should develop a project-specific specification with vetting by the project team to procure the design and construction of ground improvement.

For preliminary planning purposes, the ground improvement needed to fully mitigate soil liquefaction and secondary effects and increase the Site Class for seismic design would extend to a depth of about 60 feet below existing grades (-50 feet NGVD 29) and be installed at least 20 feet horizontally outside of the plan limits of the structure to be protected from liquefaction.

7.2 Earthwork

Earthwork should consist of demolition and removal of existing structures and abandoned utilities, removal of existing soils as recommended in this report, replacement and recompaction of the removed existing soils with soils that are suitable for reuse as recommended in this report, and the placement and compaction of new fill to raise the site. Earthwork should also consist of importing soils needed to raise the site, installing new underground utilities, and excavating and exporting soils generated from ground improvement and piling that will mostly occur below groundwater.

Earthwork should be conducted per the current applicable requirements of the City of San Diego, the California Building Code, and the project specifications (that will be prepared). This report provides recommendations for specific aspects of the earthwork, which may need to be revised based on the conditions observed during construction.

7.2.1 General Site Preparation

General site preparation should begin with the removal of deleterious materials, such as landscaping and topsoil; demolition debris, such as existing structures, foundations, concrete slabs, and asphalt concrete that are not recycled as new construction materials; and expansive soils

(Expansive Index greater than 50). Areas disturbed by demolition should be restored with a subgrade that is stabilized to the satisfaction of the Geotechnical Engineer.

Piles below the existing Pechanga stadium should be cut down at least 5 feet below the lowest planned excavation for utilities or other infrastructure requiring excavation. In areas where no excavations are planned, the piles should be cut down at least 5 feet below finish grade. The cut off portion of the pile should be disposed of offsite.

Existing subsurface utilities that will be abandoned should be removed and the excavations backfilled and compacted as recommended in this report. Alternatively, abandoned pipes may be grouted using a two-sack sand-cement slurry under the observation of the Geotechnical Engineer.

Areas to receive fill should be scarified 12 inches and recompacted to 90 percent or more of the maximum dry density based on ASTM D1557. In areas of saturated or “pumping” subgrade, a bi- or tri-axial geogrid may be placed directly on the excavation bottom, and then covered with at least 12 inches of ¾-inch aggregate base. Once the subgrade is firm enough to attain compaction in the aggregate base, the remainder of the excavation may be backfilled. It may be necessary to place additional aggregate base to stabilize the subgrade sufficiently to place fill. The placement of the geogrid and aggregate base should also follow the specific installation guidelines from the manufacturer of the geogrid. Note that it may be necessary to use crushed rock (¾-inch) completely wrapped in filter fabric (Mirafi 140N, or approved equivalent) where stabilization occurs at elevations where groundwater may rise to in the future (tidally or long term).

7.2.2 Remedial Earthwork

Remedial earthwork that requires removing existing soils and replacing them with properly processed and recompacted soils is recommended prior to placing new fill, structures, slabs-on-grade, roadways, and exterior surface improvements. The purposes of remedial earthwork are to:

1. Provide a uniform surface to place fill or to construct new surface improvements due to the uncontrolled nature of the existing fill soils.
2. Allow for observation of unsuitable soils (clayey, wet, loose) in the exposed subgrade.
3. Reestablish subgrades that are disturbed by the ground improvement operations (if adopted).

The soils removed from remedial earthwork may be recompacted provided it is processed as recommended in the *On-Site Soils and Materials Management* section of this report. The existing soils should be removed and replaced with compacted fill to a depth that is three feet below:

1. The existing surface levels (following removal of existing hardscaped surfaces) in proposed fill areas or in areas where minimal grade changes are proposed.
2. The proposed subgrade levels in cut areas.
3. The grade from which ground improvement has been performed.

The recommendation does not consider the following factors that could increase the depth of soil removal:

- Some areas may require additional soil removal considering the disturbance caused by demolition or removal of contaminated soils.
- The undocumented fill soils may possess increased physical variability and consequently increase the need for deeper removal.
- The variability inherent in native subgrades where there may be loose and/or soft areas.
- The findings from additional subsurface exploration and/or observations by the Geotechnical Engineer during earthwork.
- Planned hardscape, graded paths, pavements, concrete slabs, and structural improvements in the park sites could require additional removal for subgrade preparation.

The level of groundwater during remedial earthwork may hinder the ability to achieve the recommended depth of soil removal. The Geotechnical Engineer can provide specific guidance, if and where this condition occurs.

7.2.3 Fill Placement and Compaction

All fill and backfill should be placed at slightly above optimum moisture content using equipment that can produce a uniformly compacted product. The 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 recommended relative compaction is 90 percent or more, or 95 percent or more as specified in Table 1, of the maximum dry density based on ASTM D1557.

A two-sack sand and cement slurry may also be used for structural fill as an alternative to compacted soil. Slurry is often useful in confined areas that may be difficult to access with typical compaction equipment. Samples of the slurry should be fabricated and tested for compressive strength during construction. A 28-day compressive strength of 100 pounds per square inch (psi) or more is recommended for the sand and cement slurry. Crushed rock (¾-inch) completely wrapped in filter fabric (Mirafi 140N, or approved equivalent) may also be used as backfill in confined areas.

7.2.4 On-Site Soils and Materials Management

The following existing soils and materials are available for processing and reuse.

- Soil
- Asphalt Concrete (AC)
- Portland Cement Concrete (PCC)

The following sections provide recommendations for processing and reuse as fill. During earthwork, soil types may be encountered by the Contractor that do not conform to those discussed within this report. The Geotechnical Engineer should evaluate the suitability of these soils for their proposed use.

7.2.4.1 Soil - Geotechnical

Most of the existing soils above groundwater should be suitable for reuse from a geotechnical standpoint. Table 1 provides material requirements for on-site soils to be used as fill. Soil excavated from below groundwater may not be suitable for reuse. Earthwork contractors may not want to use these soils due to the extra handling and processing needed to dry them for placement and compaction.

7.2.4.2 Asphalt Concrete

Existing asphalt concrete should be crushed to less than 3 inches in maximum dimension and blended with approved fill soils provided this is considered acceptable by the project environmental consultant. Existing asphalt concrete can also be recycled, reprocessed, and reused as a base course for new asphalt concrete paving. Alternatively, properly crushed asphalt concrete that is combined with crushed Portland Cement Concrete will often meet the gradation and quality criteria from Section 200-2.5 of the Standard Specifications for Public Works Construction for use as Processed Miscellaneous Base (PMB). Paving fabric could preclude reusing asphalt concrete. We did not observe this fabric in the limited opportunity for observation provided by drilling.

7.2.4.3 Portland Cement Concrete

Concrete may be crushed to less than 3 inches in maximum dimension for use as fill. It should be added to other soils to create a well graded fill material. Reinforcing steel should be removed prior to crushing the concrete. Properly crushed concrete will often meet the gradation and quality criteria from Section 200-2.4 of the Standard Specifications for Public Works Construction for use as Crushed Miscellaneous Base (CMB).

7.2.5 Import Soil

Import sources should be observed and tested by the Geotechnical Engineer prior to hauling onto the site to determine the suitability of the soils for use. For each proposed fill source, the Contractor should provide a submittal to the Geotechnical Engineer demonstrating the proposed site and materials meet the geotechnical guidelines for import and use as indicated in Table 1. The following screening tests should be performed for every 1,000 cubic yards of import, with a minimum of two sets of screening tests for each import site:

- Particle Size Distribution (ASTM D6913)
- Maximum Density (ASTM D1557)
- Expansion Index (ASTM D4829)
- Sulfate Content (ASTM D516)
- Chloride Content (ASTM D512)
- pH & Resistivity (CT 643)

The import soil testing frequency may be reduced by the Geotechnical Engineer if a long-term, steady source of import soils are used that consistently meets the requirements in Table 1.

8.0 STRUCTURAL DESIGN RECOMMENDATIONS

8.1 Seismic Design

The site classification for seismic design is Site Class F because the soils are susceptible to liquefaction and the potential for liquefaction triggering is widespread. The 2022 California Building Code and ASCE 7-16 require developing site-specific ground motions using site response analyses for Site Class F soils to capture the impact of liquefaction on the ground shaking, with one exception: relatively stiff structures with a fundamental period of 0.5-seconds or less. Structures meeting this exception may be classified as they would in the absence of liquefaction, which would be Site Class D considering the average shear wave velocity measured in the upper 100 feet at this site (602 ft/s to 688 ft/s). Site Class D may be adopted if ground improvement is completed over the entire building area to mitigate the potential for liquefaction.

For preliminary design purposes, assuming either ground improvement is performed or the exception for structures with fundamental periods of 0.5-seconds or less is met, the mapped values listed in the table below may be used for Site Class D. These are provided using the exception listed in Section 11.4.8 of Supplement 3 of ASCE 7-16, which states for structures on "Site Class D site with S_1 greater than or equal to 0.2" that a ground motion hazard analysis is not required where the mapped value of S_{M1} is increased by 50%. The parameters tabulated below were developed using the referenced ASCE 7 Hazard Tool online (ASCE, 2023).

MAPPED SEISMIC DESIGN ACCELERATION PARAMETERS (ASCE 7-16)

Design Parameters	Mapped Value
Site Latitude	32.75345
Site Longitude	-117.20699
S_S (g)	1.465
S_1 (g)	0.503
Site Class	D
F_a	1.0
F_V	1.797
T_S (sec)	0.925 [1]
T_L (sec)	8
S_{MS} (g)	1.465
S_{M1} (g)	1.356 [1]
S_{DS} (g)	0.977
S_{D1} (g)	0.904[1]

1: S_{M1} has been increased by 50% per ASCE 7-16 Supplement 3, which also impacts the value of T_S. F_V is based on Table 11.4-2.

In addition, although requirements for site response analyses at liquefiable sites remain the same in future codes (such as ASCE 7-22 and the future 2025 CBC) the general Site Classes for seismic design will change. Based on measured shear wave velocities, the site would be Site Class DE ($V_{S,30}$ between 500 and 700 ft/s) per ASCE 7-22 (ASCE, 2023). As some of the proposed structures may not be designed for some time, we are providing these values for future consideration. Note the same limitations apply – these values are only valid assuming that the structures have fundamental periods less than 0.5 seconds, or that ground improvement is completed to mitigate liquefaction. The parameters below were obtained from the ASCE 7 Hazard Tool online (ASCE, 2023).

MAPPED SEISMIC DESIGN ACCELERATION PARAMETERS(ASCE 7-22)

Design Parameters	Mapped Value
Site Latitude	32.75345
Site Longitude	-117.20699
S_S (g)	1.62
S_1 (g)	0.50
Site Class	DE
T_L (sec)	8
S_{MS} (g)	1.54
S_{M1} (g)	1.47
S_{DS} (g)	1.03
S_{D1} (g)	0.98

8.2 Shallow Foundations

Continuous strip and isolated pad footings may be used for lightly loaded buildings and other similar appurtenances where: 1) they can satisfactorily tolerate the estimated static and liquefaction-induced settlement per ASCE 7-16, or 2) it is acceptable to repair the damage caused by the settlement, and 3) they are not needed for primary ingress/egress or other essential functionality. The above recommendations assume that at least two feet below the bottom of the footing have been removed and recompacted. Strip and pad footings may be designed using the following parameters and recommendations.

- Allowable vertical bearing capacity of 1,000 pounds per square foot (psf). This parameter considers controlling static differential settlement within horizontal distances of 30 to 40 feet to ½-inch or less.
- Allowable lateral bearing capacity using an equivalent fluid weight of 250 pounds per cubic foot for footings above groundwater that are poured neat against properly compacted fill. The upper 12 inches of material in areas that are not covered with concrete slabs or pavements should not be included in the estimation of allowable lateral bearing.

- Bearing capacity and passive pressure may be increased by one-third for short term seismic and wind loads.
- Footing embedment and width as shown in Figure 9, Shallow Foundation Dimension Details.

8.3 Deep Foundations

Deep foundations use piles to transmit structure loads through the fill and upper paralic estuarine deposits that have a very low soil shear strength to the lower paralic estuarine deposits and old paralic deposits that have a high enough soil shear strength to provide geotechnical resistance. Based on the type of piles recently adopted at prior nearby projects (Group Delta, 2023), Appendix C provides preliminary recommendations for 18- and 24-inch diameter Drilled Displacement Piles (DDP). It may be necessary to adopt Auger-Cast-In-Place (ACIP) piles if larger diameters are needed to resist lateral loads.

DDP piles use a drill tool that is proprietary to the piling contractor to advance the hole and displace the soil into the ground. They do not generate significant amounts of spoil. ACIP piles use a continuous flight auger to advance the hole and remove the soil.

Driven precast concrete piles are also suitable. We have not considered them further because of the noise associated with driving and the current piling contracting industry’s more prevalent use of DDP and ACIP piles.

8.3.1 Axial Capacity

The piles derive axial capacity from shaft resistance and end bearing within the lower paralic deposits and old paralic deposits. Per ASCE 7-16, no capacity is derived from the fill and the upper paralic deposits due to the potential for liquefaction. Appendix C provides downward and upward pile capacities versus embedment and the assumptions used to estimate these capacities.

8.3.2 Static Settlement

Single isolated piles loaded to the allowable axial capacities should experience less than ½ inch of total settlement. Settlement should occur when building loads are applied.

8.3.3 Downdrag

Downdrag is the downward load resulting from friction along the soil-pile interface that is generated from settlement of the soils surrounding the pile. ASCE 7-16 Section 12.13.9.3.1 states the following regarding liquefaction-induced downdrag (ASCE, 2017):

Design of piles shall incorporate the effects of downdrag caused by liquefaction. For geotechnical design, the liquefaction-induced downdrag shall be determined as the downward skin friction on the pile within and above the liquefied zone(s). The net geotechnical ultimate capacity of the pile shall be the ultimate geotechnical capacity of the below the liquefiable layer(s) reduced by the downdrag *load. For structural design, downdrag load induced by liquefaction shall be treated as a seismic load and factored accordingly.*

The Structural Engineer should include liquefaction settlement induced downdrag loads at the pile head. Piles that support buildings where fill will be placed should be installed after settlement of the underlying soils is substantially complete to avoid additional static settlement-induced downdrag loads on the piles. Appendix C provides a summary table with the recommended downdrag loads as well as downward ultimate pile capacities versus embedment that have been adjusted to account for the liquefaction induced downdrag loads.

8.3.4 Lateral Capacity

Resistance to lateral loads can be estimated using the passive soil pressure against the pile caps and grade beams above the design groundwater level and the bending resistance of the piles. The passive pressure at the pile caps and grade beams is dependent on the depth of these foundations, the allowable deflection of the structure, and the geotechnical engineering properties of the soil against these foundations. The bending resistance of a pile depends on its length, stiffness in the direction of loading, proximity to other piles, the degree of fixity at the head, the allowable deflection at the pile head, and the geotechnical engineering properties of the soil surrounding the pile. Specific recommendations and preliminary design parameters are provided in Appendix C. *Group Delta should be contacted for revised recommendations if the pile caps are deeper than stated in Appendix C. The lateral capacity is highly influenced by the depth of the pile cap relative to the depth of the potentially liquefiable soils.*

8.4 Interior Reinforced Concrete Slabs

A slab-on-grade may be adopted with: 1) confirmation of the estimated static settlement and duration using the settlement monitoring discussed in the *Construction on Compressible Soils* section of this report, 2) the removal and recompaction recommended in the *Remedial Earthwork* section of this report, and 3) the acceptance of the potential for some local repairs to the slab-on-grade from liquefaction-induced settlement discussed in the *Liquefaction and Secondary Effects* section of this report. A structural slab that does not rely on the support of the underlying soil subgrade should be adopted where all three of the above conditions cannot be met.

8.4.1 Soil Subgrade

The subgrade should be prepared as recommended in the *Remedial Earthwork* section of this report. Where expansive soils are encountered in the upper 24 inches of subgrade, which are soils with an Expansion Index greater than 20, we recommend removing and replacing them with properly compacted non-expansive soils (Expansion Index less than 20).

8.4.2 Thickness and Reinforcement

There are several chart solutions (ACI, 2006) to complete analyses to develop the slab-on-grade thickness and reinforcement. These charts use a modulus of subgrade reaction (k). We recommend using 150 pounds per cubic inch (pci) assuming the slab is underlain with compacted fill prepared as recommended in this report. Where software is used, the Geotechnical Engineer should review the specific input parameter needed and how it is applied in the software used by the Structural Engineer. A Structural Engineer should design the slab thickness, control joints, and reinforcement considering the type of support (structural or subgrade) and should conform to the requirements of the California Building Code.

8.4.3 Moisture Protection for Interior Slabs

The requirements for moisture protection should consider that the design groundwater level may be near the finished slab-on-grade/structural slab elevation. Moisture protection should comply with the requirements of the current California Building Code, American Concrete Institute (ACI 302.1R-15), and the desired functionality of the interior ground level spaces. The Architect typically specifies an appropriate level of moisture protection considering allowable moisture transmission rates for the flooring or other functionality considerations.

Moisture protection may be a “Vapor Retarder” or “Vapor Barrier” that use membranes with a thickness of 10 and 15 mil or more. ACI 302.1R-15 provides a flow chart to determine when and where these membranes should be used. Note the CBC specifies a Capillary Break, as defined and installed per the California Green Building Standards, with a Vapor Retarder.

9.0 CIVIL DESIGN RECOMMENDATIONS

9.1 Surface Drainage

Foundation and slab performance depend on how well surface runoff drains from the site. The ground surface should be graded so that water flows rapidly away from the structures and tops of slopes without ponding. The surface gradient needed to achieve this may depend on the planned landscaping. Planters and landscaped areas should be built so that water does not seep into the foundation, slab, or pavement areas. If roof drains are used, the drainage should be channeled by pipe to storm drains or discharged 10 feet or more from buildings. Irrigation should be limited to that needed to sustain landscaping. Excessive irrigation, surface water, water line breaks, or rainfall may cause perched groundwater to develop within the underlying soil.

9.2 Design Groundwater Elevation

The recommended design groundwater elevation is + 3 feet NGVD 29. This elevation may differ from groundwater levels that could be encountered during construction.

9.3 Storm Water Infiltration

Our preliminary recommendation is a “no infiltration” condition. The design groundwater elevation recommended in this report will be near to the bottom of the infiltration surface of the storm water Best Management Practices, and the site is underlain by more than 5 feet of fill, which would preclude using infiltration.

9.4 New Underground Utilities

The redevelopment will include new sewer, storm drain, water and fireline, and dry utilities. The following sections provide preliminary geotechnical recommendations.

9.4.1 Soil Loads

A soil unit weight of 130 and 68 pounds per cubic foot may be used to evaluate soil loads on pipes that are above and below the design groundwater elevation.

9.4.2 Uplift Pressures

Pipes and structures installed below groundwater will be subject to uplift pressures. Figure 10, Uplift Pressures for Underground Structures provides recommendations for calculating the groundwater uplift pressure and soil resistance to uplift for structures embedded below groundwater. The recommended factor of safety against uplift is 1.5 or more. Soil above the structure and the self-weight of the structure may be used as resistance against uplift.

9.4.3 Thrust Blocks

The passive soil pressure for the design of thrust blocks may be estimated using an equivalent fluid weight of 250 and 125 pounds per cubic foot for the portions of the thrust block that are above and below the design groundwater elevation. These passive pressures are allowable and assume a factor of safety of 1.5. The pressures are for static loading and level ground surface conditions. The upper 12 inches of material in areas without concrete slabs or pavement should not be included in the estimation of passive resistance.

9.4.4 Modulus of Soil Reaction

The modulus of soil reaction (E’) characterizes the stiffness of soil backfill placed along the sides of buried flexible pipelines. To evaluate deflection due to the load associated with trench backfill over the pipe, we recommend using 600 pounds per square inch (psi) assuming granular bedding material is placed around the pipe and the bedding is above groundwater (Hartley and Duncan, 1987). We can provide specific recommendations bedding materials placed below groundwater.

9.4.5 Pipe Bedding

Typical pipe bedding as specified in the Standard Specifications for Public Works Construction or City of San Diego Standard Drawings may be used. We recommend using a filter fabric separator (such as Mirafi 140N or an approved similar product) to completely envelope the open graded rock used for bedding and/or backfill where: 1) the alignment is within roadways or near settlement sensitive improvements (e.g., structures, flatwork), 2) the bedding material is below the design groundwater elevation, or 3) the pipe diameter is larger than 18 inches. The Geotechnical Engineer may waive the filter fabric separator based on the soil conditions observed in the trench.

9.5 Existing Utilities

The permissible depth of cover and settlement tolerances should be evaluated where new fill will be placed over underground utilities that will remain. The permissible depth of cover and settlement tolerances for construction traffic and equipment loads should also be evaluated.

9.6 Settlement of Utilities

The design and construction of new underground utilities, and existing underground utilities that will remain, will need to consider the static settlement caused from fill placement and the settlement caused by liquefaction, as discussed in the following sections. These utilities may also need to consider the potential for the differential settlement that could occur between different subgrades, such as the transition at the edge of ground improvement or between a pile supported structure and unimproved ground.

9.6.1 Static Settlement

New and existing underground utilities within or below new fill will experience some time dependent settlement. For new utilities, the effect of settlement should depend on the timing of their installation following the placement of fill. The estimated long-term static settlement and their duration for substantial completion are described in the *Compressible Soils* section of this report.

The Civil Engineer should evaluate the ability of utilities to tolerate the estimated long-term settlement. Some form of mitigation will be needed if the utility cannot tolerate these settlements. Mitigation could be delaying the installation until the settlement is substantially complete, preloading the utility alignment area prior to utility installation with a fill surcharge, using lightweight fill or geofoam above the utility instead of fill soil, or using ground improvement to reduce the compressibility of the soils underlying the pipe.

9.6.2 Liquefaction-Induced Settlement

Liquefaction induced settlement could damage pipelines. Liquefaction of soils can also cause flotation where there are empty pipes (e.g., sewer and storm drains) below groundwater. *Critical* pipelines that service a large number of people or could be a substantial hazard to human life in the event of failure, and *Essential* pipelines that must remain operable at all times require mitigation to withstand the effects of liquefaction.

The Civil Engineer should identify existing and proposed pipelines that must remain in operation following a seismic event and develop mitigation. Mitigation depends on the serviceability required (e.g., Critical or Essential), pipeline function (e.g., transmission, distribution, or laterals), and pipeline materials. The Seismic Guidelines for Water Pipelines (America Lifelines Alliance, 2005) provides chart solutions that relates these factors to liquefaction-induced settlement and the type of pipeline design. To use this flow chart, differential settlement may be assumed to be in the 6 *inches < Permanent Ground Deformation (PGD) ≤ 12 inches* category.

9.7 Exterior Surface Improvements

Exterior surface improvements consist of the following types of paving surfaces:

- Asphalt concrete paving for interior streets and parking.
- Portland cement concrete paving for vehicles, fire lanes, and the truck loading areas for the arena.
- Portland cement concrete paving for pedestrian sidewalks and enhanced pedestrian concrete, such as an exposed aggregate finish.

The recommendations below apply to the above exterior surface improvements, which is followed by recommendations that are specific to each type of improvement.

- The upper 24-inches of the subgrade should consist of soils with a “Very Low” potential expansion (Expansion Index less than 20).
- The upper 12 inches of all paving subgrades should be scarified immediately prior to constructing the paving, brought to slightly above optimum moisture content, and compacted to 95 percent or more of the maximum dry density per ASTM D1557.
- Aggregate Base, where specified, should also be brought to slightly above optimum moisture content and compacted to 95 percent of the maximum dry density. Imported aggregate base should conform to Section 200-2.2, Crushed Aggregate Base (Public Works Standards, Inc., 2021). Where onsite concrete and/or asphalt are crushed to produce aggregate base for exterior surface improvements, the base should conform to Section 200-2.4, Crushed Miscellaneous Base, or Section 200-2.5, Processed Miscellaneous Base, meeting the fine grading in Table 2001-2.4.2 (Public Works Standards, Inc., 2021).
- An R-Value of 10 has been assumed for the preliminary assessment of paving surfaces (where it is part of the design methodology). Based on our review of the geotechnical data, the subgrade R-Value within the upper 36 inches of subgrade could range from 10 to 30 assuming selective placement of fill near the finished subgrade. The design subgrade R-Value should be confirmed by R-Value testing of the subgrade soils during precise grading.

9.7.1 Asphalt Concrete Pavements

Preliminary pavement sections designed in accordance with the Caltrans Design Method, Topic 633.1 (Caltrans, 2018b) are summarized in the table below. A 20-year pavement design life was assumed for the analyses.

PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS

Traffic Index	Asphalt Section	Base Section
5.0	3 inches	9 inches
6.0	3 inches	13 inches
7.0	4 inches	15 inches
8.0	5 inches	16 inches
9.0	6 inches	18 inches
10.0	6 inches	22 inches

Asphalt concrete should conform to Section 203-6 and should be compacted to 91 and 97 percent of the Rice density per ASTM D2041 (Public Works Standards, Inc., 2021).

9.7.2 Portland Cement Concrete Paving

9.7.2.1 Vehicular Paving

Preliminary concrete pavement sections are provided below using the simplified design procedure of the Portland Cement Association, the Caltrans Highway Design Manual, and typical sections from the City of San Diego Standard Drawings as guidelines (Caltrans, 2018; City of San Diego, 2019; PCA, 1984). The methodologies generally adopt a 20-year design life. It was assumed that aggregate interlock would be used for load transfer across control joints. The subgrade materials were assumed to provide relatively “low” support. Vehicular PCC pavements should have a minimum flexural strength (modulus of rupture) of 600 psi. Based on the assumed Traffic Index, we recommend the following preliminary vehicular PCC pavement sections.

PRELIMINARY VEHICULAR PORTLAND CEMENT CONCRETE PAVEMENT SECTIONS

Traffic Index	Concrete Section	Base Section
5.0	6 inches	6 inches
6.0 to 7.0	7 inches	6 inches
8.0	8 inches	6 inches
9.0	8.5 inches	6 inches
10.0	9 Inches	6 inches

Crack control joints should be constructed for vehicular 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 a minimum No. 4 bars on 18-inch centers, each way. Reinforcing bars should be placed mid-height within the slab.

Samples of the concrete used in the new pavement areas should be collected by a qualified materials testing firm and tested for flexural strength per ASTM D78 (or CT523) to confirm that the minimum required flexural strength is achieved.

9.7.2.2 Exterior PCC Slabs and Sidewalk Paving

Exterior PCC slabs and sidewalks subjected to pedestrian and small maintenance vehicle traffic should be at least 4 inches thick and reinforced with 6x6-W2.9/W2.9 Welded Wire Fabric or rebar consisting of No. 3 bars on 18-inch centers, each way, placed securely at mid-height of the concrete section. Crack control joints should be placed on a maximum spacing of 10-foot centers, each way, for slabs, and on 5-foot centers for sidewalks. There should be adequate construction and control joints to control cracking per the latest guidance from the American Concrete Institute (ACI), Portland Cement Association or other similar guidelines. The minimum compressive strength for exterior PCC slabs and sidewalks should conform to current City of San Diego Standard Drawings or other similar guidelines.

10.0 CONSTRUCTION CONSIDERATIONS

10.1 General

Construction of the project will need to adapt to the geotechnical conditions at the site. Summarized below are the primary geotechnical-related construction considerations known at this time, followed by more comprehensive discussions of some of these considerations.

- Shallow groundwater may require soil stabilization and/or dewatering to construct the grade beams and pile caps, and underground utilities. The groundwater and soil conditions could create loose/soft sidewalls and bottom instability that could cause difficulties installing shoring and pipe bedding.
- Grade-supported heavy equipment such as cranes or drill rigs operating near the upper surface of the loose/soft and saturated fill and upper estuarine deposits may require a granular working mat to provide adequate bearing capacity during construction.
- The construction of piles will need to manage groundwater and very loose/soft soils.
- Time-dependent static settlement following placement of new fill may require a settlement waiting period prior to construction of settlement sensitive improvements, including new structures, utilities, pavements, and flatwork.
- Ground improvement pilot studies and/or pile load tests may be particularly needed to confirm the design since the presence of mica, organics, and/or seashells can influence the geotechnical engineering characteristics of the upper paralic estuarine deposits.

- A 10-foot thick fat clay layer that underlies the western portion of the site creates a potential for substantial variability in the duration for settlement to be substantially complete. A test fill should be considered in this area.

10.2 Earthwork

10.2.1 Excavation Characteristics

Trench excavation in the soil above groundwater is expected to encounter little difficulty using modern trenching machines or backhoes in good working order. Standard heavy earthmoving equipment should be able to mass excavate soil above groundwater with little difficulty. Trench and mass excavation near groundwater should be prepared to encounter loose sands and soft clay. Much of the fill soils are cohesionless and should be considered prone to caving and/or sloughing. There may be debris in the undocumented fill, which could be resistant to excavation and/or require disposal offsite.

10.2.2 Subgrade Characteristics

Subgrade stabilization may be needed where excavation near groundwater could cause yielding or “pumping” of the subgrade. The Contractor should consider using lightweight equipment when working immediately above groundwater and should anticipate the need for stabilization of the subgrade as recommended in the *General Site Preparation* section of this report.

10.3 Temporary Excavations

10.3.1 CAL/OSHA Soil Types

Temporary slopes will be needed to install shallow underground utilities and to construct footings, pile caps and grade beams. Trench boxes and shields, or timber and hydraulic shoring may be needed for deeper installations.

Based on the data interpreted from subsurface exploration, the design of these types of temporary slopes may assume Soil Type C for planning purposes. For trench boxes and shields or timber and hydraulic shoring, CAL/OSHA recommends a lateral earth pressure equal to 80H for Soil Type C (often referred to as Soil Type C-80), subject to the proprietary aspects of the system adopted. The Contractor should note the materials encountered in construction excavations could vary significantly across the site. This assessment of Soil Type is based on preliminary classifications of soils encountered in widely spaced explorations.

The design and construction of these systems along with their maintenance and monitoring during construction is the responsibility of the Contractor. The Contractor should have their Competent Person evaluate the subsurface conditions exposed during excavation to consider permissible temporary slope inclinations, loads and other measures as required by California OSHA (CAL/OSHA, 2018). A registered Civil Engineer will need to design a temporary slope that is 20 feet, or more, in height. The Competent

Person should also observe temporary excavations at regular intervals for maintenance and evidence of potential instability.

10.3.2 Dewatering

Continuous dewatering will be needed for some of the temporary excavations. Dewatering typically targets lowering the groundwater to a level that ranges from 3 to 5 feet below the planned temporary excavation bottom.

Groundwater was measured in subsurface explorations at depths ranging from 6 to 16 feet that correspond to elevations of 3.0 to -4.0 feet NGVD 29. Groundwater levels will fluctuate from tidal influence.

Widespread lowering of the groundwater level can cause settlement of the surrounding ground.

10.4 Construction on Compressible Soils

10.4.1 Settlement Waiting Period and Monitoring

Where improvements cannot tolerate the estimated long-term settlement from fill placement presented in the *Compressible Soils* section of this report, construction should be timed to begin when the settlement is substantially complete. Settlement monuments should be installed in fill areas where construction needs to be delayed. Monitoring should be completed using fluid level settlement devices or surface monument and pipe riser settlement devices and precise surveying per CTM 112 (Caltrans, 2012). Figure 11A, Settlement Monument Details–Surface Monument and Figure 11B – Settlement Monument Details–Riser Plate depict typical instrumentation. Monitoring should be completed per CTM 112 (Caltrans 2012) daily during fill placement and weekly thereafter until the settlement is substantially complete as evaluated by the Geotechnical Engineer.

10.4.2 Test Fill Embankment

A test fill embankment could be constructed and monitored to further evaluate the magnitude of settlement and the duration for it to be substantially complete. The test fill should be located in the area of large fill placement. The embankment should not be located above or near to existing utilities or other existing settlement sensitive infrastructure. Provided below are preliminary recommendations for the test fill.

- The embankment height should be one-half of the thickness of the expected fill placement or a minimum of 10 feet. More useful data will be obtained from larger test fill heights.
- The top of the embankment should be twice the width of the earthwork equipment needed for construction, but not less than 20 feet. The embankment width must permit the equipment to pass on both sides of the settlement monument riser pipe during fill placement. If needed, the top of the settlement monument riser pipe can be set back horizontally 5 to 10 feet from the crest of the embankment slope to facilitate equipment

access. More useful data will be obtained by placing the monument near the center of the embankment.

- The embankment can be constructed with side slopes inclined at 1:1 (h:v).
- The length of the embankment should be at least 100 feet.
- The configuration of the embankment should be as-built with precise surveying. The purpose of this recommendation is to calculate the embankment surcharge loading.
- The subgrade should be prepared as recommended in the *Site Preparation* section of the report. The lift thickness and compaction should be as recommended in the *Fill Placement and Compaction* section of this report. The purpose of this recommendation is to provide data to estimate the fill soil unit weight to calculate the embankment surcharge loading.
- There should be three settlement monuments. One monument should be in the center of the long axis of the embankment with the other two on either side of the center monument.
- Monitoring should be completed per CTM 112 (Caltrans, 2012). There should be daily monitoring during formation of the embankment and weekly monitoring thereafter until the settlement is substantially complete, as evaluated by the Geotechnical Engineer.

10.5 Pile Installation

10.5.1 Subsurface Conditions

The Piling Contractor that will install the planned Drilled Displacement piles should adopt methods that are suitable for installation through loose and soft soils below groundwater. Coring or similar means could be needed to install piles where underground obstructions are encountered. The Piling Contractor should independently review the exploration logs in this report to assess pile installation conditions. Any surface geophysical data, pot holing, as-built plans, and other similar information should be provided to the Piling Contractor.

10.5.2 Load Testing

Pile load testing should be adopted since the capacity analyses can be highly dependent on the assumptions regarding the method of installation. Drilled Displacement piles use a drill tool that is often proprietary to the Piling Contractor. Shaft resistance can vary substantially between different drill tools and grout pressures.

An Advance Pile Load Test (APLT) program is often completed where there is a desire to obtain additional information to further assess axial pile capacities and potentially reduce pile lengths; trial the method of pile installation for specific subsurface conditions; and establish production parameters such as drilling penetration rates, torque, and downward thrust. APLTs typically include strain gauges installed at various levels to interpret shaft resistance and end bearing. The

Geotechnical Engineer can provide guidance on the depth intervals for strain gauges and the pile test load. APLTs are typically completed on sacrificial piles.

Verification Production Pile Load Tests (VPLT) should be completed on the production piles. They may be one to two test piles, or a percentage of the production piles, depending on the size and sequencing of pile construction.

Pile load tests should be completed per the latest version of ASTM Standard D1143 / D1143M, Standard Test Methods for Deep Foundations Under Static Axial Compressive Load. The pile test load should include the liquefaction-induced downdrag load and account for the shaft resistance to be neglected in the undocumented fill and upper paralic estuarine deposits. The test piles should be installed using the same methods that would be used for production piling. An automated monitoring system should be used to monitor construction of the test piles. This same monitoring system should be used on all production piles to establish that construction of the test and production piles are similar, and that production piles will achieve performance that is the same as the test piles. The latest version of ASTM Standard D4945, Standard Test Method for High-Strain Dynamic Testing of Deep Foundations may be considered for VPLTs.

10.5.3 Construction Quality Control

Construction quality control should follow typical industry guidance, such as presented in Geotechnical Engineering Circular No. 8, Design and Construction of Continuous Flight Auger Piles (FHWA, 2007). Guidance is provided for observing pile installation and maintaining construction records, materials testing, nondestructive testing to evaluate pile integrity, and the determination and treatment of unsatisfactory piles. The Contractor should submit a pile load test plan and a production pile installation plan, which should be reviewed by the Geotechnical Engineer and Structural Engineer. There should be full time observation of pile construction by the Geotechnical Engineer along with automated monitoring of drilling and grouting.

10.6 Geotechnical Services During Construction

Geotechnical services during construction are anticipated to consist of the following activities:

- Continuous onsite observation and compaction testing by a Geotechnical Technician during earthwork with associated laboratory testing (e.g., compaction curves, physical and engineering properties of engineered fill and import soils, confirming R-Value tests).
- Full- and part-time observation and compaction testing by a Geotechnical Technician as needed during the backfill of underground utility trenches, the preparation of pavement subgrade and aggregate base, and the placement of asphalt concrete. Full time observation is needed when trench excavations are too deep to safely enter for compaction testing.
- Continuous observation of ground improvement pilot studies or pile load tests, and the production installation of ground improvement and piles by a Geotechnical Engineer.

- Observation by a Geotechnical Technician to observe that remedial grading removal bottoms extend to the correct depth and bearing strata is suitable.
- Observation by a Geotechnical Technician to observe that shallow foundation excavations have the correct plan dimensions and extend to the correct depth and bearing strata is suitable.
- Evaluation of settlement monitoring data by a Geotechnical Engineer. For this activity, the Geotechnical Engineer should be provided with timely copies of all survey monitoring data.
- Consultation by the Geotechnical Engineer for unforeseen conditions, responding to Requests for Information and Submittals, and attending construction coordination meetings.
- Preparation of an As-Built Geotechnical Report.

11.0 ADDITIONAL GEOTECHNICAL SERVICES

Development of the project will require further geotechnical services that are anticipated to consist of the following tasks:

- Conducting Site-Specific Probabilistic Seismic Hazard Analysis using site response analysis per the current version of the CBC and ASCE 7 to capture the impact of liquefaction on the ground shaking.
- Installing and measuring groundwater in monitoring wells to record the impact of daily tidal fluctuations and the seasonal variations of groundwater to better inform the recommended design groundwater level for the site.
- Completing additional cone penetration tests and geotechnical borings for changes in the redevelopment layout and as needed for the final design.
- Providing geotechnical consulting during the design development, construction document and permitting phases of the project.
- Preparing a project-specific specification with the site geotechnical information and design criteria to procure the design and construction of ground improvement.
- Preparing or supporting the preparation of geotechnical-specific construction specifications (e.g., earthwork, deep foundations).
- Reviewing the civil, structural, landscaping, and architecture (waterproofing only) plans for compatibility with the recommendations provided in the geotechnical report.
- Responding to comments by the reviewing agencies.
- Updating and finalizing this geotechnical report as needed to address changes in design, to obtain permits, and/or address comments from reviewing agencies.

12.0 LIMITATIONS

The recommendations in this report are subject to revisions for changes to the design and to accommodate changes in expected construction processes and/or subsurface conditions exposed during construction. Group Delta needs to continue to be part of the project design and construction for these recommendations to remain valid. If another geotechnical consultant provides these services, they should prepare a letter indicating their intent to assume the responsibilities of the project Geotechnical Engineer-of-Record. This letter should also indicate their concurrence with the recommendations in the report or revise them as needed to assume the role of the project Geotechnical Engineer-of-Record.

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.

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TABLE 1 - GEOTECHNICAL SPECIFICATIONS FOR COMPACTED FILL

Fill Type	**Location**	**Depth Ranges** [a]	**Material Recommendations** [b] **[Test Standard]**	**Minimum Compaction Recommendations [Test Standard]**
General	General	All	EI ≤ 50 [ASTM D4829] Passing 6” Sieve ≥ 100% [ASTM D6913] [c,d] Passing ¾” Sieve ≥ 70% [ASTM D6913]	90% RC at or slightly above OMC [ASTM D1557]
Heave-Settlement Sensitive Improvements Subgrade	Slabs-on-Grade, Structural Slabs, Pavements, Sidewalks, Curbs, Gutters	12” to 36” below FSG	EI ≤ 20 [ASTM D4829] Passing 3” Sieve ≥ 100% [ASTM D6913] Passing ¾” Sieve ≥ 70% [ASTM D6913] Passing #200 Sieve ≤ 35% [ASTM D6913]	90% RC at or slightly above OMC [ASTM D1557]
		Upper 12” below FSG		95% RC at or slightly above OMC [ASTM D1557]
Utility Trench Backfill	Bedding (i.e., Pipe Zone)	1’ above TOP to Bottom of Trench	See Geotechnical Report Text	90% RC at or slightly above OMC [ASTM D1557]
	Trench Zone	FSG to 1’ above TOP	EI ≤ 50 [ASTM D4829] Passing 3” Sieve ≥ 100% [ASTM D6913] Passing ¾” Sieve ≥ 70% [ASTM D6913]	

Notes:

a = If multiple zones overlap, the most stringent of the compaction and material recommendations should apply to that zone.

b = Additional Minimum Criteria that Apply to Material Recommendations:

- Satisfactory USCS Soil Types: GW, GP, GM, GC, SW, SP, SM, and SC, or combinations of these groups [ASTM D2487]
- Unsatisfactory USCS Soil Types: CH, MH, CL, ML, OH, OL and PT, or combinations of these groups [ASTM D2487]
- Corrosion Recommendations: Sulfate Content < 0.10%; Chloride Content < 0.03%; Minimum Soil Resistivity > 1,000 ohm-cm; 5.5 < pH < 10.0 [ASTM D516, CTM 643].

c = Fill material should be placed and processed to avoid "nesting" or concentrations of rock without sufficient fines for compaction.

d = Consider using Passing 3” Sieve ≥ 100% [ASTM D6913] to facilitate footing and utility trench excavations, subgrade scarification and preparation, and backfill.

ASTM = ASTM International; BOE = Bottom of Remedial Grading Excavation; BOF = Bottom of Foundation; BOW = Bottom of Wall; CTM = Caltrans Test Method; EI = Expansion Index; FSG = Finished Subgrade; OMC = Optimum Moisture Content; RC = Relative Compaction; RDS = Remolded Direct Shear; TOP = Top of Pipe; TOW = Top of Wall; USCS = Unified Soil Classification System.

FIGURES

REFERENCE: GOOGLE EARTH MAP, RETRIEVED ON 01-11-2023

0 1000' 2000'
1"

NOTE: SCALE, DIRECTION, AND LOCATIONS ARE APPROXIMATE

SPORTS ARENA COMPLEX
MIDWAY RISING

SITE LOCATION MAP

GROUP DELTA

PROJECT NUMBER
SD760

FIGURE NUMBER
1

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▭	APPROXIMATE LIMITS OF SITE DEVELOPMENT
⊕ A-23-016	APPROXIMATE LOCATION OF GROUP DELTA HOLLOW-STEM AUGER BORING
⊕ R-23-002	APPROXIMATE LOCATION OF GROUP DELTA MUD ROTARY WASH BORING
▲ CPT-23-028	APPROXIMATE LOCATION OF GROUP DELTA CONE PENETRATON TEST (CPT)
A A'	APPROXIMATE LOCATION OF CROSS SECTIONS
TD=106.2'	TOTAL DEPTH OF GROUP DELTA EXPLORATION

N

0 200' 400'

NOTE: DIRECTION, SCALE AND LOCATIONS ARE APPROXIMATE

REFERENCE: GOOGLE, INC (2023) GOOGLE EARTH PRO, AERIAL IMAGERY DATED: AUGUST 1, 2021

SPORTS ARENA COMPLEX
MIDWAY RISING

EXPLORATION LOCATIONS

GROUP DELTA

PROJECT NUMBER	FIGURE NUMBER
SD760	2

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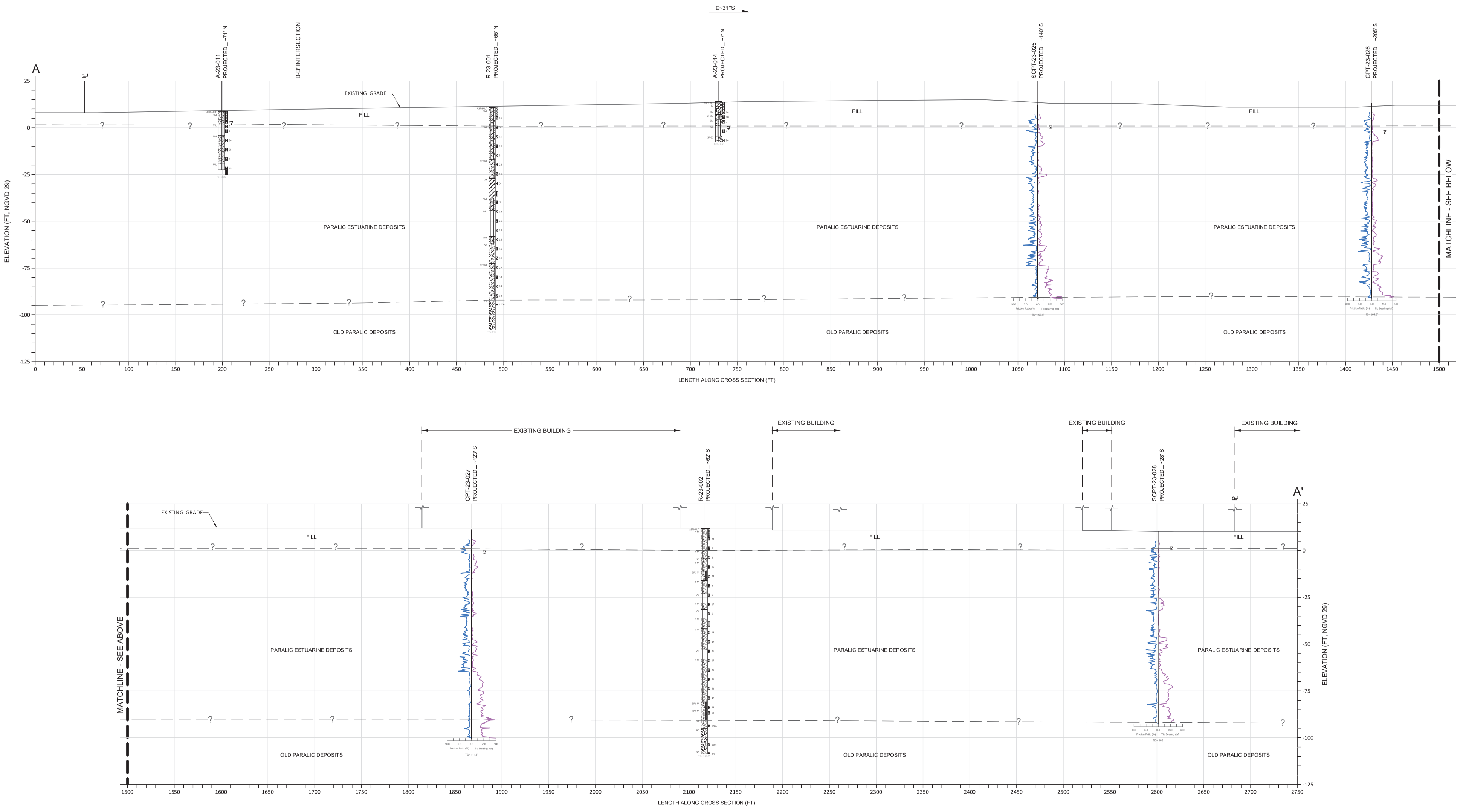
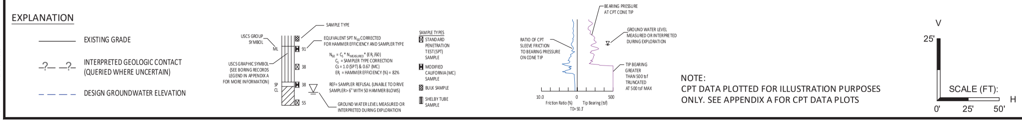


EXPLANATION



NOTE:
CPT DATA PLOTTED FOR ILLUSTRATION PURPOSES ONLY. SEE APPENDIX A FOR CPT DATA PLOTS

NOTE: DIRECTIONS, LOCATIONS, ELEVATIONS, AND SCALE ARE APPROXIMATE
REFERENCE: PRE-EXISTING TOPOGRAPHY FROM EXISTING UTILITIES AND EASEMENTS BASE MAP, PROJECT DESIGN CONSULTANTS (02/2023)

SPORTS ARENA COMPLEX
MIDWAY RISING

GEOTECHNICAL
CROSS SECTION A-A'

GROUP DELTA

PROJECT NUMBER	FIGURE NUMBER
SD760	3A

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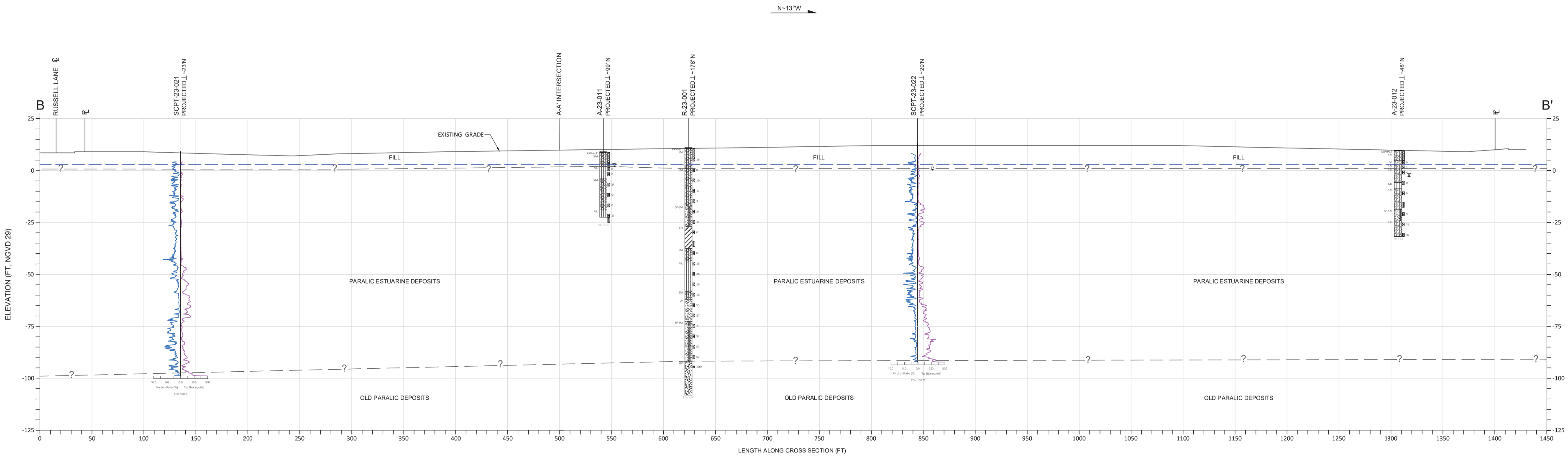

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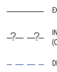
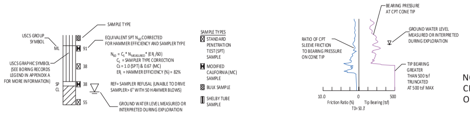

EXISTING GRADE

INTERPRETED GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)

DESIGN GROUNDWATER ELEVATION



NOTE:
CPT DATA PLOTTED FOR ILLUSTRATION PURPOSES ONLY. SEE APPENDIX A FOR CPT DATA PLOTS

V
25'
SCALE (FT):
0' 25' 50'
H

NOTE: DIRECTIONS, LOCATIONS, ELEVATIONS, AND SCALE ARE APPROXIMATE
REFERENCE: PRE-EXISTING TOPOGRAPHY FROM EXISTING UTILITIES AND EASEMENTS BASE MAP, PROJECT DESIGN CONSULTANTS (02/2023)

SPORTS ARENA COMPLEX
MIDWAY RISING

GEOTECHNICAL
CROSS SECTION B-B'

GROUP DELTA

PROJECT NUMBER	FIGURE NUMBER
SD760	3B

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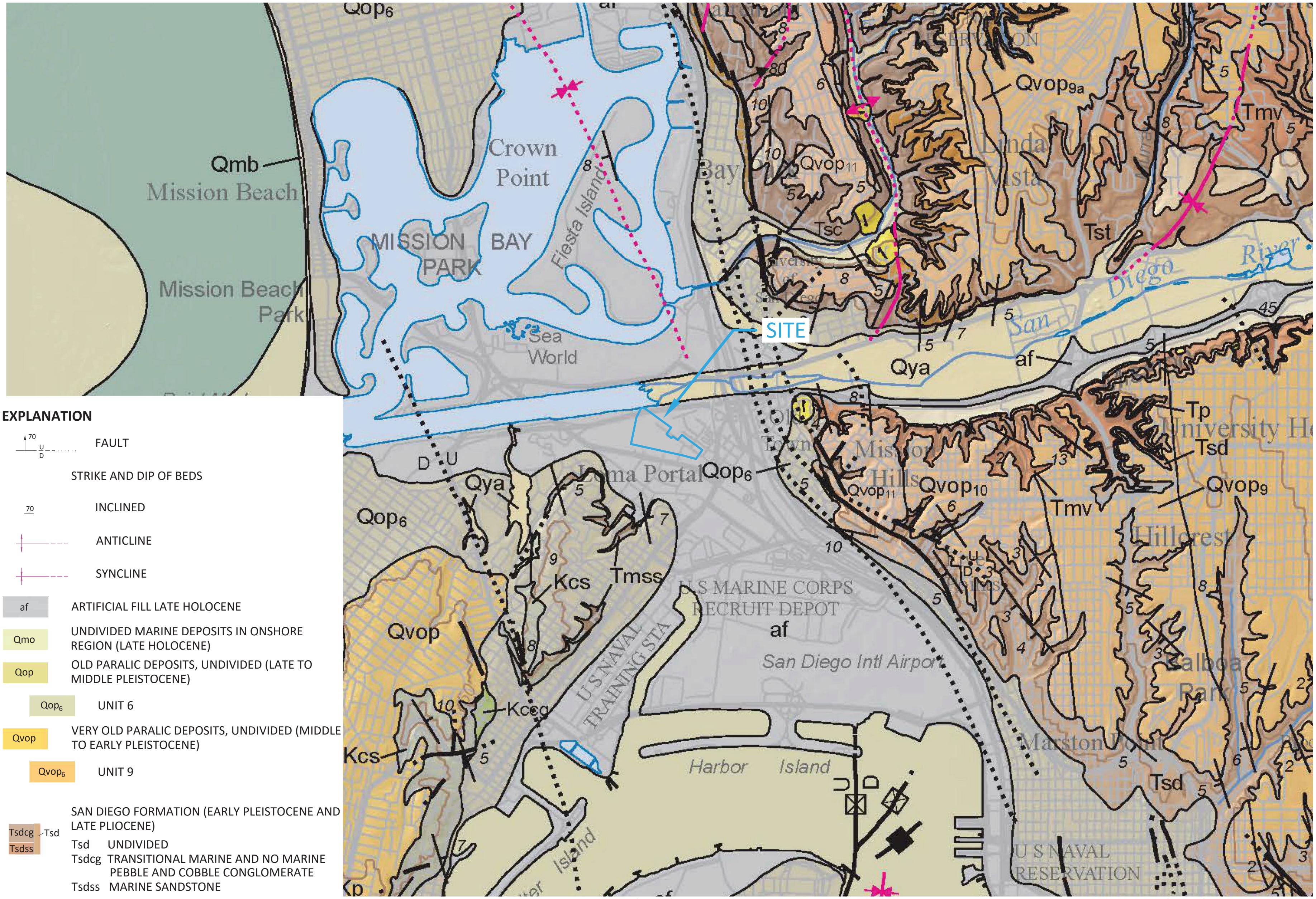

EXPLANATION

FAULT

STRIKE AND DIP OF BEDS

INCLINED

ANTICLINE

SYNCLINE

af	ARTIFICIAL FILL LATE HOLOCENE
Qmo	UNDIVIDED MARINE DEPOSITS IN ONSHORE REGION (LATE HOLOCENE)
Qop	OLD PARALIC DEPOSITS, UNDIVIDED (LATE TO MIDDLE PLEISTOCENE)
Qop6	UNIT 6
Qvop	VERY OLD PARALIC DEPOSITS, UNDIVIDED (MIDDLE TO EARLY PLEISTOCENE)
Qvop9	UNIT 9
Tsdcg / Tsdss – Tsd	SAN DIEGO FORMATION (EARLY PLEISTOCENE AND LATE PLIOCENE)

Tsd UNDIVIDED
Tsdcg TRANSITIONAL MARINE AND NO MARINE PEBBLE AND COBBLE CONGLOMERATE
Tsdss MARINE SANDSTONE

0 2500' 5000'
1"
N

REFERENCE: GEOLOGIC MAP OF THE SAN DIEGO 30' x 60' QUADRANGLE, CALIFORNIA BY MICHAEL P. KENNEDY AND SIANGS S. TAN (2008)

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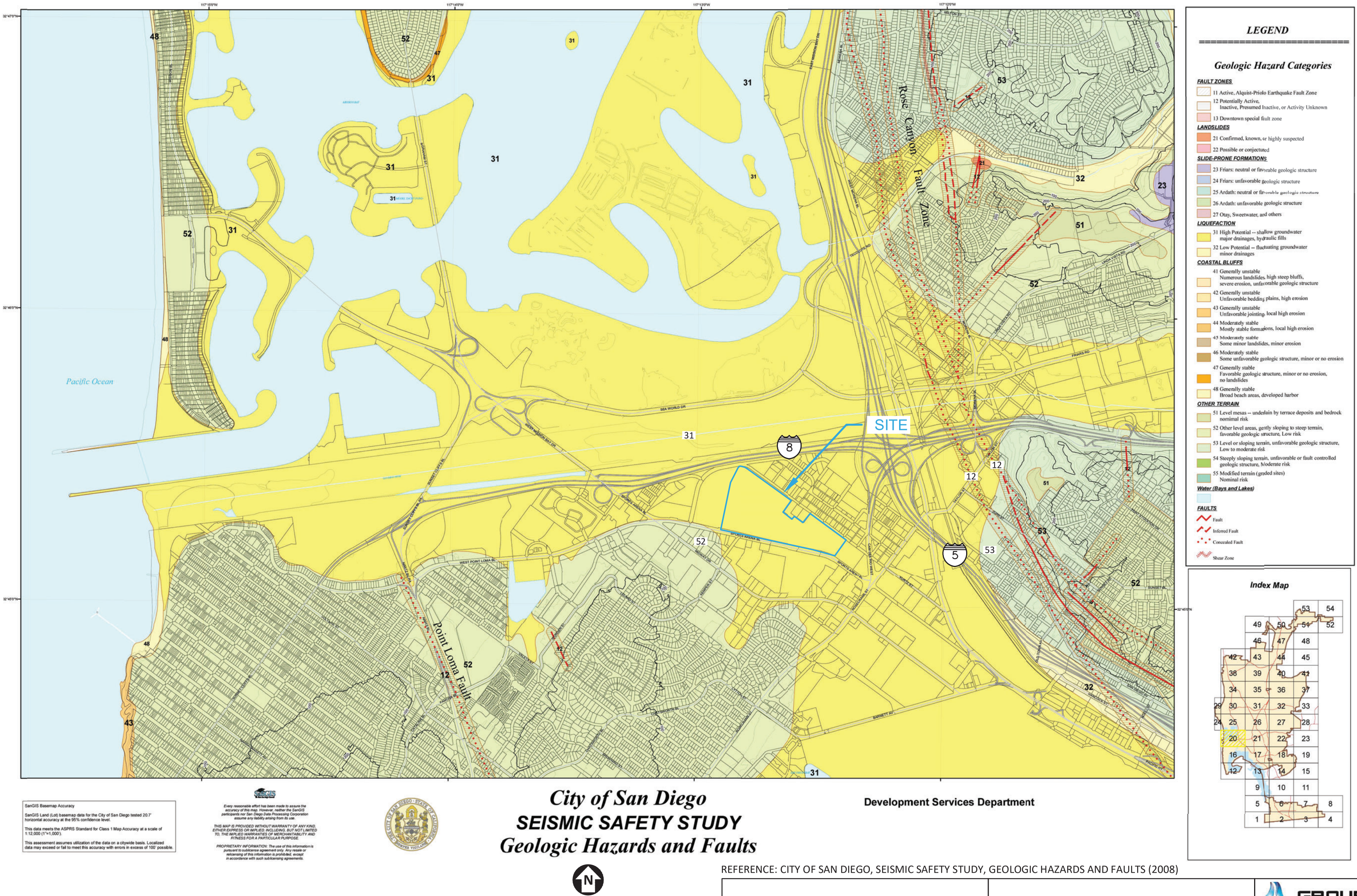
SPORTS ARENA COMPLEX
MIDWAY RISING

GEOLOGIC MAP

GROUP DELTA

PROJECT NUMBER	FIGURE NUMBER
SD760	4



City of San Diego
SEISMIC SAFETY STUDY
Geologic Hazards and Faults

Development Services Department

SanGIS Basemap Accuracy

SanGIS Land (Lot) basemap data for the City of San Diego tested 20.7' horizontal accuracy at the 95% confidence level.

This data meets the ASPRS Standard for Class 1 Map Accuracy at a scale of 1:12,000 (1"=1,000').

This assessment assumes utilization of the data on a citywide basis. Localized data may exceed or fail to meet this accuracy with errors in excess of 100' possible.

Every reasonable effort has been made to assure the accuracy of this map. However, neither the SanGIS participants nor San Diego Data Processing Corporation assume any liability arising from its use.

THIS MAP IS PROVIDED WITHOUT WARRANTY OF ANY KIND, EITHER EXPRESS OR IMPLIED, INCLUDING, BUT NOT LIMITED TO, THE IMPLIED WARRANTIES OF MERCHANTABILITY AND FITNESS FOR A PARTICULAR PURPOSE.

PROPRIETARY INFORMATION: The use of this information is pursuant to sublicense agreement only. Any resale or relicensing of this information is prohibited, except in accordance with such sublicensing agreements.

REFERENCE: CITY OF SAN DIEGO, SEISMIC SAFETY STUDY, GEOLOGIC HAZARDS AND FAULTS (2008)

SPORTS ARENA COMPLEX MIDWAY RISING	CITY OF SAN DIEGO SEISMIC SAFETY ELEMENT	GROUP DELTA	
		PROJECT NUMBER SD760	FIGURE NUMBER 5

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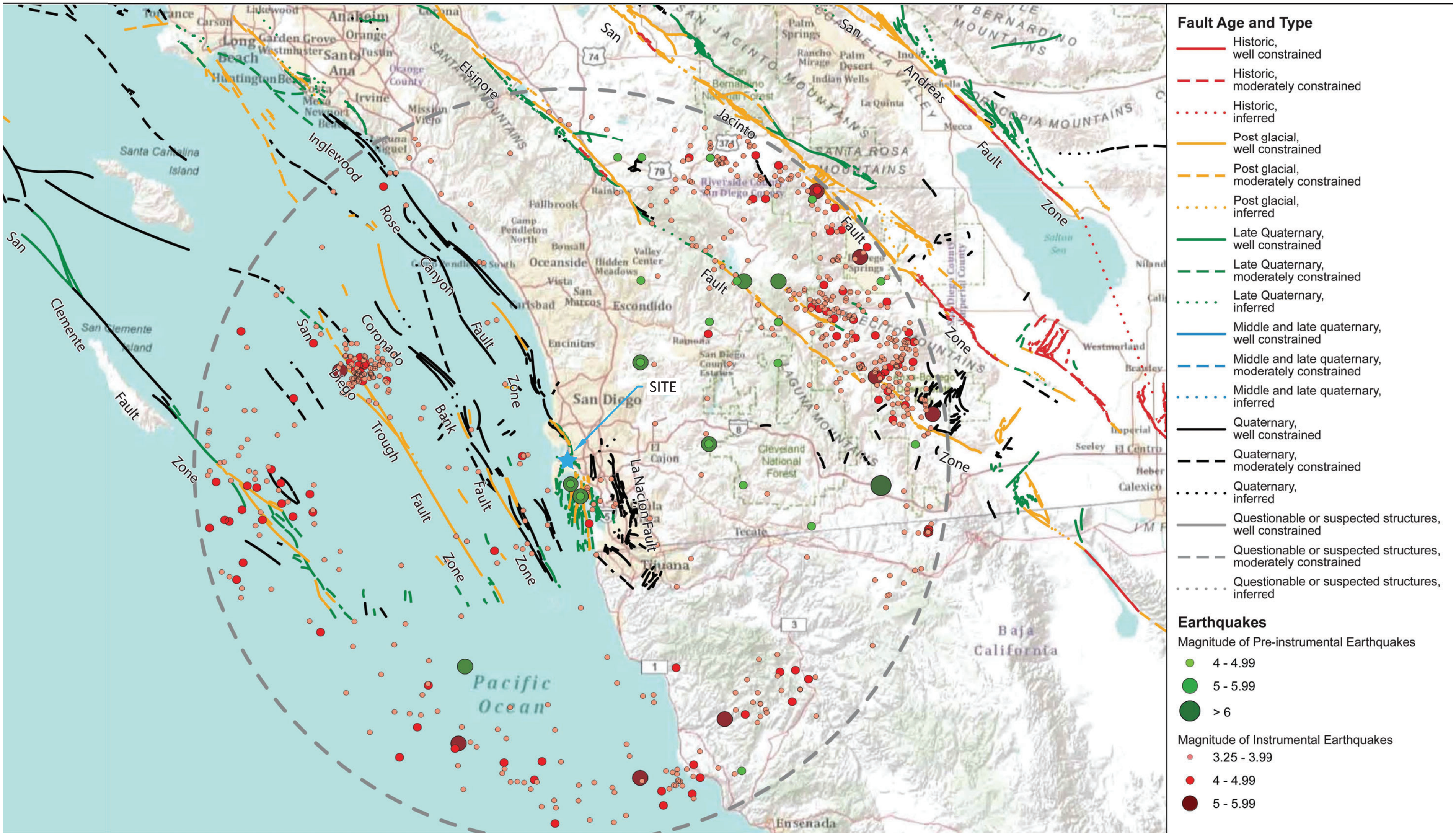


REFERENCE: USGS AND CGS, "SEISMIC AND GEOLOGIC TECHNICAL BACKGROUND REPORT FOR THE CITY OF SAN DIEGO MIDWAY-PACIFIC HIGHWAY AND OLD TOWN COMMUNITY PLAN UPDATES" (2010).

N

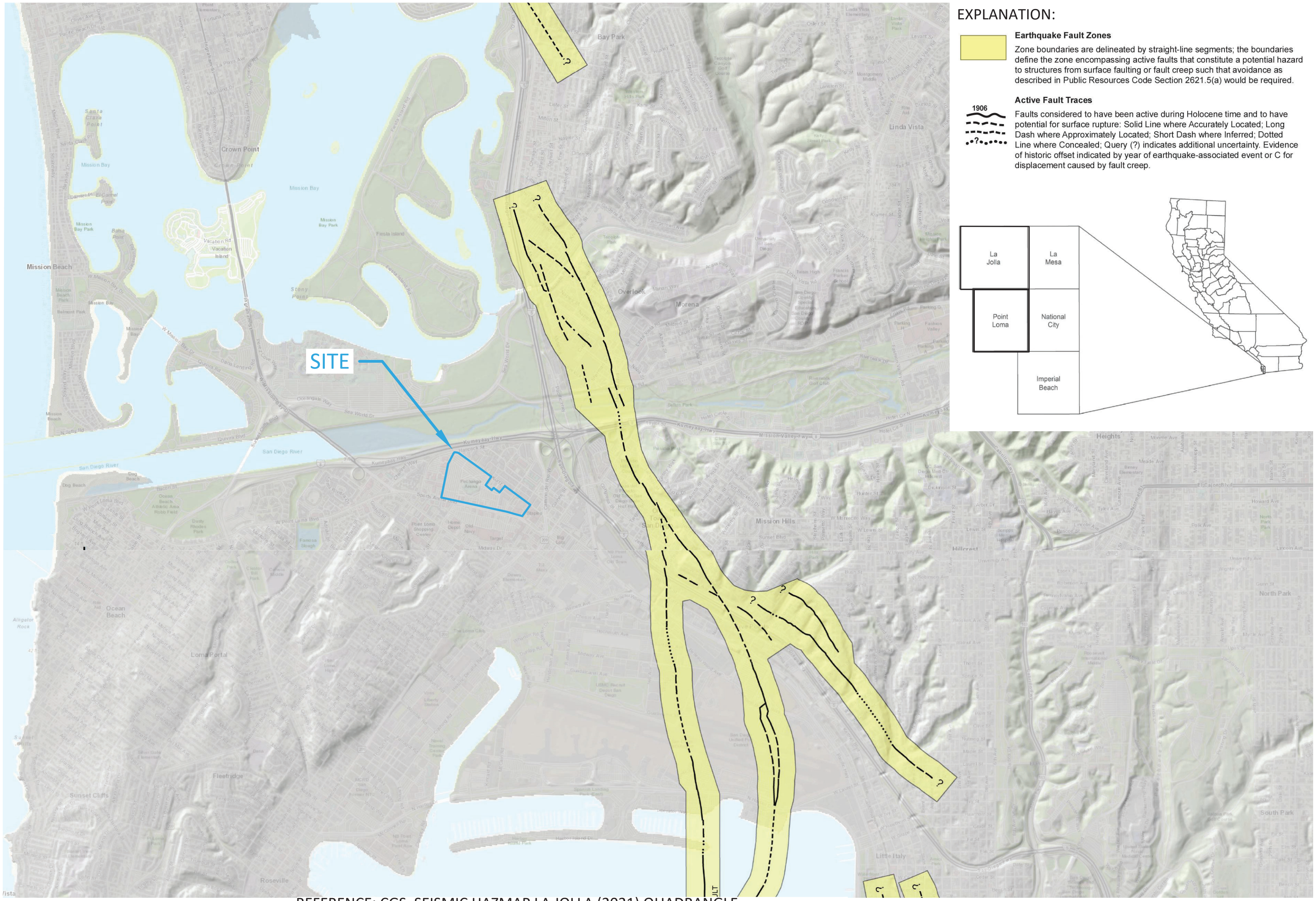
0 16 32 MILES

SPORTS ARENA COMPLEX
MIDWAY RISING

REGIONAL FAULTS AND
EARTHQUAKES MAP

GROUP DELTA

PROJECT NUMBER	FIGURE NUMBER
SD760	6



EXPLANATION:

Earthquake Fault Zones

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

Active Fault Traces

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

REFERENCE: CGS, SEISMIC HAZMAP LA JOLLA (2021) QUADRANGLE

0 2000' 4000'
1"

N

SPORTS ARENA COMPLEX
MIDWAY RISING

EARTHQUAKE ZONE
OF REQUIRED INVESTIGATION
LA JOLLA QUADRANGLE

GROUP DELTA

PROJECT NUMBER
SD760

FIGURE NUMBER
7

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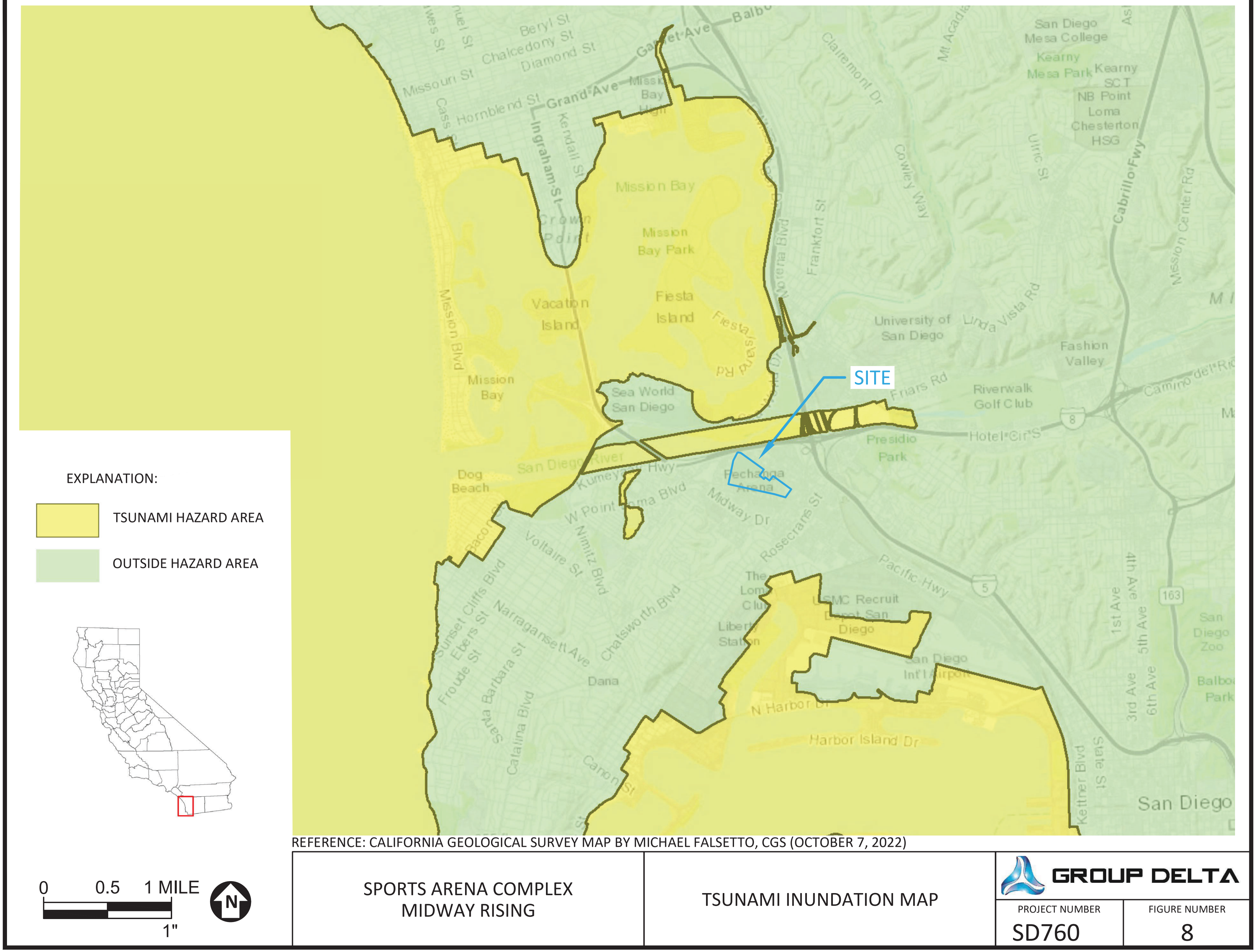

REFERENCE: CALIFORNIA GEOLOGICAL SURVEY MAP BY MICHAEL FALSETTO, CGS (OCTOBER 7, 2022)

SPORTS ARENA COMPLEX
MIDWAY RISING

TSUNAMI INUNDATION MAP

GROUP DELTA

PROJECT NUMBER
SD760

FIGURE NUMBER
8

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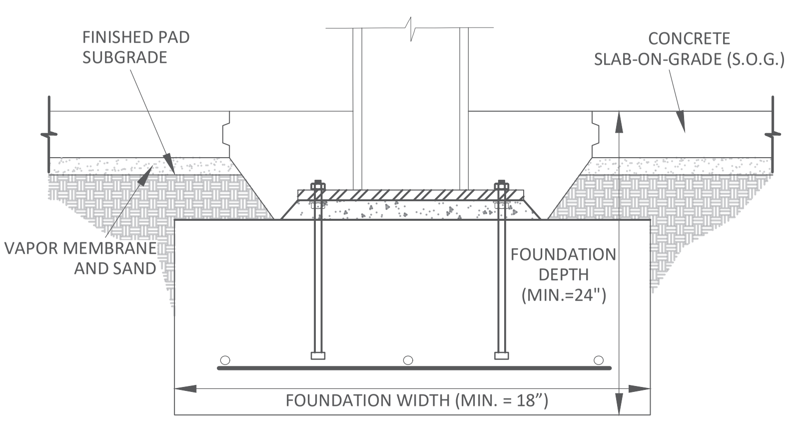
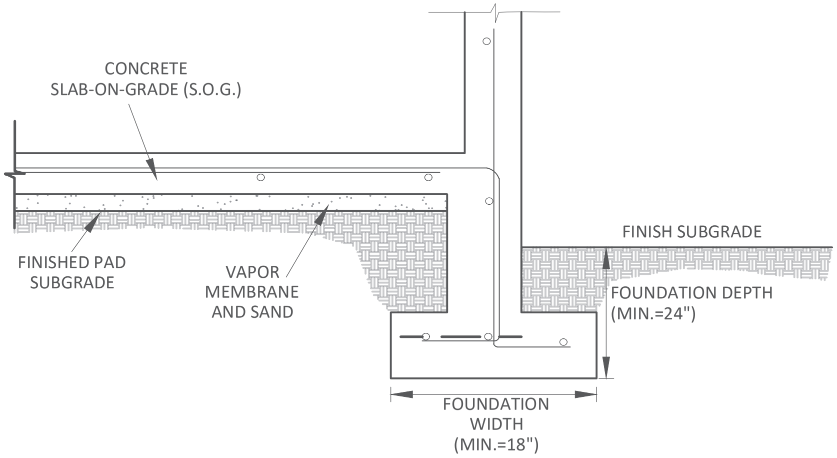
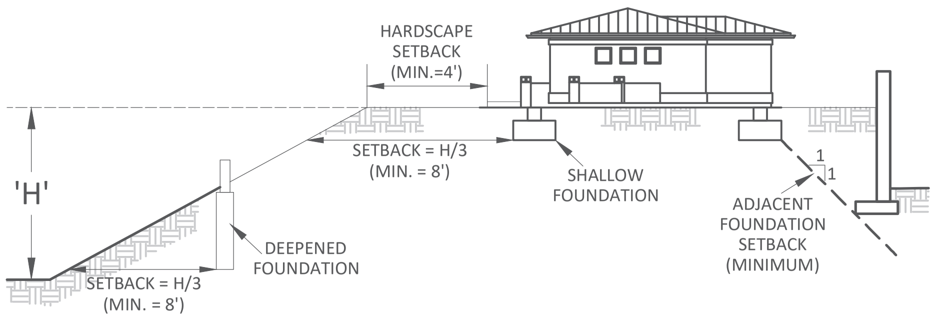


INTERIOR FOUNDATION Ⓐ



PERIMETER /EXTERIOR FOUNDATION Ⓑ



SLOPE AND ADJACENT FOUNDATION SETBACKS Ⓒ

NOTES:
1) FOUNDATION REINFORCING AND SIZING PER STRUCTURAL ENGINEER. (SHOWN FOR ILLUSTRATION PURPOSES ONLY)
2) VAPOR MEMBRANE AND SAND PER ARCHITECT. (SHOWN FOR ILLUSTRATION PURPOSES ONLY)

NOT TO SCALE

SPORTS ARENA COMPLEX
MIDWAY RISING

SHALLOW FOUNDATION
DIMENSION DETAILS

GROUP DELTA

PROJECT NUMBER
SD760

FIGURE NUMBER
9

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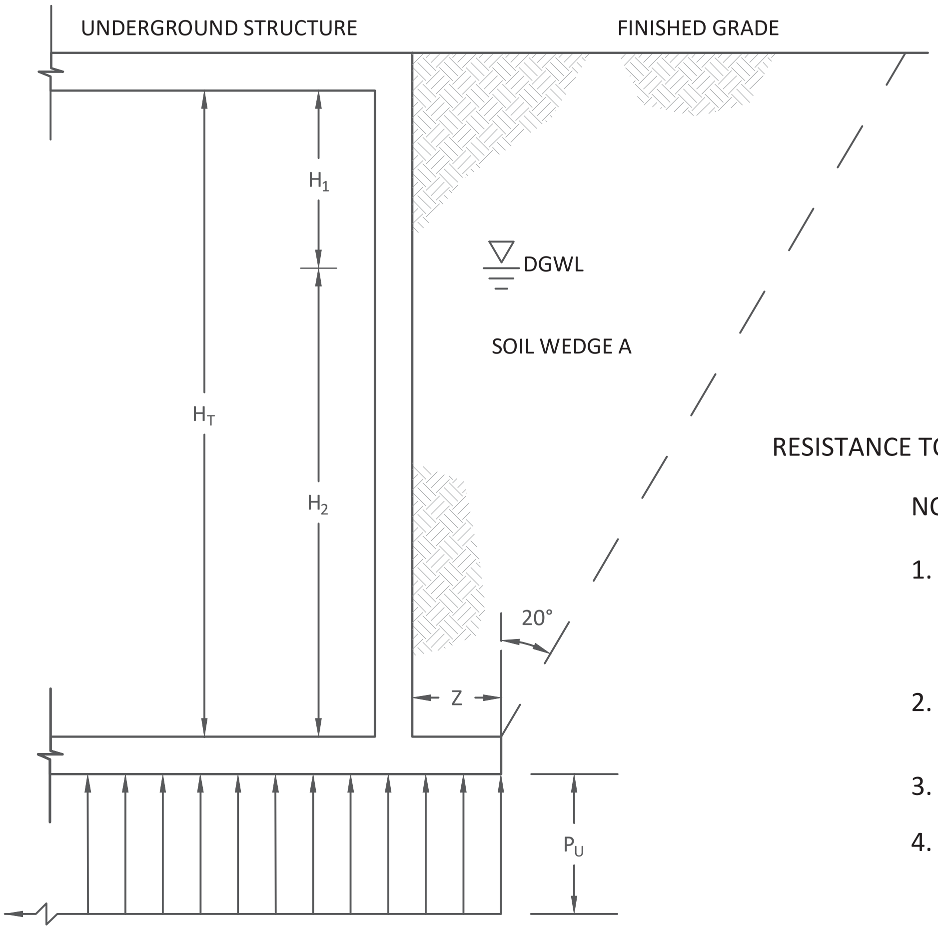

RESISTANCE TO UPLIFT = WEIGHT OF STRUCTURE + WEIGHT OF SOIL WEDGE A

NOTES:

1. UNIT WEIGHT OF SOILS, γ OR γ_B
   γ = 120 PCF ABOVE GROUNDWATER TABLE
   γ_B = 58 PCF BELOW GROUNDWATER TABLE
2. UPLIFT PRESSURE, P_U
   $P_U = 63H_2$ PSF
3. H_T, Z, H_1 AND H_2 ARE IN FEET
4. DGWL: DESIGN GROUNDWATER LEVEL PER GEOTECHNICAL REPORT

NOT TO SCALE

SPORTS ARENA COMPLEX
MIDWAY RISING

UPLIFT PRESSURES
FOR UNDERGROUND STRUCTURES

GROUP DELTA

PROJECT NUMBER	FIGURE NUMBER
SD760	10

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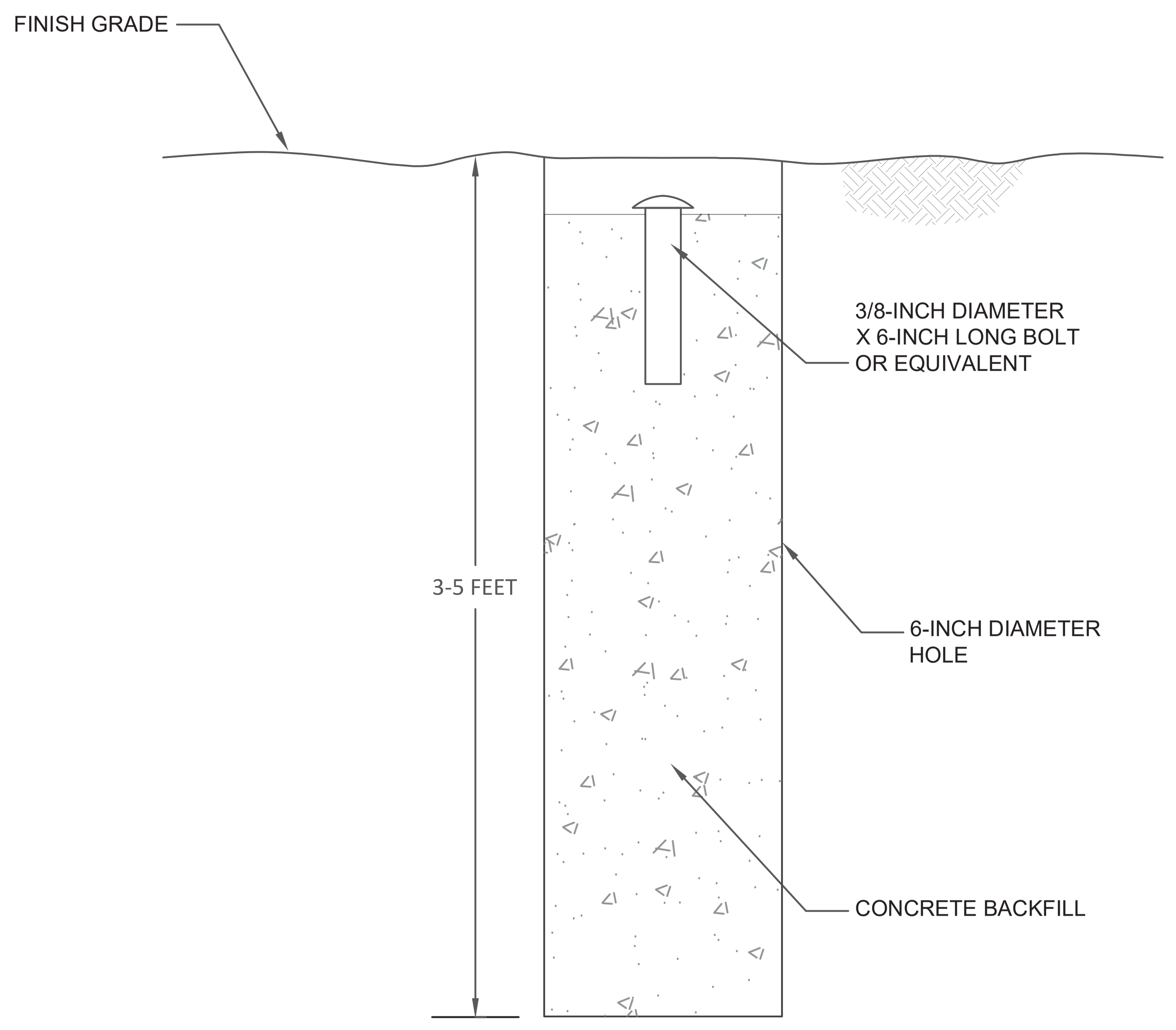

NOT TO SCALE

SPORTS ARENA COMPLEX
MIDWAY RISING

SETTLEMENT
MONUMENT DETAILS
SURFACE MONUMENT

GROUP DELTA

PROJECT NUMBER	FIGURE NUMBER
SD760	11A

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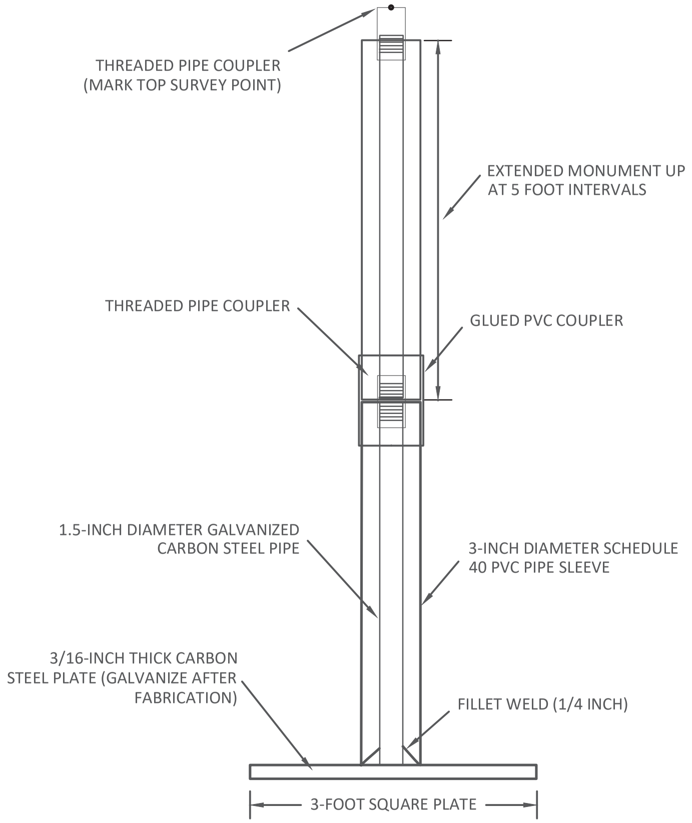

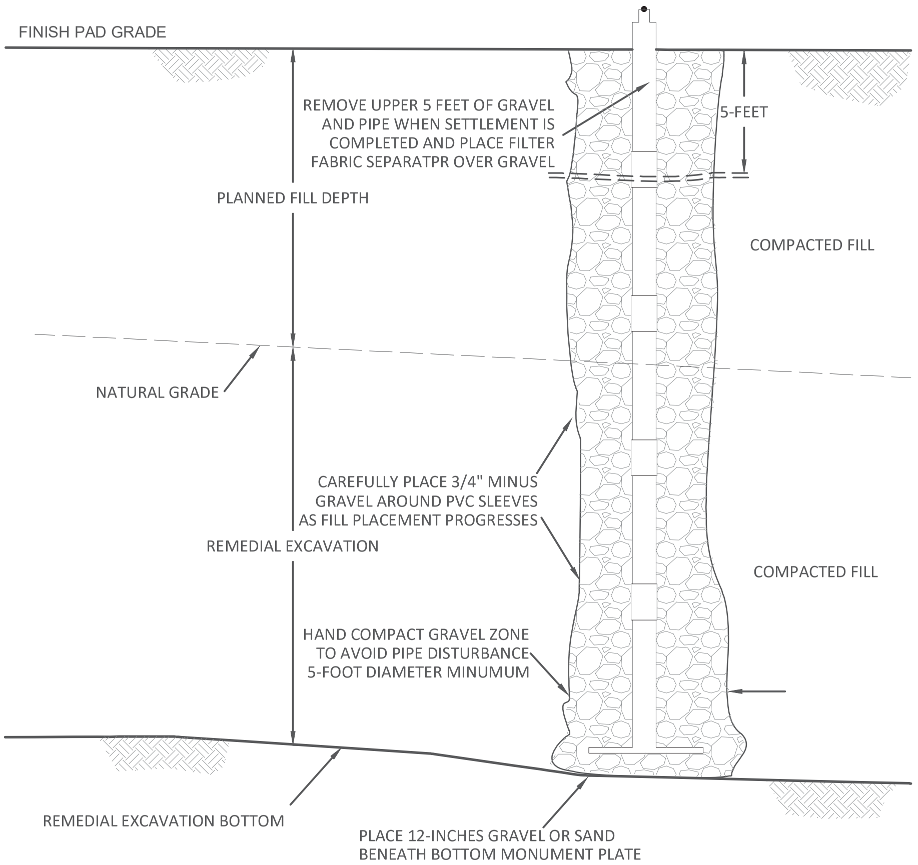

NOTES:

1) SETTLEMENT MONUMENTS SHOULD BE SURVEYED DAILY DURING FILL PLACEMENT, AND WEEKLY THEREAFTER BY A CALIFORNIA REGISTERED LAND SURVEYOR SURVEY DATA SHOULD BE PROVIDED WEEKLY TO GROUP DELTA CONSULTANTS FOR REVIEW UNTIL SETTLEMENT IS DEEMED SUBSTANTIALLY COMPLETED.

2) SETTLEMENT MONUMENTS SHOULD BE SURVEYED IMMEDIATELY BEFORE AND AFTER ADDING EACH 5-FOOT PIPE SECTION TO ACCOMMODATE CHANGE IN REFERENCE ELEVATION.

3) TO DESTROY MONUMENTS, UNCOUPLE AND REMOVE UPPER 5-FOOT PIPE SECTIONS, PLACE MIRAFI 140N FILTER FABRIC OVER GRAVEL, AND BACKFILL THE EXCAVATION WITH COMPACTED SOIL.

NOT TO SCALE

SPORTS ARENA COMPLEX
MIDWAY RISING

SETTLEMENT
MONUMENT DETAILS
RISER PLATE

GROUP DELTA

PROJECT NUMBER	FIGURE NUMBER
SD760	11B

APPENDIX A
EXPLORATION RECORDS

APPENDIX A

EXPLORATION RECORDS

Field exploration included a visual reconnaissance of the site, the drilling of eight (8) hollow stem and mud rotary exploratory borings, and the advancement of eight (8) cone penetration tests (CPTs). Borings A-23-011 through R-23-002 were drilled between February 6 and February 10, 2023. SCPT-23-021 through SCPT-23-028 were advanced on February 6 and February 7, 2023, and March 15, 2023. The maximum depth of exploration was about 120.5 feet below surrounding grades. A summary of the explorations is included in Table A-1. A summary of the groundwater measurements performed at the exploration locations is included in Table A-2. The approximate exploration locations are shown in Figure 2. Logs of the explorations and plots of the CPT data and interpretations are provided in Figures A-1 through A-16, immediately after the Boring Record Legends.

HOLLOW STEM AND MUD ROTARY BORINGS

The hollow stem and mud rotary exploratory borings were advanced by Pacific Drilling using MARL M10 and MARL MTXD truck mounted drill rigs. 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. 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 Table A-1. Groundwater measurements from the borings, where performed, are included in Table A-2.

The drive samples were collected from the exploratory borings using automatic hammers with average Energy Transfer Ratios (ETR) of approximately 97 percent. For each sample, the 6-inch incremental blowcounts was 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. Where sampler refusal was encountered (i.e., unable to drive the sampler more than the first six inches with 50 hammer blows), the blowcount is denoted as "REF".

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

APPENDIX A

EXPLORATION RECORDS (Continued)

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 (q_c);
- Sleeve Friction (f_s); and
- Dynamic Pore Pressure (u).

Soil Behavior Type Interpretations: The Soil Behavior Type (SBT) shown on the CPT plots is a stratigraphic interpretation based on relationships between qc, fs, and u (Robertson, 2009) that represents major soil lithologic changes. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures. However, the presence of mica, organics, and/or seashells coupled with the very low apparent density observed in the borings appears to have influenced the interpretation of SBT within the fill and upper paralic estuarine deposits. Therefore, for analysis purposes, the SBT correlated from the CPT data was adjusted to best fit the observations, classifications, and material properties of the soils observed within the borings using guidance provided by Kehoe for interpreting SBT based on their experience from prior projects with similar subsurface conditions.

Shear Wave Velocity Testing: At locations SCPT-23-021, SCPT-23-022, SCPT-23-024, SCPT-23-025 and SCPT-23-028, shear wave velocity measurements were obtained at various depths to a depth of approximately 100 feet. The shear wave was generated using an air-actuated hammer located inside the front jack of the CPT rig. 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 are presented in Figure A-17 through A-21.

Pore Pressure Dissipation Testing: Pore Pressure Dissipation (PPD) tests were performed at select CPT soundings to approximate the depth to groundwater. PPD tests consist of advancing the cone to a target depth below the suspected groundwater level and recording the dynamic pore pressure over a period of time until it stabilizes to a constant pressure. The stabilized pressure can be used to back-calculate the hydrostatic pressure, and consequently the depth to groundwater. Groundwater depths interpreted from PPD tests performed at the CPTs, where performed, are included in Table A-2.

APPENDIX A

EXPLORATION RECORDS (Continued)

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

Exploration ID	Latitude [°]	Longitude [°]	Top Elevation NGVD 29 [FT]	Exploration Depth [FT]	Bottom Elevation NGVD 29 [FT]	Figure No.
A-23-011	32.756717	-117.214067	9	31.5	-23	A-1
A-23-012	32.754567	-117.214333	10	41.5	-32	A-2
A-23-013	32.757233	-117.212200	9	21.5	-13	A-3
A-23-014	32.756100	-117.212383	14	21.5	-8	A-4
A-23-015	32.754817	-117.211433	14	21.5	-8	A-5
A-23-016	32.754083	-117.210400	11	41.5	-31	A-6
R-23-001	32.756317	-117.213200	11	119.0	-108	A-7
R-23-002	32.754367	-117.208317	12	120.5	-109	A-8
SCPT-23-021[1]	32.757850	-117.213583	9	108.1	-99	A-9
SCPT-23-022[1]	32.755800	-117.213850	13	105.5	-93	A-10
CPT-23-023	32.756817	-117.211517	9	108.3	-99	A-11
SCPT-23-024[1]	32.754800	-117.212933	13	106.2	-93	A-12
SCPT-23-025[1]	32.756000	-117.211167	12	103.5	-92	A-13
CPT-23-026	32.754667	-117.210700	13	104.3	-91	A-14
CPT-23-027	32.754850	-117.208917	11	111.6	-101	A-15
SCPT-23-028[1]	32.753517	-117.207133	10	103.0	-93	A-16

[1] Shear wave velocity measurements shown on Figure A-17 through A-21.

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 by interpolation using the referenced plans provided by Project Design Consultants, *which utilizes the Northern Geodetic Vertical Datum of 1929 (NVGD 29) as the vertical datum* (see Figure 2). 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.

APPENDIX A

EXPLORATION RECORDS (Continued)

Table A-2 – Groundwater Measurements Summary (see Figure 2)

Exploration ID	Groundwater Depth [FT]	Groundwater Elevation NGVD 29 [FT]	Date of Measurement	Type of Measurement
A-23-011	7.0	2.0	2/06/2023	Encountered During Drilling
A-23-012	12.5	-2.5	2/07/2023	Well Sounder in Boring
A-23-013	7.3	1.7	2/06/2023 (3:00 PM)	Well Sounder in Temporary Well Casing
A-23-014	14.5	-0.5	2/06/2023	Well Sounder in Boring
A-23-015	14.0	0.0	2/07/2023	Well Sounder in Boring
A-23-016	15.0	-4.0	2/07/2023	Well Sounder in Boring
SCPT-23-022	12.6	0.4	2/06/2023	Pore Pressure Dissipation Test
CPT-23-023	6.1	2.9	2/06/2023	Pore Pressure Dissipation Test
SCPT-23-025	12.3	-0.3	2/07/2023	Pore Pressure Dissipation Test
CPT-23-026	15.8	-2.8	2/07/2023	Pore Pressure Dissipation Test
CPT-23-027	12.4	-1.4	3/15/2023	Pore Pressure Dissipation Test
SCPT-23-028	9.4	0.6	3/15/2023	Pore Pressure Dissipation Test

SOIL IDENTIFICATION AND DESCRIPTION SEQUENCE

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

Describe the soil using descriptive terms in the order shown

Minimum Required Sequence:

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

⬬ = optional for non-Caltrans projects

Where applicable:

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

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. SD760

MIDWAY RISING SPORTS ARENA COMPLEX
3220, 3240, 3250, and 3500
SPORTS ARENA BOULEVARD
SAN DIEGO, 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. SD760

MIDWAY RISING SPORTS ARENA COMPLEX
3220, 3240, 3250, and 3500
SPORTS ARENA BOULEVARD
SAN DIEGO, 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. SD760

MIDWAY RISING SPORTS ARENA COMPLEX
3220, 3240, 3250, and 3500
SPORTS ARENA BOULEVARD
SAN DIEGO, CALIFORNIA

BORING RECORD LEGEND #3

LEGEND OF ROCK MATERIALS

⊠	IGNEOUS ROCK
▤	SEDIMENTARY ROCK
▧	METAMORPHIC ROCK

BEDDING SPACING

Description	Thickness/Spacing
Massive	Greater than 10 ft
Very Thickly Bedded	3 ft - 10 ft
Thickly Bedded	1 ft - 3 ft
Moderately Bedded	4 in - 1 ft
Thinly Bedded	1 in - 4 in
Very Thinly Bedded	1/4 in - 1 in
Laminated	Less than 1/4 in

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Leaching		
Description	Body of Rock	Fracture Surfaces		Texture	Leaching	General Characteristics
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No leaching	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, grain boundary conditions	All fracture surfaces are discolored or oxidized; surfaces friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Texture altered by chemical disintegration (hydration, argillation)	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

PERCENT CORE RECOVERY (REC)

$$\frac{\Sigma\ \text{Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

ROCK QUALITY DESIGNATION (RQD)

$$\frac{\Sigma\ \text{Length of intact core pieces} \geq 4\ \text{in.}}{\text{Total length of core run (in.)}} \times 100$$

RQD* indicates soundness criteria not met.

ROCK HARDNESS

Description	Criteria
Extremely Hard	Cannot be scratched with a pocketknife or sharp pick. Can only be chipped with repeated heavy hammer blows
Very Hard	Cannot be scratched with a pocketknife or sharp pick. Breaks with repeated heavy hammer blows.
Hard	Can be scratched with a pocketknife or sharp pick with difficulty (heavy pressure). Breaks with heavy hammer blows.
Moderately Hard	Can be scratched with a pocketknife or sharp pick with light or moderate pressure. Breaks with moderate hammer blows
Moderately Soft	Can be grooved 1/16 in. deep with a pocketknife or sharp pick with moderate or heavy pressure. Breaks with light hammer blow or heavy manual pressure.
Soft	Can be grooved or gouged easily with a pocketknife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	Can be readily indented, grooved or gouged with fingernail, or carved with a pocketknife. Breaks with light manual pressure.

FRACTURE DENSITY

Description	Observed Fracture Density
Unfractured	No fractures
Very Slightly Fractured	Core lengths greater than 3 ft.
Slightly Fractured	Core lengths mostly from 1 to 3 ft.
Moderately Fractured	Core lengths mostly 4 in. to 1 ft.
Intensely Fractured	Core lengths mostly from 1 to 4 in.
Very Intensely Fractured	Mostly chips and fragments.

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

GROUP DELTA

PROJECT NO. SD760

MIDWAY RISING SPORTS ARENA COMPLEX
3220, 3240, 3250, and 3500
SPORTS ARENA BOULEVARD
SAN DIEGO, CALIFORNIA

BORING RECORD LEGEND #4

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **A-23-011**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/6/2023
FINISH 2/6/2023
SHEET NO. 1 of 2
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6
TOTAL DEPTH (ft) 31.5
GROUND ELEV (ft) 9
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ 7.0 / *2.0*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												PAVEMENT: Approximately 3 1/2 inches of asphalt concrete.
												FILL: SILTY SAND (SM); very dark grayish brown (10YR 3/2); moist; mostly fine SAND; some fines; nonplastic; micaceous. (53% Sand; 47% Fines)
			B1				11.9		PA CR EI			
	5											
5												
			R2	2 2 2	4	4						With GRAVEL; very loose; dark yellowish brown (10YR 4/4); mostly fine to medium SAND; little fine gravel; low plasticity.
												PARALIC ESTUARINE DEPOSITS: SILT with SAND (ML); very loose; very dark grayish brown (10YR 3/2); wet; mostly fines; few fine SAND; nonplastic.
			S3	P P P	0	0						
	0											
10												Micaceous. (73% Fines)
			R4	P P P	0	0	42.7	79	WA PI			
												SILTY SAND (SM); medium dense; very dark gray (10YR 3/1); wet; mostly fine SAND; little fines; nonplastic micaceous.
	-5											
15												
			S5	4 7 8	15	24						
	-10											
20												Very dense; some fines.
			R6	11 24 27	51	55	23.9	106				
	-15											

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

FIGURE

A-1 a

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex		**PROJECT NUMBER** SD760		**BORING** **A-23-011**	
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California		**START** 2/6/2023	**FINISH** 2/6/2023	**SHEET NO.** 2 of 2	
DRILLING COMPANY Pacific Drilling	**DRILLING METHOD** Hollow Stem Auger		**LOGGED BY** D. Guzman	**CHECKED BY** C. Vonk	
DRILLING EQUIPMENT MARL M10	**BORING DIA. (in)** 6	**TOTAL DEPTH (ft)** 31.5	**GROUND ELEV (ft)** 9	**DEPTH/*ELEV.* GROUNDWATER (ft)** ▼ 7.0 / *2.0*	
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)	**NOTES** ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}				

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			S7	P P P	0	0						**PARALIC ESTUARINE DEPOSITS (CONTINUED):** SILTY SAND (SM); very loose; very dark gray (10YR 3/1); wet; mostly fine SAND; little fines; nonplastic micaceous.
	-20											SANDY SILT (ML); stiff; very dark gray (10YR 3/1); wet; mostly fines; some fine SAND; low plasticity; seashells present. PP = 1.5 tsf.
30			R8	5 6 8	14	15	39.9	82				@31.5': No recovery with Shelby tube.
			SH9									Total Depth = 31.5 feet (Target depth reached). Groundwater encountered during drilling at a depth of 7 feet. Boring backfilled on 2/6/2023 shortly after drilling with bentonite and portland cement and capped with black-dyed rapid set concrete. This Boring Record is part of a geotechnical report which must be considered in its entirety. The exploration elevations were estimated by interpolation using the referenced plans provided by Project Design Consultants, which utilizes the Northern Geodetic Vertical Datum of 1929 (NGVD 29) as the vertical datum (see Figure 2).
	-25											
35												
	-30											
40												
	-35											
45												
	-40											

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FIGURE

A-1 b

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex	**PROJECT NUMBER** SD760	**BORING** **A-23-012**	
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California	**START** 2/7/2023	**FINISH** 2/7/2023	**SHEET NO.** 1 of 2
DRILLING COMPANY Pacific Drilling	**DRILLING METHOD** Hollow Stem Auger	**LOGGED BY** D. Guzman	**CHECKED BY** C. Vonk
DRILLING EQUIPMENT MARL M10	**BORING DIA. (in)** 6	**TOTAL DEPTH (ft)** 41.5	**GROUND ELEV (ft)** 10
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ 12.5 / *-2.5*			
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)	**NOTES** ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}		

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												PAVEMENT: Approximately 2 inches of asphalt concrete.
			B1				14.0					**FILL:** SILTY SAND (SM); light olive brown (2.5Y 5/6); moist; mostly fine to medium SAND; little fines; nonplastic. @ 3': Dark brown (10YR 3/3).
5	5		R2	3 4 2	6	6						Poorly graded SAND (SP); loose; dark brown (10YR 3/3); moist; trace fines; mostly fine SAND; nonplastic; slightly micaceous.
			S3	2 1 1	2	2						SILTY SAND with GRAVEL (SM); very loose; dark reddish brown (5YR 3/4); moist; mostly fine to medium SAND; little fines; little fine gravel; nonplastic.
10	0		R4	P P 4	4	4						**PARALIC ESTUARINE DEPOSITS:** SILTY SAND (SM); very loose; yellowish red (5YR 5/6); moist; mostly fine to medium SAND; little fines; nonplastic. No recovery.
											▼	Wet.
15	-5											@ 15': Dark brown (10YR 3/3).
			S5	P P P	0	0						SILT (ML); very soft; very dark grayish brown (2.5Y 3/2); wet; mostly fines; few fine SAND; low plasticity; slightly micaceous.
												SILTY SAND (SM); loose; dark gray (2.5Y 4/1); wet; mostly fine SAND; some fines; low plasticity; micaceous. (38% Fines)
20	-10		R6	3 4 4	8	9	30.7	91	WA PI			

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FIGURE

A-2 a

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **A-23-012**

SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/7/2023
FINISH 2/7/2023
SHEET NO. 2 of 2

DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger
LOGGED BY D. Guzman
CHECKED BY C. Vonk

DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6
TOTAL DEPTH (ft) 41.5
GROUND ELEV (ft) 10
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ 12.5 / *-2.5*

SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		▮	SH7									**PARALIC ESTUARINE DEPOSITS (CONTINUED):** SILTY SAND (SM); loose; very dark gray (2.5Y 3/1); wet; mostly fine SAND; little fines; nonplastic; micaceous.
												Poorly graded SAND with SILT (SP-SM); loose; very dark gray (2.5Y 3/1); wet; mostly fine to medium SAND; few fines; trace fine GRAVEL; low plasticity. (1% Gravel; 87% Sand; 12% Fines)
30	-20	⧗	R8	6 3 5	8	9	16.8	108	PA PI			
												SILTY SAND (SM); dense; very dark gray (5Y 3/1); wet; mostly fine SAND; little fines; trace fine GRAVEL; nonplastic.
35	-25	☒	S9	7 10 9	19	31						
40	-30	⧗	R10	4 4 6	10	16	20.3	109				Medium dense; gray (5Y 5/1); mostly fine to medium SAND; seashells present.
45	-35											

Total Depth = 41.5 feet (Target depth reached).

Groundwater measured during drilling at a depth of 12.5 feet.

Boring backfilled on 2/7/2023 shortly after drilling with bentonite and portland cement and capped with black-dyed rapid set concrete.

This Boring Record is part of a geotechnical report which must be considered in its entirety.

The exploration elevations were estimated by interpolation using the referenced plans provided by Project Design Consultants, which utilizes the Northern Geodetic Vertical Datum of 1929 (NGVD 29) as the vertical datum (see Figure 2).

BORING RECORD

PROJECT NAME: Midway Rising Sports Arena Complex
PROJECT NUMBER: SD760
BORING: **A-23-013**
SITE LOCATION: 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START: 2/6/2023
FINISH: 2/6/2023
SHEET NO.: 1 of 2
DRILLING COMPANY: Pacific Drilling
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: D. Guzman
CHECKED BY: C. Vonk
DRILLING EQUIPMENT: MARL M10
BORING DIA. (in): 6
TOTAL DEPTH (ft): 21.5
GROUND ELEV (ft): 9
DEPTH/*ELEV.* GROUNDWATER (ft): ▼ 7.3 / *1.7*
SAMPLING METHOD: Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES: ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DESCRIPTION AND CLASSIFICATION
0	9									**PAVEMENT:** Approximately 3 1/2 inches of asphalt concrete.
										FILL: SILTY SAND (SM); dark olive gray (5Y 3/2); moist; mostly fine SAND; little fines; nonplastic; micaceous.
		▨	B1				12.6			
	5									
5		⊠	S2	2 2 1	3	5			WA	Loose. (13% Fines)
										▼ **PARALIC ESTUARINE DEPOSITS:** CLAYEY SAND (SC); very loose; dark yellowish brown (10YR 4/4); wet; mostly fine SAND; some fines; medium plasticity; micaceous.
		⧗	R3	3 1 1	2	2				
	0									
10		⊠	S4	P P P	0	0				SILT (ML); very soft; dark grayish brown (10YR 4/2); wet; mostly fines; few fine SAND; nonplastic.
										SILTY SAND (SM); medium dense; very dark gray (2.5Y 2/1); wet; mostly fine SAND; little fines; nonplastic; micaceous. (82% Sand; 18% Fines)
	-5									
15		⧗	R5	5 11 13	24	26	28.4	96	PA	
	-10									
20		⊠	S6	5 8 9	17	27				Laminated; strong organic odor.
										Total Depth = 21.5 feet (Target depth reached). Groundwater measured at a depth of 7.3 feet approximately 5 hours after drilling completed. Boring backfilled on 2/6/2023 shortly after drilling with
	-15									

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
Midway Rising Sports Arena Complex	SD760	**A-23-013**

SITE LOCATION	START	FINISH	SHEET NO.
3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California	2/6/2023	2/6/2023	2 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Hollow Stem Auger	D. Guzman	C. Vonk

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUNDWATER (ft)
MARL M10	6	21.5	9	▼ 7.3 / *1.7*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												bentonite and portland cement and capped with black-dyed rapid set concrete. This Boring Record is part of a geotechnical report which must be considered in its entirety. The exploration elevations were estimated by interpolation using the referenced plans provided by Project Design Consultants, which utilizes the Northern Geodetic Vertical Datum of 1929 (NGVD 29) as the vertical datum (see Figure 2).
	-20											
30												
	-25											
35												
	-30											
40												
	-35											
45												
	-40											

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FIGURE

A-3 b

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **A-23-014**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/6/2023
FINISH 2/6/2023
SHEET NO. 1 of 2
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6
TOTAL DEPTH (ft) 21.5
GROUND ELEV (ft) 14
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ 14.5 / *-0.5*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												PAVEMENT: Approximately 4 inches of asphalt concrete.
	10	▨	B1				12.0		EI			**FILL:** CLAYEY SAND with GRAVEL (SC); dark brown (10YR 3/3); moist; mostly fine to medium SAND; little fines; little fine to coarse GRAVEL; some COBBLES up to 6-inch diameter; low plasticity; scattered construction debris (e.g. brick).
5		⊠	S2	5 10 5	15	24			PA			SILTY SAND (SM); medium dense; pale brown (5Y 2/2); moist; mostly fine to medium SAND; some fines; trace fine GRAVEL; nonplastic. (71% Sand; 29% Fines)
	5	⧓	R3	5 5 4	9	10	10.6	90				Poorly-graded SAND with SILT (SP-SM); medium dense; moist, mostly fine SAND; few fines; nonplastic.
10		⊠	S4	2 1 1	2	2						SILTY SAND (SM); very loose; brown (10YR 4/3); moist; mostly fine SAND; little fines; nonplastic; micaceous.
	0										▼	**PARALIC ESTUARINE DEPOSITS:** SILT (ML); soft; very dark gray (5YR 3/1); wet; mostly fines; few fine SAND; low plasticity. PP=0.25 tsf.
15		⧓	R5	P P P	0	0	49.0	73				
	-5											Poorly-graded SAND with CLAY (SP-SC); medium dense; very dark gray (5YR 3/1); wet; mostly fine SAND; few fines; medium plasticity; micaceous. (7% Fines)
20		⊠	S6	4 6 9	15	24	32.4		WA PI			
	-10											Total Depth = 21.5 feet (Target depth reached). Groundwater measured shortly after drilling at a depth of 14.5 feet. Boring backfilled on 2/6/2023 shortly after drilling with

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FIGURE
A-4 a

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex

PROJECT NUMBER SD760

BORING **A-23-014**

SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California

START 2/6/2023

FINISH 2/6/2023

SHEET NO. 2 of 2

DRILLING COMPANY Pacific Drilling

DRILLING METHOD Hollow Stem Auger

LOGGED BY D. Guzman

CHECKED BY C. Vonk

DRILLING EQUIPMENT MARL M10

BORING DIA. (in) 6

TOTAL DEPTH (ft) 21.5

GROUND ELEV (ft) 14

DEPTH/*ELEV.* GROUNDWATER (ft) ▼ 14.5 / *-0.5*

SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)

NOTES ETR ~ 97%, $N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}$

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												bentonite and portland cement and capped with black-dyed rapid set concrete. This Boring Record is part of a geotechnical report which must be considered in its entirety. The exploration elevations were estimated by interpolation using the referenced plans provided by Project Design Consultants, which utilizes the Northern Geodetic Vertical Datum of 1929 (NGVD 29) as the vertical datum (see Figure 2).
	-15											
30												
	-20											
35												
	-25											
40												
	-30											
45												
	-35											

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **A-23-015**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/7/2023
FINISH 2/7/2023
SHEET NO. 1 of 2
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6
TOTAL DEPTH (ft) 21.5
GROUND ELEV (ft) 14
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ 14.0 / *0.0*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												PAVEMENT: Approximately 6 inches of asphalt concrete.
	10		B1				13.0		PA CR EI			**FILL:** CLAYEY SAND with GRAVEL (SC); strong brown (7.5Y 4/6); moist; some fine to medium SAND; some fines; some fine to coarse GRAVEL; medium plasticity; COBBLES up to 4-inch diameter; scattered construction debris (e.g. brick). (27% Gravel; 36% Sand; 37% Fines)
5			R2	4 9 11	20	22						SILTY SAND (SM); medium dense; brown (10YR 5/3); moist; mostly fine SAND; few fines; trace fine GRAVEL; nonplastic; slightly micaceous.
	5		S3	2 4 6	10	16						Poorly graded SAND (SP); medium dense; dark gray (10YR 4/1); mostly fine SAND; trace fines; nonplastic.
10			R4	3 4 5	9	10	15.7	100				SILTY SAND (SM); loose; dark gray (10YR 4/1); moist; mostly fine to medium SAND; little fines; low plasticity.
15	0		S5	P P 2	2	3			PA		▼	**PARALIC ESTUARINE DEPOSITS:** Poorly graded SAND with SILT (SP-SM); very loose; dark gray (5YR 4/1); wet; mostly fine to medium SAND; few fines; nonplastic. (90% Sand; 10% Fines)
20	-5		R6	4 9 15	24	26	27.0	96				SILTY SAND (SM); medium dense; very dark gray (7.5YR 3/1); wet; mostly fine to medium SAND; little fines; nonplastic; micaceous.
	-10											Total Depth = 21.5 feet (Target depth reached). Groundwater encountered during drilling at a depth of 14.0 feet. Boring backfilled on 2/7/2023 shortly after drilling with

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 Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **A-23-015**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/7/2023
FINISH 2/7/2023
SHEET NO. 2 of 2
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6
TOTAL DEPTH (ft) 21.5
GROUND ELEV (ft) 14
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ 14.0 / *0.0*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												bentonite and portland cement and capped with black-dyed rapid set concrete. This Boring Record is part of a geotechnical report which must be considered in its entirety. The exploration elevations were estimated by interpolation using the referenced plans provided by Project Design Consultants, which utilizes the Northern Geodetic Vertical Datum of 1929 (NGVD 29) as the vertical datum (see Figure 2).
	-15											
30												
	-20											
35												
	-25											
40												
	-30											
45												
	-35											

GDC_LOG_BORING_MMX_SOIL_SD SD760 LOGS.GPJ GDCLOG.GDT 4/17/23

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **A-23-016**

SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/7/2023
FINISH 2/7/2023
SHEET NO. 1 of 2

DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger
LOGGED BY D. Guzman
CHECKED BY C. Vonk

DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6
TOTAL DEPTH (ft) 41.5
GROUND ELEV (ft) 11
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ 15.0 / *-4.0*

SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS
	10	B1				5.9		PA
5	5	S2	4 3 3	6	10			
		R3	1 3 5	8	9			
10	0	S4	P P P	0	0			
15	-5	R5	3 8 10	18	29	32.6	92	
20	-10	S6	P P P	0	0			

DESCRIPTION AND CLASSIFICATION

PAVEMENT: Approximately 4 inches of asphalt concrete.

FILL: SILTY SAND (SM); grayish brown (10YR 5/2); moist; mostly fine SAND; little fines; nonplastic. (84% Sand; 16% Fines)

Loose; slightly micaceous.

Light brownish gray (10YR 6/2); mostly fine to coarse SAND.

PARALIC ESTUARINE DEPOSITS: SILTY SAND (SM); very loose; dark reddish brown (5YR 3/3); moist; mostly fine SAND; little fines; nonplastic; slightly micaceous.

▼ Medium dense; dark gray (5YR 4/1); wet.

SANDY SILT (ML); very loose; black (2.5Y 2.5/1); wet; mostly fines; some fine SAND; nonplastic.

(See description on following page)

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FIGURE A-6 a

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **A-23-016**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/7/2023
FINISH 2/7/2023
SHEET NO. 2 of 2
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6
TOTAL DEPTH (ft) 41.5
GROUND ELEV (ft) 11
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ 15.0 / *-4.0*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	-15		SH7				33.3	90	WA PI C			**PARALIC ESTUARINE DEPOSITS (CONTINUED):** Poorly graded SAND (SP); dark gray (10YR 4/1); wet; trace fines; mostly fine SAND; nonplastic. SANDY SILT (ML); loose; very dark gray (10YR 3/1); wet; mostly fines; some fine SAND; low plasticity; micaceous. (60% Fines)
30	-20		S8	2 2 2	4	6						Very dark gray (5Y 3/1); very micaceous.
35	-25		R9	5 6 6	12	13	35.4	86	WA			Stiff; seashells present. (60% Fines)
												SILTY SAND (SM); medium dense; dark gray (10YR 4/1); wet; mostly fine SAND; little fines; nonplastic; very micaceous. (85% Sand; 15% Fines)
40	-30		S10	4 6 7	13	21			PA			
45	-35											

Total Depth = 41.5 feet (Target depth reached).

Groundwater measured during drilling at a depth of 15.0 feet.

Boring backfilled on 2/7/2023 shortly after drilling with bentonite and portland cement and capped with black-dyed rapid set concrete.

This Boring Record is part of a geotechnical report which must be considered in its entirety.

The exploration elevations were estimated by interpolation using the referenced plans provided by Project Design Consultants, which utilizes the Northern Geodetic Vertical Datum of 1929 (NGVD 29) as the vertical datum (see Figure 2).

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FIGURE

A-6 b

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
Midway Rising Sports Arena Complex	SD760	**R-23-001**

SITE LOCATION	START	FINISH	SHEET NO.
3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California	2/7/2023	2/9/2023	1 of 6

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Hollow Stem Auger / Mud Rotary Wash	D. Guzman	C. Vonk

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUNDWATER (ft)
MARL M10	6/4	119	11	▼ NM / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	10											**PAVEMENT:** Approximately 4 inches of asphalt concrete. **FILL:** SILTY SAND (SM); brownish yellow (10YR 6/8); moist; mostly fine SAND; little fines; trace Gravel; low to nonplastic.
			B1				12.2					
5												Medium dense.
	5		S2	5 4 6	10	16						
10												**PARALIC ESTUARINE DEPOSITS:** SILTY SAND (SM); very loose; very dark grayish brown (10YR 3/2); wet; mostly fine SAND; little fines; nonplastic. @ 11.25': Black (10YR 2/1); micaceous.
	0		R3	P P P	0	0						
15												Loose; reddish brown (5YR 5/4). (16% Fines)
	-5		S4	2 2 4	6	10	34.7		WA PI			
20												Medium dense; very dark gray (5YR 3/1). (83% Sand, 17% Fines)
	-10		R5	4 2 8	10	11			PA			

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BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **R-23-001**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/7/2023
FINISH 2/9/2023
SHEET NO. 2 of 6
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger / Mud Rotary Wash
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6/4
TOTAL DEPTH (ft) 119
GROUND ELEV (ft) 11
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	-15	⊠	S6	2 1 1	2	3						**PARALIC ESTUARINE DEPOSITS (CONTINUED):** SILTY SAND (SM); very loose; very dark gray (2.5Y 3/1); wet; mostly fine SAND; little fines; nonplastic; micaceous; slight organic odor.
												Poorly graded SAND with SILT (SP-SM); medium dense; gray (2.5Y 6/1); wet; mostly fine SAND; few fines; nonplastic; micaceous. (88% Sand; 12% Fines)
30	-20	⧗	R7	4 7 15	22	24	23.3	104	PA			
35	-25	⊠	S8	2 3 4	7	11						Gray (10YR 5/1); seashells present.
												Fat CLAY (CH); medium stiff; very dark gray (5Y 3/1); wet; mostly fines; few fine SAND; medium plasticity; seashells present. PP = 0.5 tsf. (91% Fines)
40	-30	⧗	R9	P P P	0	0	45.7	75	WA PI			
45	-35	▮	SH10				60.5 51.4	66 69	WA PI UC C			(91% Fines)
												(See description on following page)

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FIGURE

A-7 b

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **R-23-001**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/7/2023
FINISH 2/9/2023
SHEET NO. 3 of 6
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger / Mud Rotary Wash
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6/4
TOTAL DEPTH (ft) 119
GROUND ELEV (ft) 11
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	-40	⧗	R11	P P P	0	0	34.1	88				**PARALIC ESTUARINE DEPOSITS (CONTINUED):** SILTY SAND (SM); very loose; dark gray (5Y 4/1); wet; mostly fine SAND; little fines; low to nonplastic.
55	-45	⧖	S12	4 5 6	11	18			WA			SANDY SILT (ML); medium dense; dark gray (5Y 4/1); wet; mostly fines; little fine SAND; low to nonplastic. (88% Fines)
60	-50	⧗	R13	5 10 14	24	26	36.2	87	PA DS			Very dark gray (2.5Y 3/1). (27% Sand; 73% Fines)
65	-55	⧖	S14	6 4 8	12	19						
70	-60	⧖	S15	4 4 7	11	18						SILTY SAND (SM); medium dense; very dark grayish brown (2.5Y 4/2); wet; mostly fine SAND; little fines; nonplastic.
												(See description on following page)

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San Diego, California 92126

FIGURE
A-7 c

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **R-23-001**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/7/2023
FINISH 2/9/2023
SHEET NO. 4 of 6
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger / Mud Rotary Wash
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL M10
BORING DIA. (in) 6/4
TOTAL DEPTH (ft) 119
GROUND ELEV (ft) 11
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	-65		R16	7 22 29	51	55						**PARALIC ESTUARINE DEPOSITS (CONTINUED):** Poorly graded SAND (SP); very dense; gray (5Y 6/1); wet; mostly fine to medium SAND; trace fines; nonplastic.
80												Dense; very dark gray (5Y 3/1); slightly micaceous.
	-70		S17	7 12 11	23	37						
												Poorly graded SAND with SILT (SP-SM); medium dense; very dark gray (2.5Y 3/1); wet; mostly fine SAND; few fines; low to nonplastic; micaceous.
85												
	-75		S18	4 6 11	17	27						
												Very dense; gray (7.5Y 5/1); mostly fine to medium SAND. (93% Sand; 7% Fines)
90												
	-80		R19	11 14 35	49	53	28.5	96	PA DS			
95												
	-85		S20	12 16 17	33	53						

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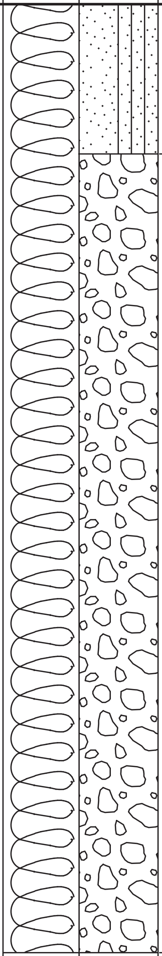
FIGURE

A-7 d

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex	PROJECT NUMBER SD760	BORING **R-23-001**	
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California	START 2/7/2023	FINISH 2/9/2023	SHEET NO. 5 of 6
DRILLING COMPANY Pacific Drilling	DRILLING METHOD Hollow Stem Auger / Mud Rotary Wash	LOGGED BY D. Guzman	CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL M10	BORING DIA. (in) 6/4	TOTAL DEPTH (ft) 119	GROUND ELEV (ft) 11
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*	SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)	NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}	



DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	-90	⊠	S21	11 14 18	32	52						**PARALIC ESTUARINE DEPOSITS (CONTINUED):** Poorly graded SAND with SILT(SP-SM); very dense; gray (7.5Y 5/1); wet; mostly fine to medium SAND; trace fines; nonplastic; slightly micaceous.
												OLD PARALIC DEPOSITS: Poorly graded GRAVEL with SAND (GP); wet; mostly fine to coarse gravel; little fine SAND; trace fines; nonplastic; slightly micaceous. Difficult drilling on fine to coarse GRAVELS and possible COBBLES from 103' to 112'.
105	-95	⧓	R22	37 50/4	100+							
110	-100											Very difficult drilling on fine to coarse GRAVELS and possible COBBLES from 112' to 119'.
115	-105											
120	-110											Total Depth = 119 feet (Refusal on GRAVELS and COBBLES). Groundwater not measured due to use of mud rotary drilling method. Gravel-rich layers encountered may include up to 30% COBBLES, estimated based on drill rig chatter and visual evaualation of drill cuttings. Gravel-rich layers encountered in this exploration were approximately 15 to 20 feet thick.

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FIGURE

A-7 e

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex	**PROJECT NUMBER** SD760	**BORING** **R-23-001**	
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California	**START** 2/7/2023	**FINISH** 2/9/2023	**SHEET NO.** 6 of 6
DRILLING COMPANY Pacific Drilling	**DRILLING METHOD** Hollow Stem Auger / Mud Rotary Wash	**LOGGED BY** D. Guzman	**CHECKED BY** C. Vonk
DRILLING EQUIPMENT MARL M10	**BORING DIA. (in)** 6/4	**TOTAL DEPTH (ft)** 119	**GROUND ELEV (ft)** 11
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*			
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)	**NOTES** ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}		

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												Boring backfilled on 2/9/2023 after drilling with bentonite and portland cement and capped with black-dyed rapid set concrete. This Boring Record is part of a geotechnical report which must be considered in its entirety. The exploration elevations were estimated by interpolation using the referenced plans provided by Project Design Consultants, which utilizes the Northern Geodetic Vertical Datum of 1929 (NGVD 29) as the vertical datum (see Figure 2).
	-115											
130												
	-120											
135												
	-125											
140												
	-130											
145												
	-135											

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FIGURE

A-7 f

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **R-23-002**

SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/9/2023
FINISH 2/10/2023
SHEET NO. 1 of 6

DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger / Mud Rotary Wash
LOGGED BY D. Guzman
CHECKED BY C. Vonk

DRILLING EQUIPMENT MARL MTXD
BORING DIA. (in) 6/4
TOTAL DEPTH (ft) 120.5
GROUND ELEV (ft) 12
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*

SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												PAVEMENT: Approximately 3 inches of asphalt concrete.
	10											**FILL:** SILTY SAND (SM); dark yellowish brown (10YR 4/2); moist; mostly fine SAND; little fines; nonplastic.
		▨	B1				4.2					
5		☒	S2	7 9 5	14	23						Medium dense.
	5											
10		⧗	R3	5 4 3	7	8						Loose; no recovery.
	0											
												PARALIC ESTUARINE DEPOSITS: SILTY SAND (SM); dark yellowish brown (10YR 4/2); moist; mostly fine SAND; some fines; nonplastic.
15		☒	S4	2 1 P	1	2			WA			Very loose; dusky yellowish brown (10YR 2/2); wet; micaceous.
	-5											CLAYEY SAND (SC); very loose; grayish black (N2); wet; mostly fine SAND; some fines; low plasticity; slightly micaceous. (47% Fines)
												SILTY SAND (SM); medium dense; grayish black (N2); wet; mostly fine SAND; little fines; nonplastic; micaceous.
20		⧗	R5	3 4 10	14	15	31.6	92	CR			
	-10											
												(See description on following page)

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San Diego, California 92126

FIGURE

A-8 a

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **R-23-002**

SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/9/2023
FINISH 2/10/2023
SHEET NO. 2 of 6

DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger / Mud Rotary Wash
LOGGED BY D. Guzman
CHECKED BY C. Vonk

DRILLING EQUIPMENT MARL MTXD
BORING DIA. (in) 6/4
TOTAL DEPTH (ft) 120.5
GROUND ELEV (ft) 12
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*

SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		☒	S6	5 6 8	14	23	31.2		PA			**PARALIC ESTUARINE DEPOSITS (CONTINUED):** Poorly graded SAND with SILT (SP-SM); medium dense; dark gray (N3); wet; mostly fine to medium SAND; few fines; trace fine GRAVEL; nonplastic; micaceous. (91% Sand; 9% Fines)
	-15											
												SILTY SAND (SM); loose; grayish black (N2); wet; mostly fine SAND; little fines; low to nonplastic; micaceous; seashells present.
30		■	R7	3 3 4	7	8	39.6	82				
	-20											
35		☒	S8	1 2 2	4	6	35.6		WA PI			SANDY SILT (ML); loose; grayish black (N2); wet; mostly fines; some SAND; low to nonplastic; micaceous; light odor. (59% Fines)
	-25											
40		■	R9	3 5 11	16	17						SILTY SAND (SM); medium dense; grayish black (N2); wet; mostly fine SAND; little fines; nonplastic; micaceous.
	-30											
												SILT with SAND (ML); very soft; grayish black (N2); wet; mostly fines; some fine SAND; low to medium plasticity. (80% Fines)
45		☒	S10	P P P	0	0	46.5		WA PI			
	-35											
												(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.

FIGURE

A-8 b

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
Midway Rising Sports Arena Complex	SD760	**R-23-002**

SITE LOCATION	START	FINISH	SHEET NO.
3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California	2/9/2023	2/10/2023	3 of 6

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Hollow Stem Auger / Mud Rotary Wash	D. Guzman	C. Vonk

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUNDWATER (ft)
MARL MTXD	6/4	120.5	12	▼ NM / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	-40		SH11				38.7	77	WA PI UC C			**PARALIC ESTUARINE DEPOSITS (CONTINUED):** SILTY SAND (SM); grayish black (N2); wet; mostly fine SAND; some fines; low to medium plasticity; micaceous. (44% Fines)
55	-45		S12	2 5 10	15	24	36.0	89	PA			SILTY SAND (SM); medium dense; dark gray (N3); wet; mostly fine SAND; some fines; trace fine GRAVEL; nonplastic; light odor; micaceous. (1% Gravel; 55% Sand; 44% Fines)
60	-50		S13	7 12 14	26	42						Dense; mostly fine to medium sand.
65	-55		R14	7 10 14	24	26	38.5	88	PA DS			SANDY SILT (ML); medium dense; dark gray (N3); wet; mostly fines; some fine SAND; nonplastic; very micaceous. (30% Sand; 70% Fines)
70	-60		S15	4 5 7	12	19						SILTY SAND (SM); medium dense; dark gray (N3); wet; mostly fine SAND; little fines; nonplastic; micaceous. Low plasticity.

GDC_LOG_BORING_MMX_SOIL_SD SD760 LOGS.GPJ GDCLOG.GDT 4/17/23

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BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **R-23-002**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/9/2023
FINISH 2/10/2023
SHEET NO. 4 of 6
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger / Mud Rotary Wash
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL MTXD
BORING DIA. (in) 6/4
TOTAL DEPTH (ft) 120.5
GROUND ELEV (ft) 12
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		☒	S16	5 3 10	13	21						**PARALIC ESTUARINE DEPOSITS (CONTINUED):** SILTY SAND (SM); medium dense; dark gray (N3); wet; mostly fine SAND; little fines; nonplastic; micaceous.
	-65											
												@ 79': Hard drilling.
80												Very dense; medium gray (N5); mostly fine to medium SAND. (88% Sand; 12% Fines)
		⧗	R17	29 38 50	88	95	21.3	103	PA DS			
	-70											@ 80' - 82': Borehole caving.
85												
		☒	S18	10 15 17	32	52						
	-75											Very dense; brownish gray (5YR 4/1).
90												Dense; dark gray (N3).
		☒	S19	11 13 16	29	47						
	-80											
												Poorly graded SAND with SILT (SP-SM); very dense; medium gray (N5); wet; mostly fine to coarse SAND; few fines; nonplastic.
95												
		⧗	R20	17 24 26	50	54	23.4	100				
	-85											With GRAVEL; brownish black (5YR 2/1); mostly fine to medium SAND; some fine GRAVEL. @ 97-98': Difficult drilling on fine to coarse GRAVELS and possibe COBBLES.
		☒	S21	22 18 19	37	60						

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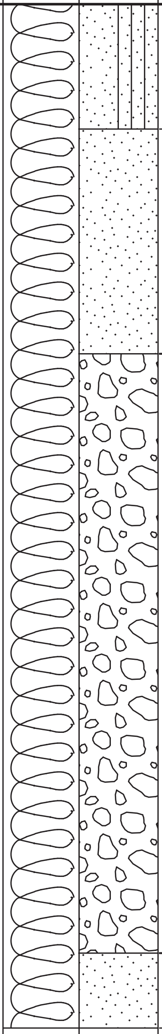
FIGURE

A-8 d

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex
PROJECT NUMBER SD760
BORING **R-23-002**
SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California
START 2/9/2023
FINISH 2/10/2023
SHEET NO. 5 of 6
DRILLING COMPANY Pacific Drilling
DRILLING METHOD Hollow Stem Auger / Mud Rotary Wash
LOGGED BY D. Guzman
CHECKED BY C. Vonk
DRILLING EQUIPMENT MARL MTXD
BORING DIA. (in) 6/4
TOTAL DEPTH (ft) 120.5
GROUND ELEV (ft) 12
DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*
SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DESCRIPTION AND CLASSIFICATION
	-90									**PARALIC ESTUARINE DEPOSITS (CONTINUED):** Poorly graded SAND with SILT and GRAVEL (SP-SM); very dense; brownish black (5YR 2/1); wet; mostly fine to medium SAND; some fine GRAVEL; few fines; nonplastic; micaceous.
										OLD PARALIC DEPOSITS: Poorly graded SAND with GRAVEL (SP); very dense; grayish black (N2); wet; mostly fine SAND; little fine to coarse GRAVEL; trace fines; nonplastic; coarse GRAVEL up to 3-inch diameter in sampler; slightly micaceous.
105		⧗	R22	27 50/5"	50/5"	100+				
	-95									Difficult drilling on fine to coarse GRAVELS and possible COBBLES from 103' to 112'.
110										
	-100									Very difficult drilling on fine to coarse GRAVELS and possible COBBLES from 112' to 119'.
115		☒	S23	44 39 45	84	100+				Poorly graded GRAVEL with SAND (GP); very dense; moderate yellowish brown (10YR 5/4) to dark yellowish brown (10YR 4/2) and traces of moderate brown (5YR 3/4); wet; mostly fine to coarse GRAVEL; some fine SAND; trace fines; nonplastic.
	-105									
120		⧗	R24	50/5"	50/5"	REF				Poorly graded SAND with GRAVEL (SP); very dense; moderate olive brown (5Y 4/4); mostly fine SAND; little fine GRAVEL; trace fines; nonplastic; fine GRAVEL up to 3/4-inch diameter.
	-110									Total Depth = 120.5 feet. Groundwater not measured due to use of mud rotary drilling method. Gravel-rich layers encountered may include up to 30% COBBLES, estimated based on drill rig chatter and



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San Diego, California 92126

FIGURE
A-8 e

BORING RECORD

PROJECT NAME Midway Rising Sports Arena Complex

PROJECT NUMBER SD760

BORING **R-23-002**

SITE LOCATION 3220, 3240, 3250, and 3500 Sports Arena Boulevard, San Diego, California

START 2/9/2023

FINISH 2/10/2023

SHEET NO. 6 of 6

DRILLING COMPANY Pacific Drilling

DRILLING METHOD Hollow Stem Auger / Mud Rotary Wash

LOGGED BY D. Guzman

CHECKED BY C. Vonk

DRILLING EQUIPMENT MARL MTXD

BORING DIA. (in) 6/4

TOTAL DEPTH (ft) 120.5

GROUND ELEV (ft) 12

DEPTH/*ELEV.* GROUNDWATER (ft) ▼ NM / *na*

SAMPLING METHOD Hammer: 140 lbs., Drop: 30 in. (Automatic)

NOTES ETR ~ 97%, N_{60} = 1.62*N_{SPT} = 1.08*N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
130 135 140 145	-115 -120 -125 -130 -135											visual evaulation of drill cuttings. Gravel-rich layers encountered in this exploration were approximately 6 to 10 feet thick. Boring backfilled on 2/10/2023 shortly after drilling completion with bentonite and portland cement and capped with black-dyed rapid set concrete. This Boring Record is part of a geotechnical report which must be considered in its entirety. The exploration elevations were estimated by interpolation using the referenced plans provided by Project Design Consultants, which utilizes the Northern Geodetic Vertical Datum of 1929 (NGVD 29) as the vertical datum (see Figure 2).

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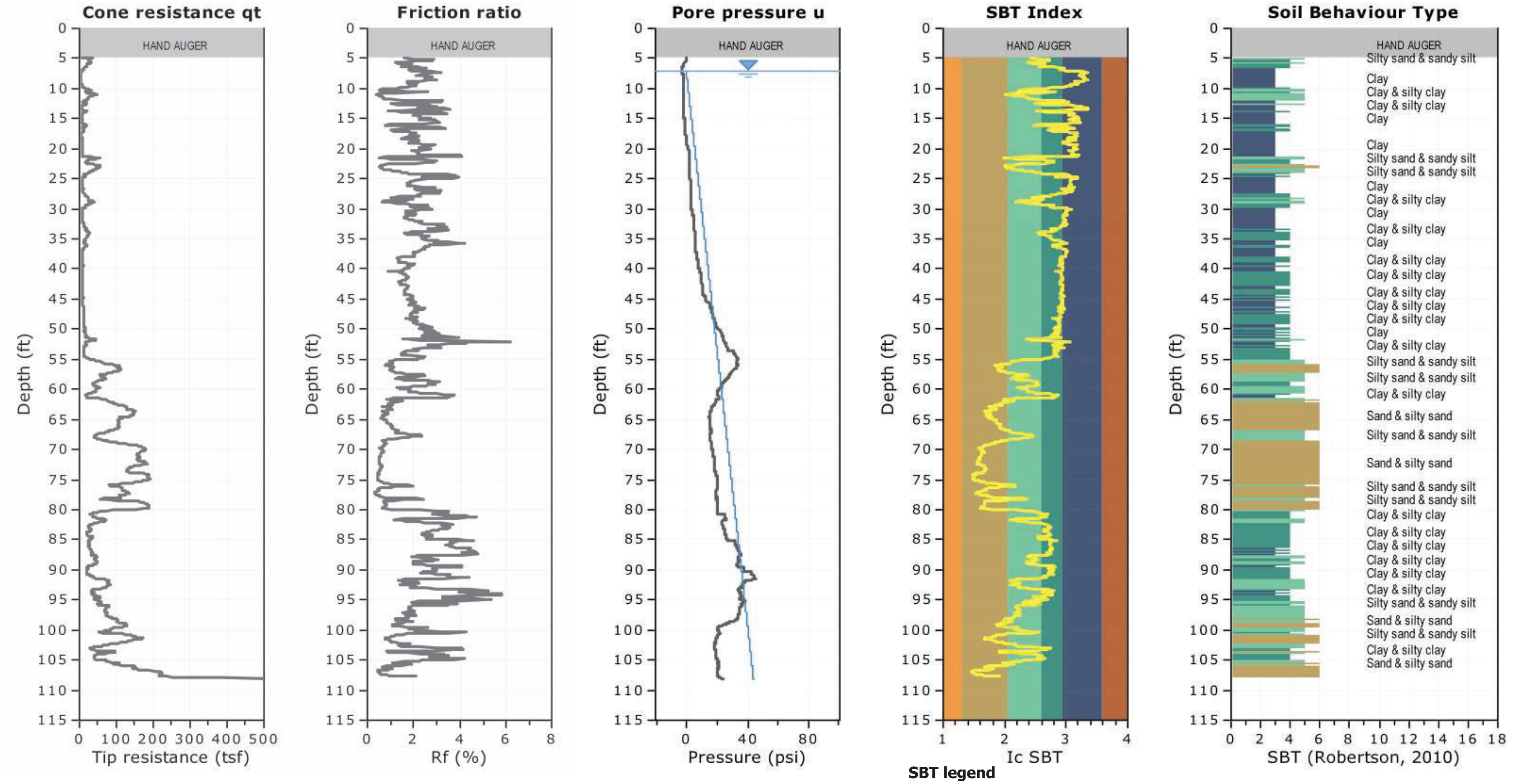
Project: **Midway Rising Sports Arena Complex**

Location: **3220, 3240, 3250, and 3500 Sports Arena Blvd, San Diego, California**

SCPT-23-021

Total depth: 108.14 ft, Date: 2/6/2023



SBT legend

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

FIGURE A-9

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Project: **Midway Rising Sports Arena Complex** **SCPT-23-022**

Location: **3220, 3240, 3250, and 3500 Sports Arena Blvd, San Diego, California** Total depth: 105.45 ft, Date: 2/6/2023

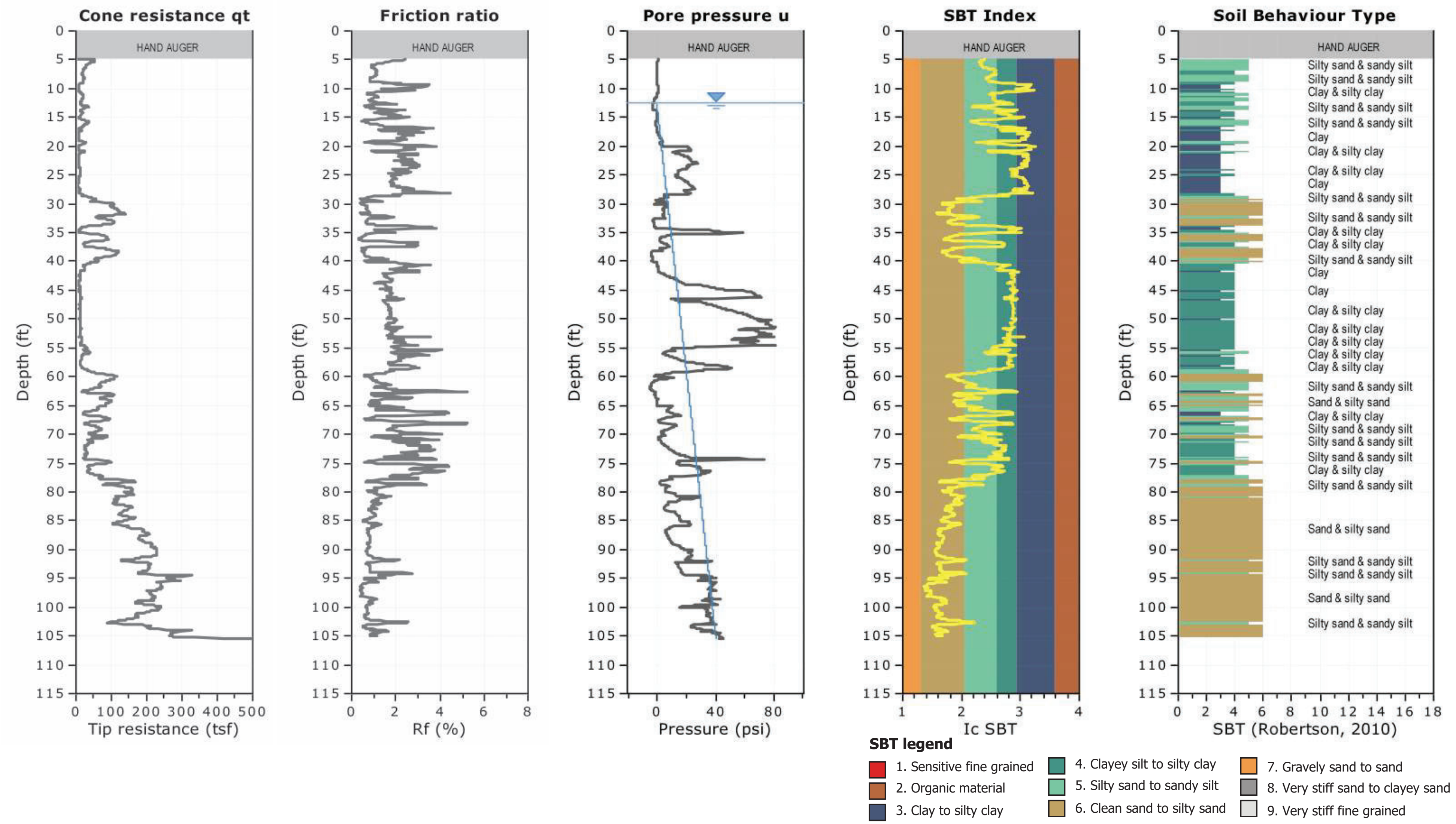

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FIGURE A-10

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Project: **Midway Rising Sports Arena Complex**
Location: **3220, 3240, 3250, and 3500 Sports Arena Blvd, San Diego, California**

CPT-23-023
Total depth: 108.33 ft, Date: 2/6/2023

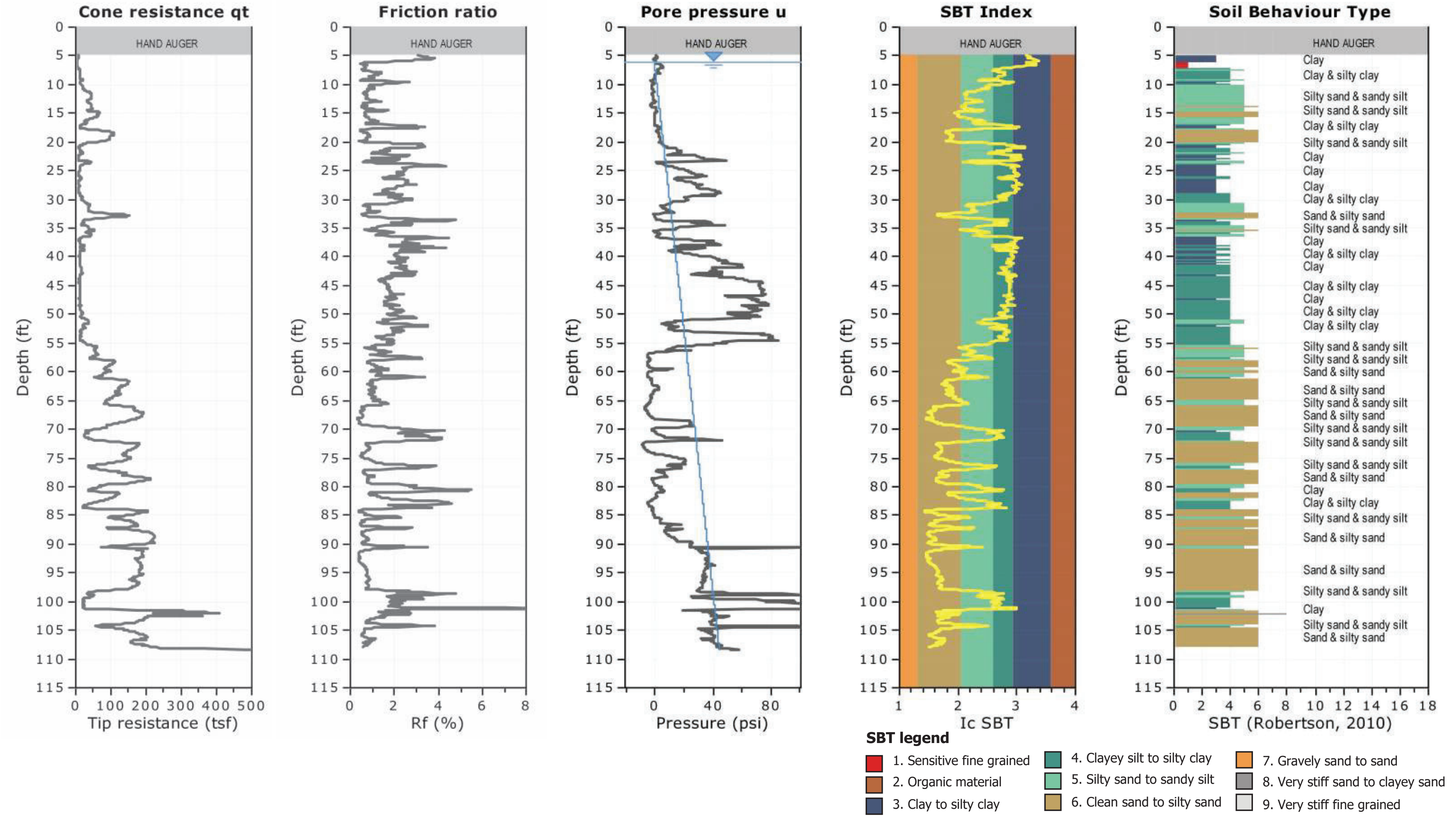

FIGURE A-11

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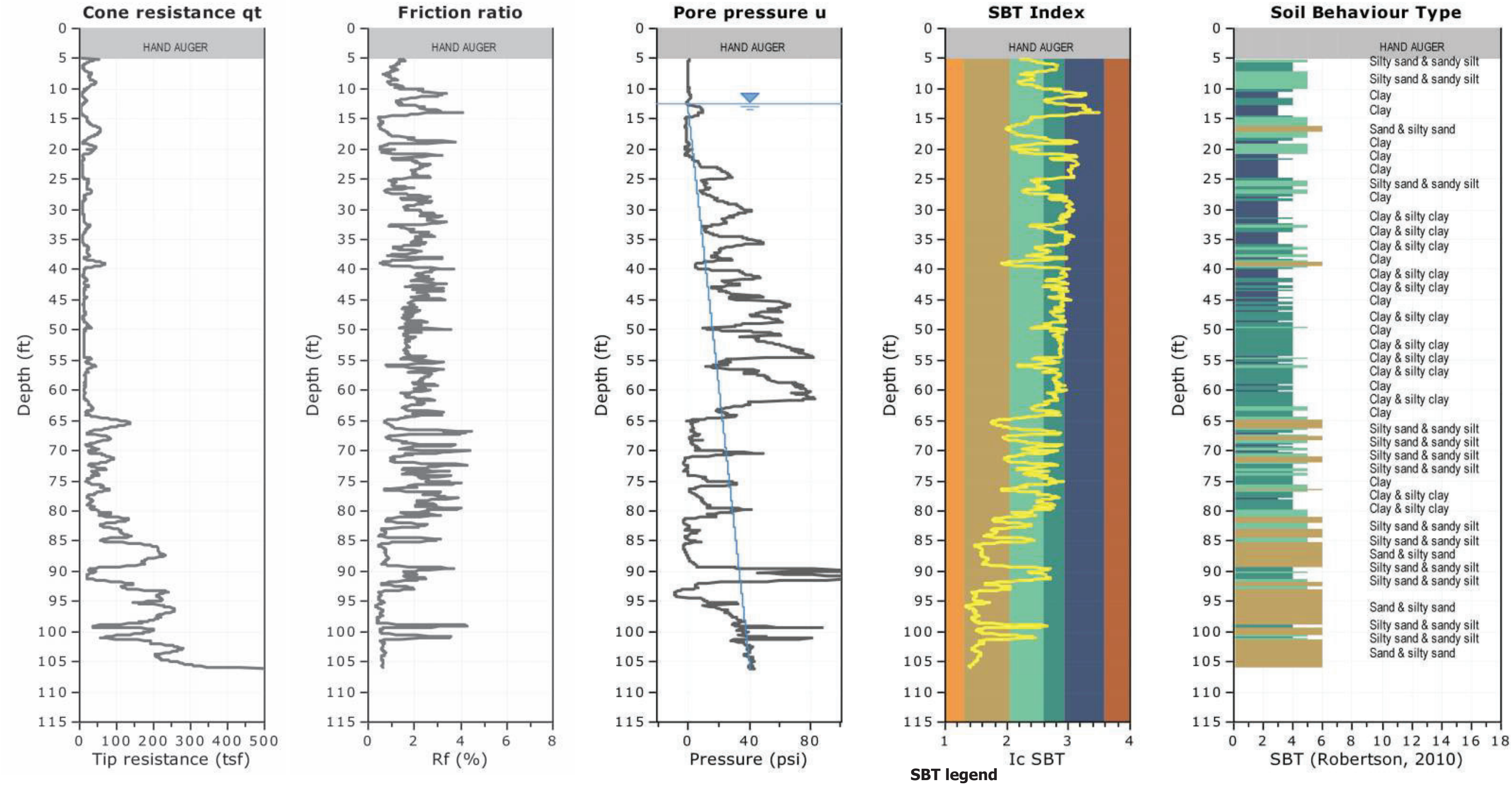
Project: **Midway Rising Sports Arena Complex**
Location: **3220, 3240, 3250, and 3500 Sports Arena Blvd, San Diego, California**

SCPT-23-024
Total depth: 106.23 ft, Date: 2/7/2023



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-12

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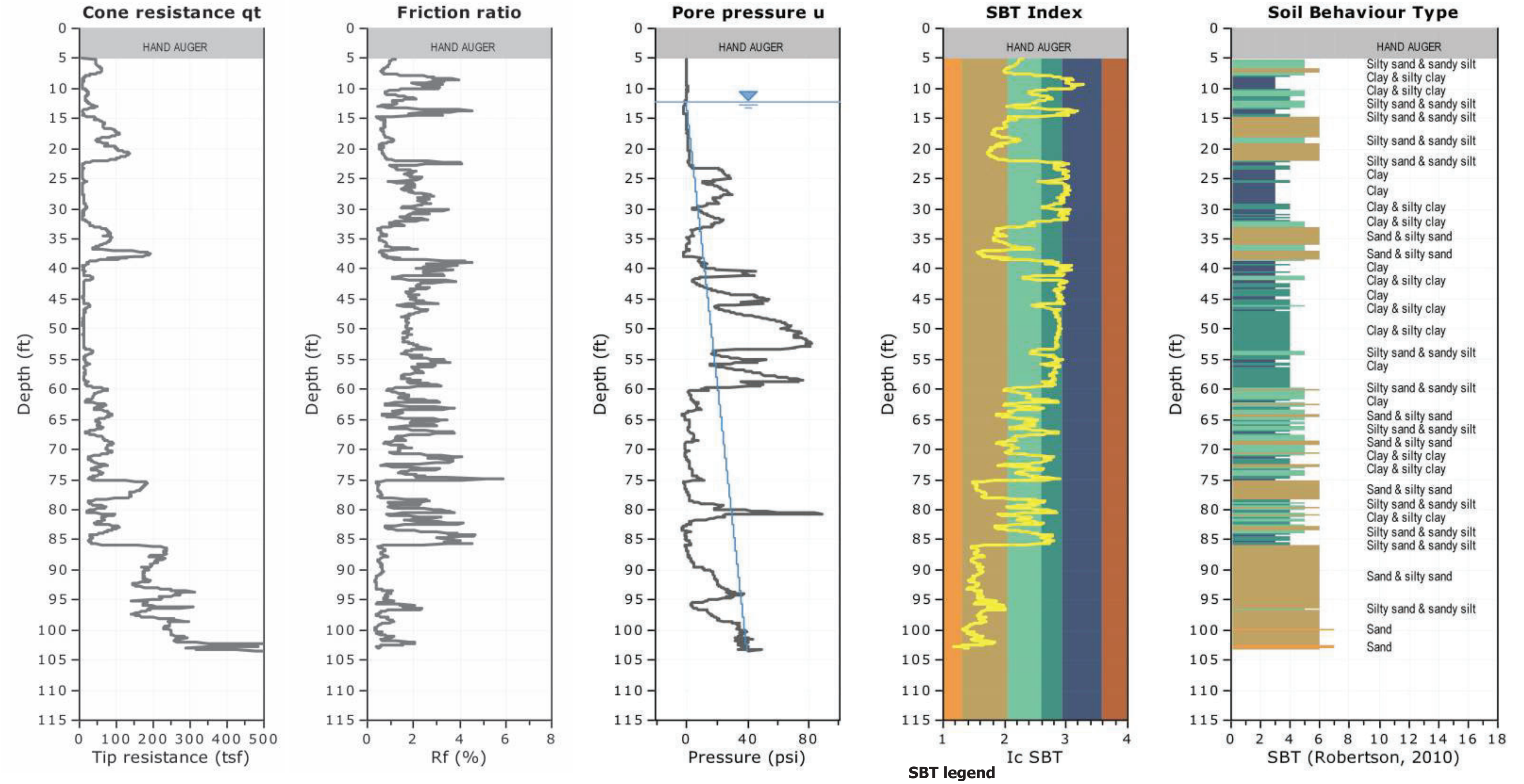
San Diego, California 92126
www.GroupDelta.com

Project: **Midway Rising Sports Arena Complex**
Location: **3220, 3240, 3250, and 3500 Sports Arena Blvd, San Diego, California**

SCPT-23-025
Total depth: 103.48 ft, Date: 2/7/2023



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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Project: **Midway Rising Sports Arena Complex**

Location: **3220, 3240, 3250, and 3500 Sports Arena Blvd, San Diego, California**

CPT-23-026

Total depth: 104.27 ft, Date: 2/7/2023

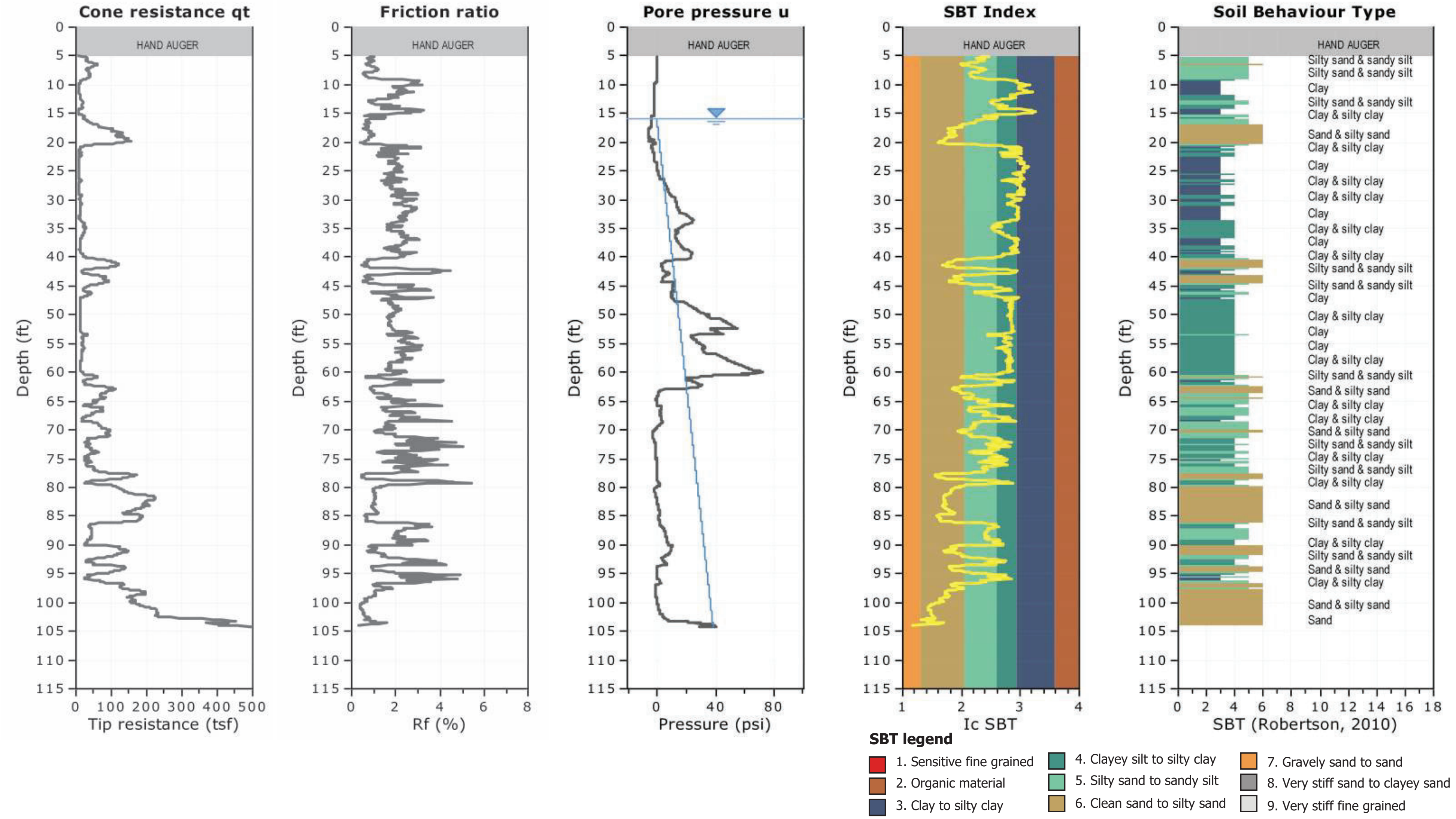

FIGURE A-14

GROUP DELTA

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Project: **Midway Rising Sports Arena Complex**
Location: **3220, 3240, 3250, and 3500 Sports Arena Blvd, San Diego, California**

CPT-23-027
Total depth: 111.56 ft, Date: 3/15/2023

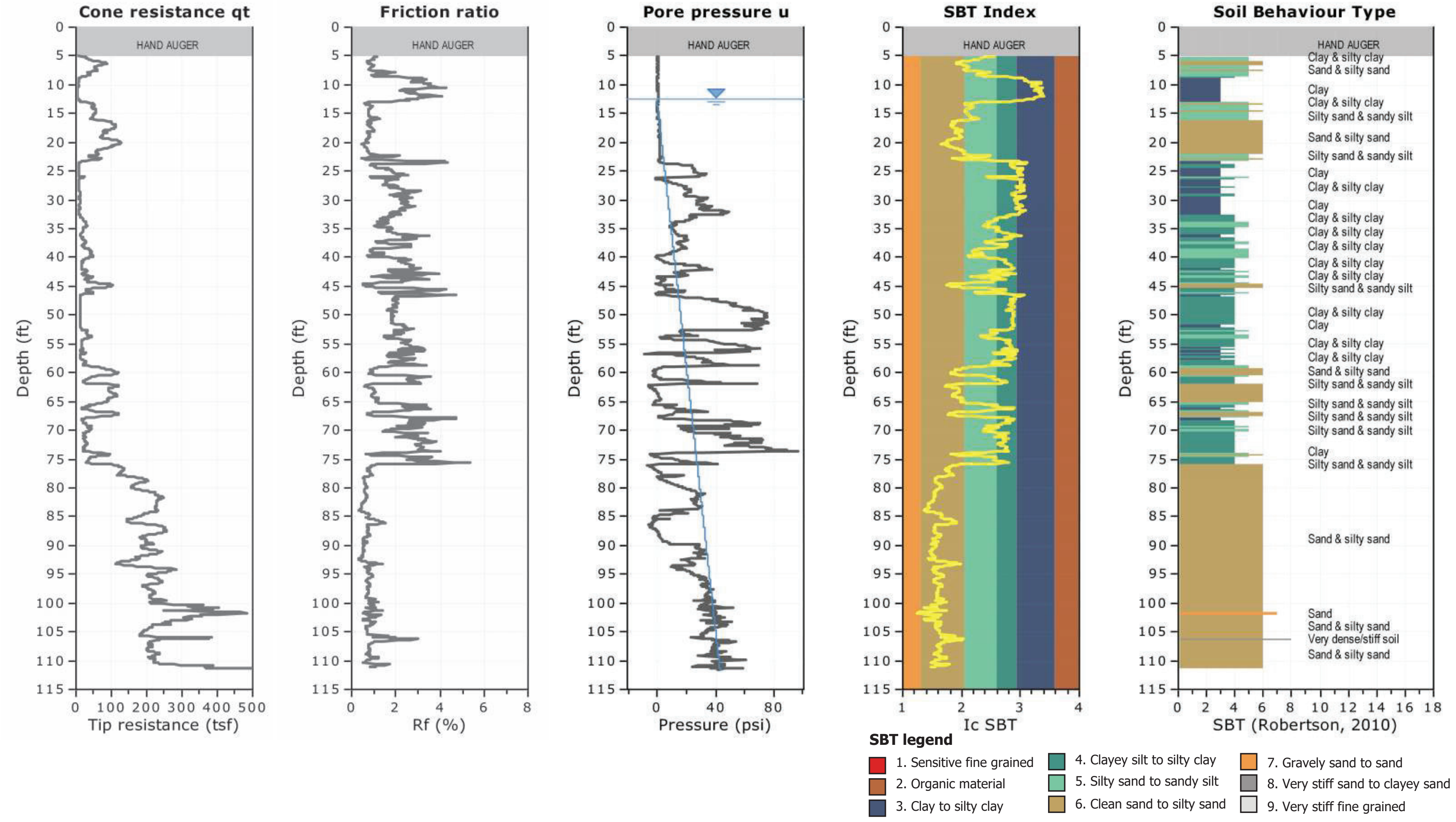

FIGURE A-15

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Project: **Midway Rising Sports Arena Complex**
Location: **3220, 3240, 3250, and 3500 Sports Arena Blvd, San Diego, California**

SCPT-23-028
Total depth: 102.97 ft, Date: 3/15/2023

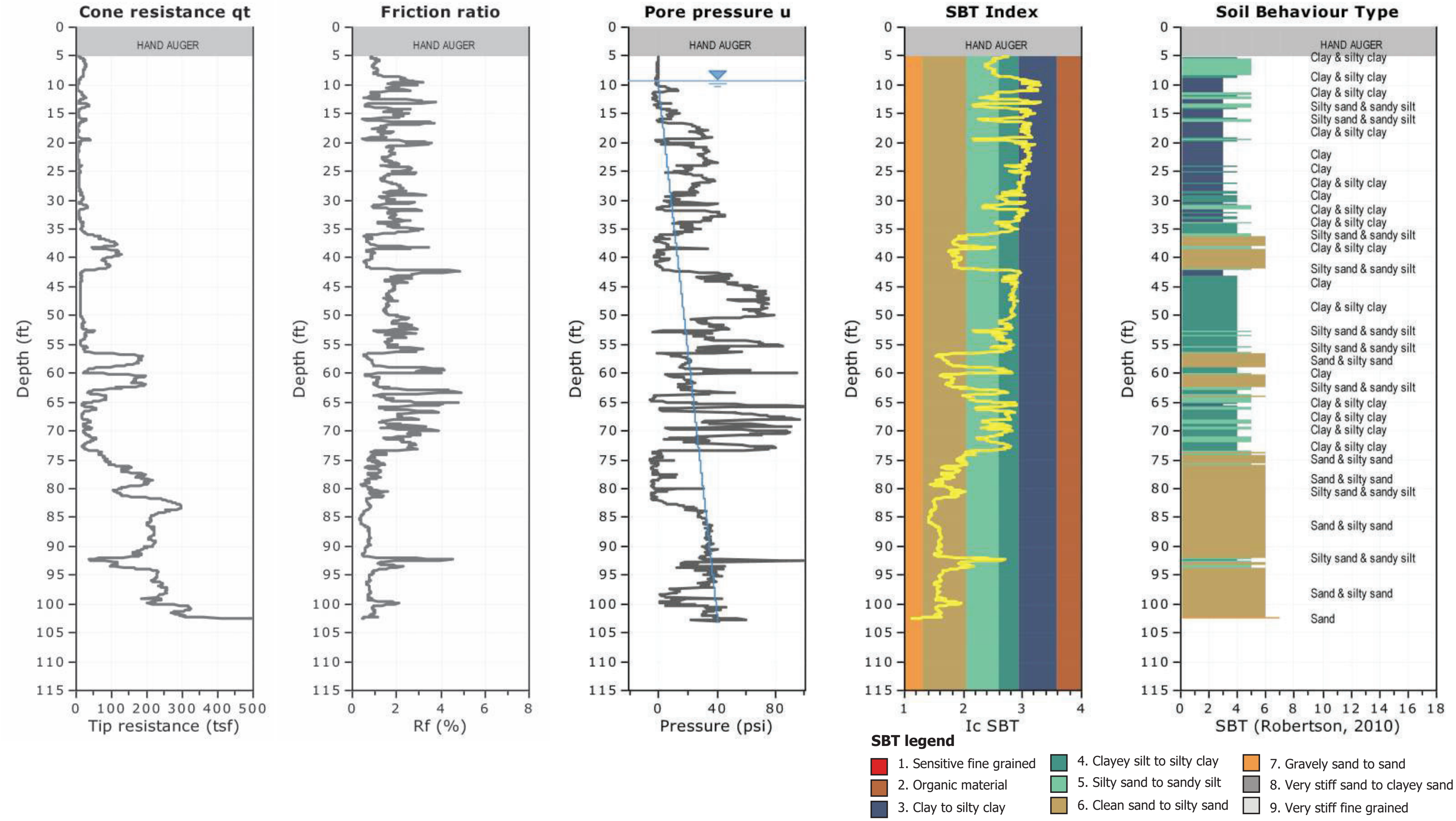

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FIGURE A-16

Midway Rising Sports Arena Complex
3220, 3240, 3250, and 3500 Sports Arena Boulevard
San Diego, California

CPT Shear Wave 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)
SCPT-23-021	5.31	4.31	4.75	6.20	766	
	10.14	9.14	9.36	16.04	583	468
	15.06	14.06	14.20	26.44	537	466
	20.18	19.18	19.28	37.68	512	452
	25.03	24.03	24.11	49.36	489	413
	30.05	29.05	29.12	59.15	492	511
	40.65	39.65	39.70	79.00	503	533
	45.41	44.41	44.46	89.16	499	468
	50.46	49.46	49.50	98.48	503	541
	55.15	54.15	54.19	106.84	507	561
	60.30	59.30	59.33	113.48	523	775
	65.06	64.06	64.09	120.12	534	716
	69.98	68.98	69.01	126.12	547	820
	75.10	74.10	74.13	133.10	557	733
	80.05	79.05	79.08	138.72	570	880
	85.17	84.17	84.19	145.60	578	744
	90.16	89.16	89.18	152.24	586	751
	95.05	94.05	94.07	158.14	595	829
	100.03	99.03	99.05	164.44	602	790

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)

Midway Rising Sports Arena Complex
3220, 3240, 3250, and 3500 Sports Arena Boulevard
San Diego, California

CPT Shear Wave 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)
SCPT-23-022	5.02	4.02	4.49	5.88	764	
	10.27	9.27	9.48	12.06	786	808
	15.03	14.03	14.17	22.68	625	441
	20.05	19.05	19.15	32.24	594	521
	25.03	24.03	24.11	44.52	542	404
	30.09	29.09	29.16	55.76	523	449
	35.07	34.07	34.13	63.04	541	683
	40.06	39.06	39.11	70.64	554	656
	45.05	44.05	44.10	79.96	551	535
	50.00	49.00	49.04	89.92	545	497
	55.02	54.02	54.06	98.18	551	607
	60.04	59.04	59.07	106.10	557	633
	65.06	64.06	64.09	113.92	563	642
	70.05	69.05	69.08	120.32	574	779
	75.03	74.03	74.06	127.20	582	724
	80.02	79.02	79.05	134.78	586	658
	85.04	84.04	84.06	140.38	599	896
	90.03	89.03	89.05	145.16	613	1044
	95.01	94.01	94.03	149.44	629	1163
	100.00	99.00	99.02	153.96	643	1104

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)

Midway Rising Sports Arena Complex
3220, 3240, 3250, and 3500 Sports Arena Boulevard
San Diego, California

CPT Shear Wave 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)
SCPT-23-024	5.05	4.05	4.52	5.80	779	
	10.07	9.07	9.29	11.96	777	775
	15.06	14.06	14.20	22.88	621	450
	20.08	19.08	19.18	31.36	612	588
	25.07	24.07	24.15	41.42	583	494
	30.02	29.02	29.09	51.60	564	485
	35.10	34.10	34.16	60.60	564	563
	40.06	39.06	39.11	68.92	567	595
	45.11	44.11	44.16	79.10	558	495
	50.13	49.13	49.17	87.34	563	609
	55.41	54.41	54.45	96.30	565	589
	60.07	59.07	59.10	102.86	575	710
	65.06	64.06	64.09	111.36	576	587
	70.05	69.05	69.08	118.40	583	708
	75.07	74.07	74.10	124.84	594	779
	80.09	79.09	79.12	130.60	606	871
	85.10	84.10	84.12	137.40	612	737
	90.03	89.03	89.05	143.14	622	859
	95.11	94.11	94.13	149.58	629	789
	100.00	99.00	99.02	155.62	636	809

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)

Midway Rising Sports Arena Complex
3220, 3240, 3250, and 3500 Sports Arena Boulevard
San Diego, California

CPT Shear Wave 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)
SCPT-23-025	4.99	3.99	4.46	4.28	1043	
	10.04	9.04	9.26	10.92	848	722
	15.06	14.06	14.20	20.42	695	520
	20.14	19.14	19.24	28.56	674	619
	25.07	24.07	24.15	37.98	636	521
	30.05	29.05	29.12	49.24	591	441
	35.07	34.07	34.13	58.12	587	564
	40.16	39.16	39.21	65.96	594	648
	45.37	44.37	44.42	74.82	594	587
	50.07	49.07	49.11	83.84	586	521
	55.09	54.09	54.13	92.92	583	552
	60.01	59.01	59.04	101.28	583	588
	65.09	64.09	64.12	107.92	594	765
	70.11	69.11	69.14	114.76	602	734
	75.03	74.03	74.06	121.20	611	764
	80.09	79.09	79.12	126.56	625	944
	85.10	84.10	84.12	133.40	631	732
	90.09	89.09	89.11	140.48	634	705
	95.08	94.08	94.10	144.86	650	1139
	100.03	99.03	99.05	149.68	662	1027

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)

Midway Rising Sports Arena Complex
3220, 3240, 3250, and 3500 Sports Arena Boulevard
San Diego, California

CPT Shear Wave 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)
SCPT-23-028	5.02	4.02	4.49	4.26	1054	
	10.04	9.04	9.26	15.76	587	415
	15.03	14.03	14.17	26.20	541	471
	20.05	19.05	19.15	38.22	501	415
	25.07	24.07	24.15	51.28	471	383
	30.02	29.02	29.09	62.18	468	453
	35.04	34.04	34.10	71.20	479	555
	40.06	39.06	39.11	78.80	496	660
	45.05	44.05	44.10	86.40	510	656
	50.49	49.49	49.53	93.78	528	736
	55.45	54.45	54.49	101.08	539	679
	60.07	59.07	59.10	107.28	551	745
	65.09	64.09	64.12	113.38	566	823
	70.08	69.08	69.11	121.64	568	604
	75.07	74.07	74.10	128.04	579	779
	80.05	79.05	79.08	133.60	592	895
	85.04	84.04	84.06	138.52	607	1014
	90.06	89.06	89.08	144.02	619	912
	95.08	94.08	94.10	148.80	632	1050
	100.07	99.07	99.09	152.68	649	1286

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)

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-1.1 through B-1.17.

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-1.16 and are also summarized in Figure B-2.1 and B-2.2.

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-3. Figure B-3 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-4.

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-4, 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-4.

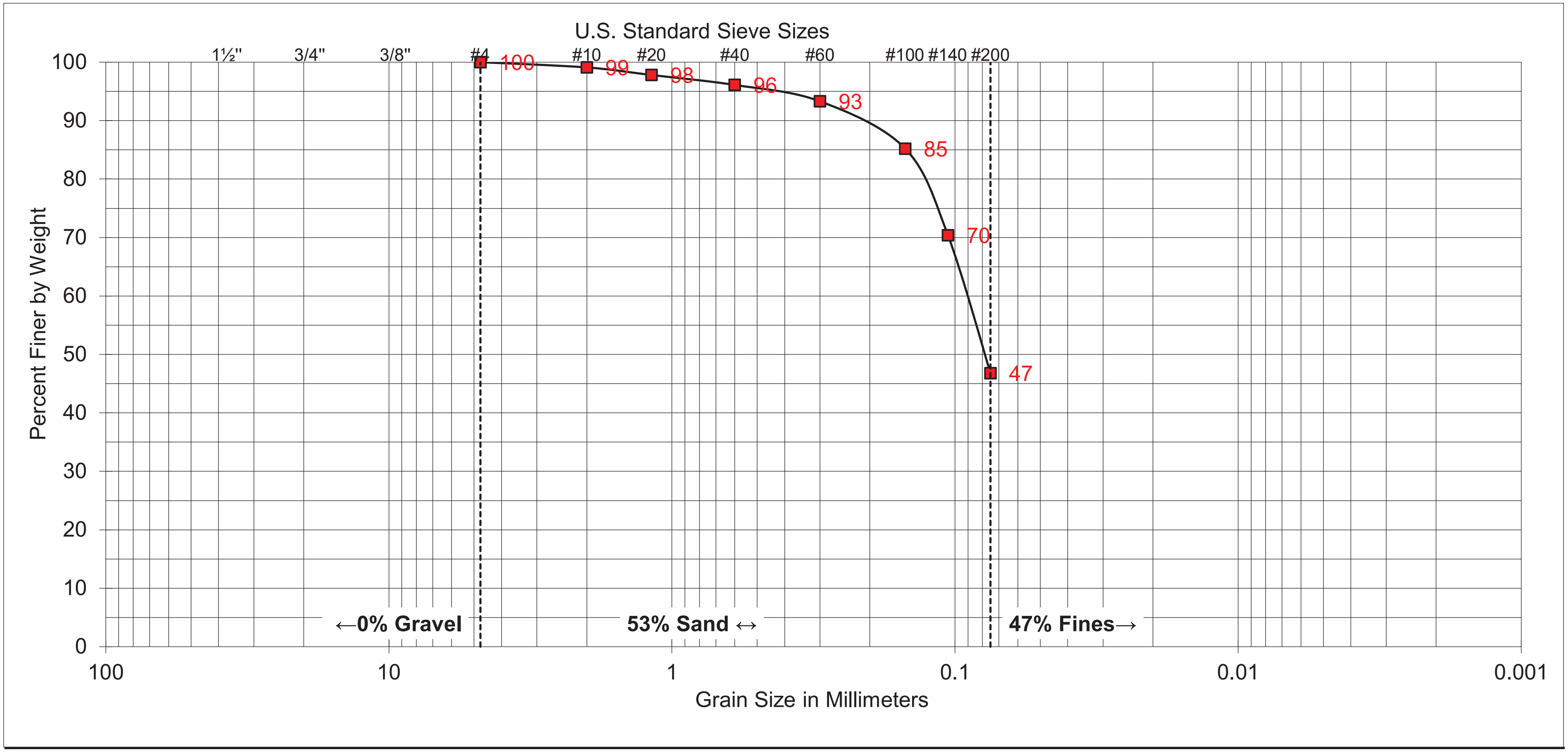
APPENDIX B

LABORATORY TESTING (Continued)

Direct Shear: The shear strength of selected partially intact samples of the soils from the site were assessed using direct shear testing performed in general accordance with ASTM D3080. The test results are shown in Figures B-5.1 through B-5.4.

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.



<table>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td><td rowspan="2">SILT AND
CLAY</td></tr>
<tr><td colspan="2">GRAVEL</td><td colspan="3">SAND</td></tr>
</table>

SAMPLE

EXPLORATION ID: A-23-011

SAMPLE DEPTH: 0.5' - 5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

ATTERBERG LIMITS

LIQUID LIMIT: --

PLASTIC LIMIT: --

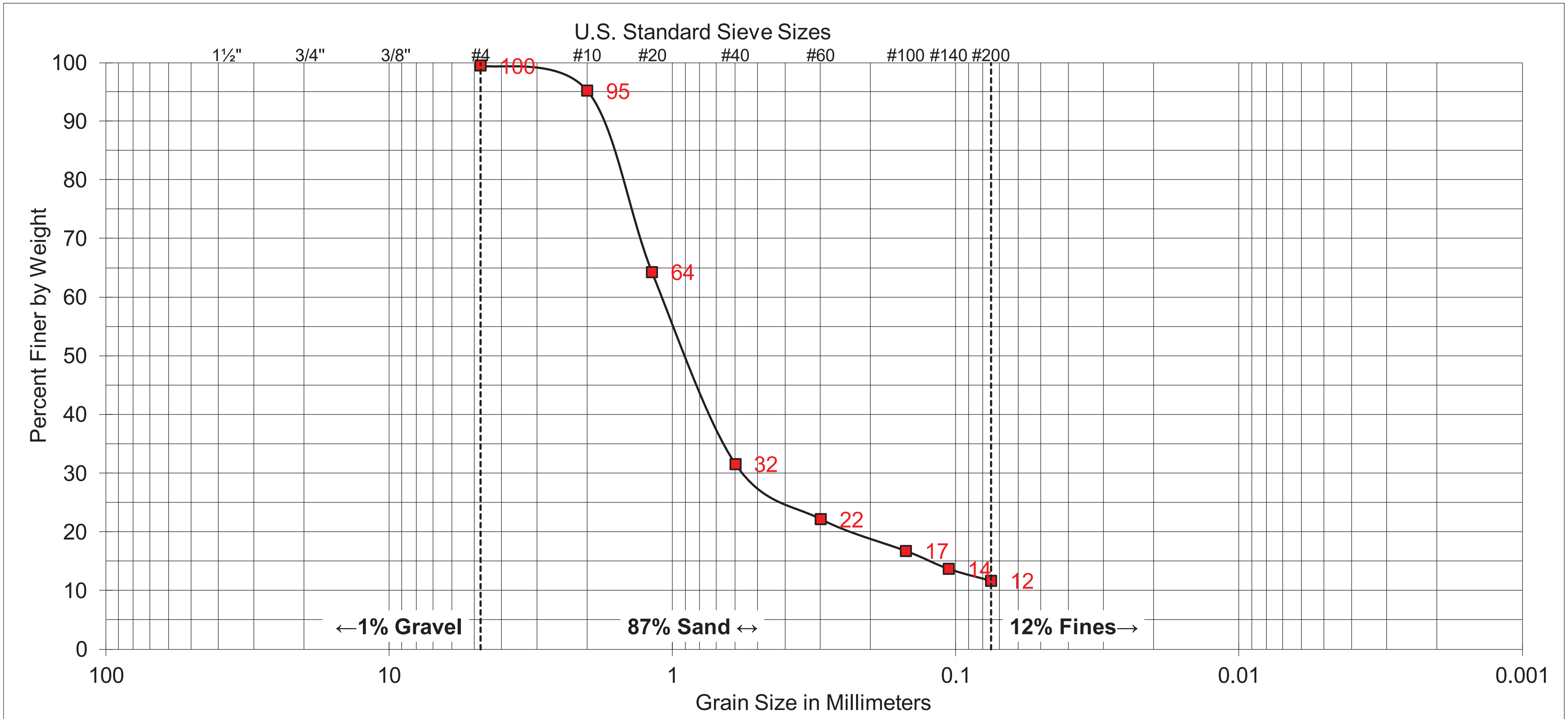
PLASTICITY INDEX: --

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.1



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

EXPLORATION ID: A-23-012
SAMPLE DEPTH: 31-31.5'

UNIFIED SOIL CLASSIFICATION: SP-SM

DESCRIPTION: POORLY GRADED SAND WITH SILT

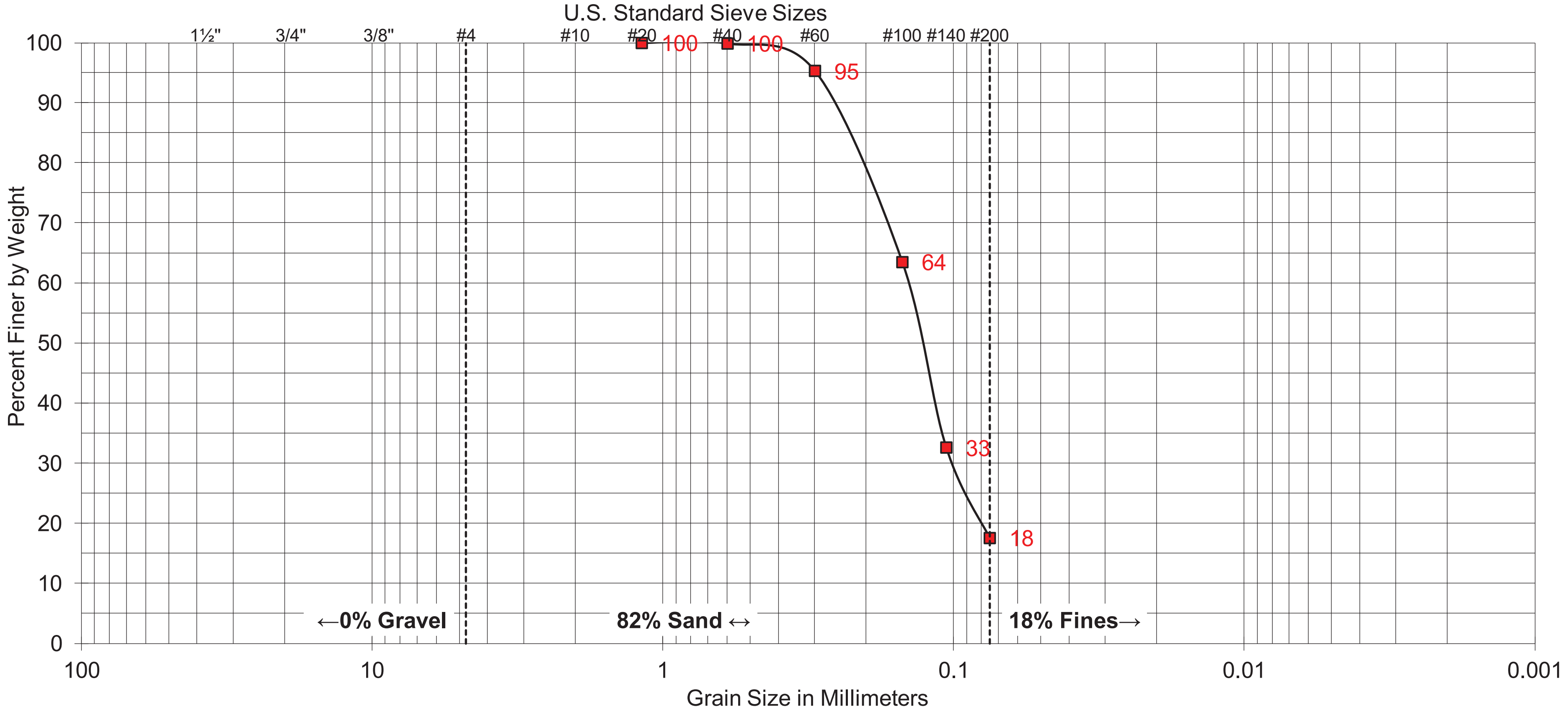
ATTERBERG LIMITS

LIQUID LIMIT: --
PLASTIC LIMIT: --
PLASTICITY INDEX: --

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760
FIGURE B-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	A-23-013
SAMPLE DEPTH:	16-16.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

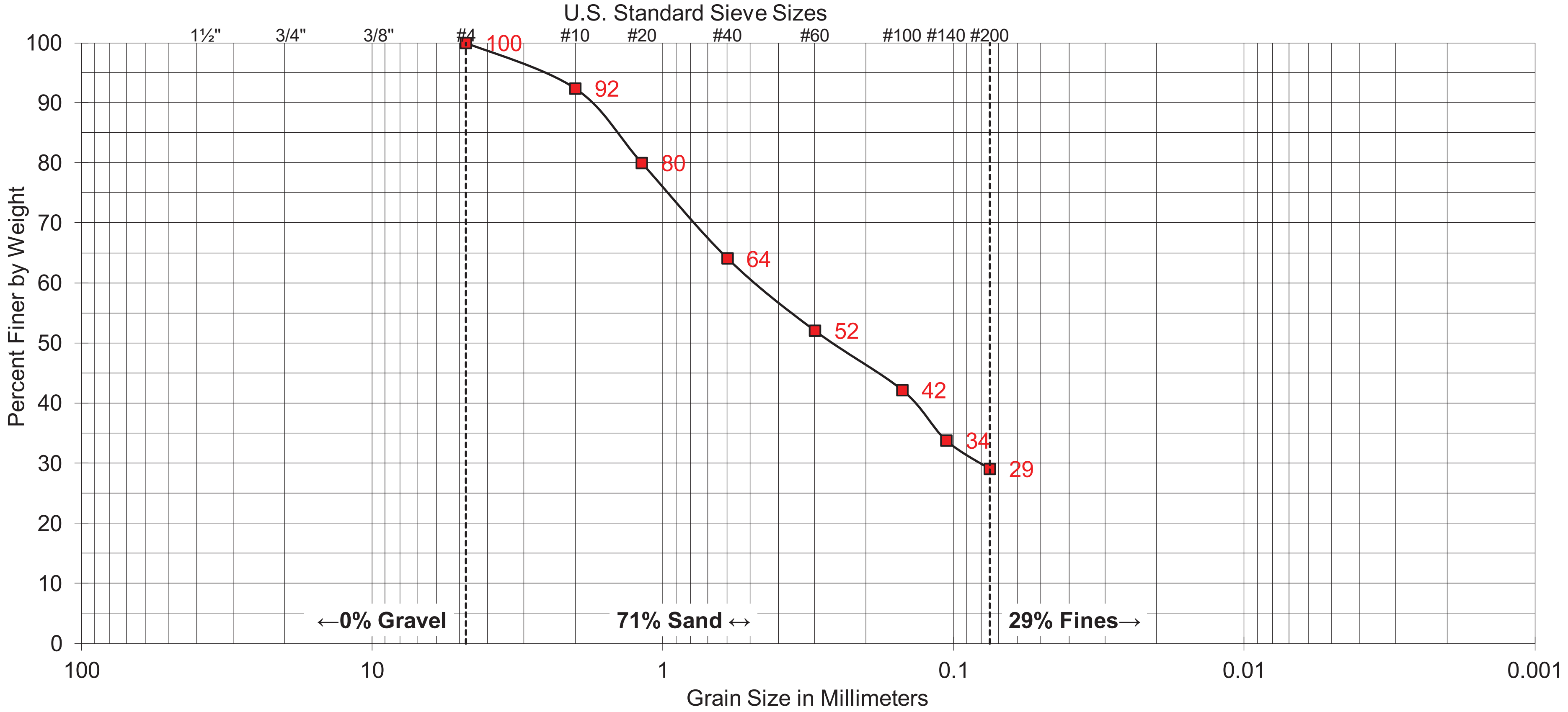
ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.3



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	A-23-014
SAMPLE DEPTH:	5-6.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

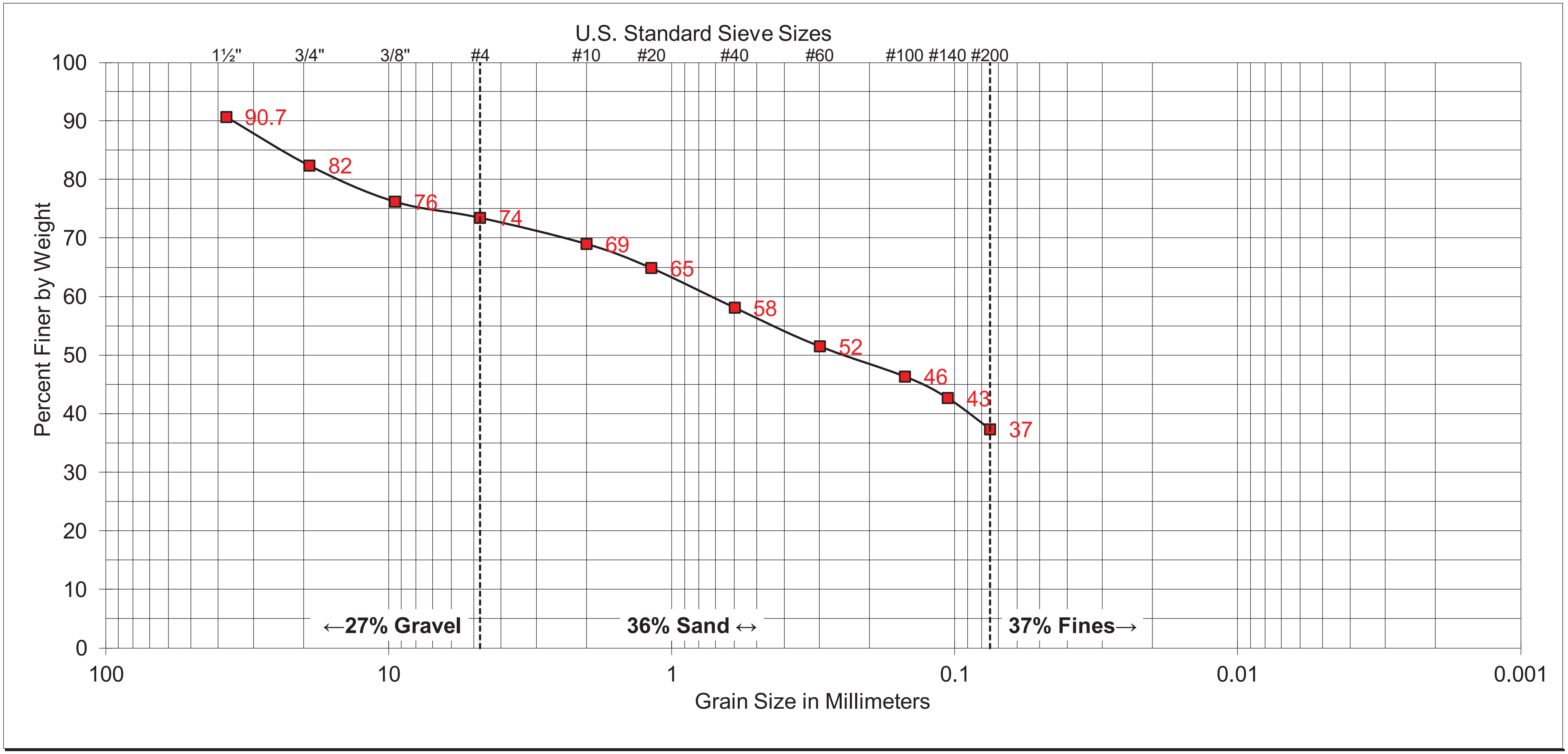
ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.4



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE

EXPLORATION ID: A-23-015

SAMPLE DEPTH: 0.5-5'

UNIFIED SOIL CLASSIFICATION: SC-SM

DESCRIPTION: CLAYEY SAND WITH GRAVEL

ATTERBERG LIMITS

LIQUID LIMIT: --

PLASTIC LIMIT: --

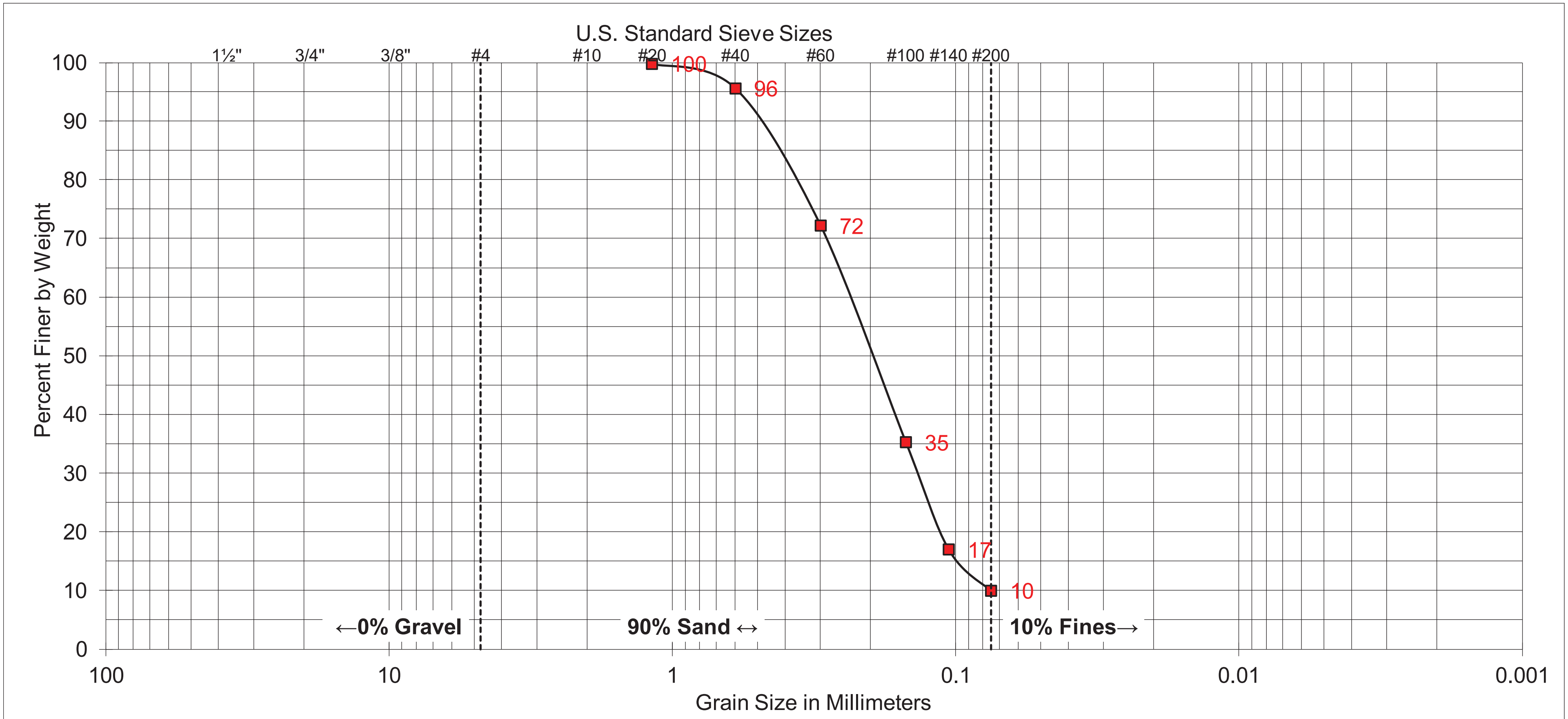
PLASTICITY INDEX: --

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.5



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	A-23-015
SAMPLE DEPTH:	15-16.5'

UNIFIED SOIL CLASSIFICATION: SP-SM

DESCRIPTION: POORLY GRADED SAND WITH SILT

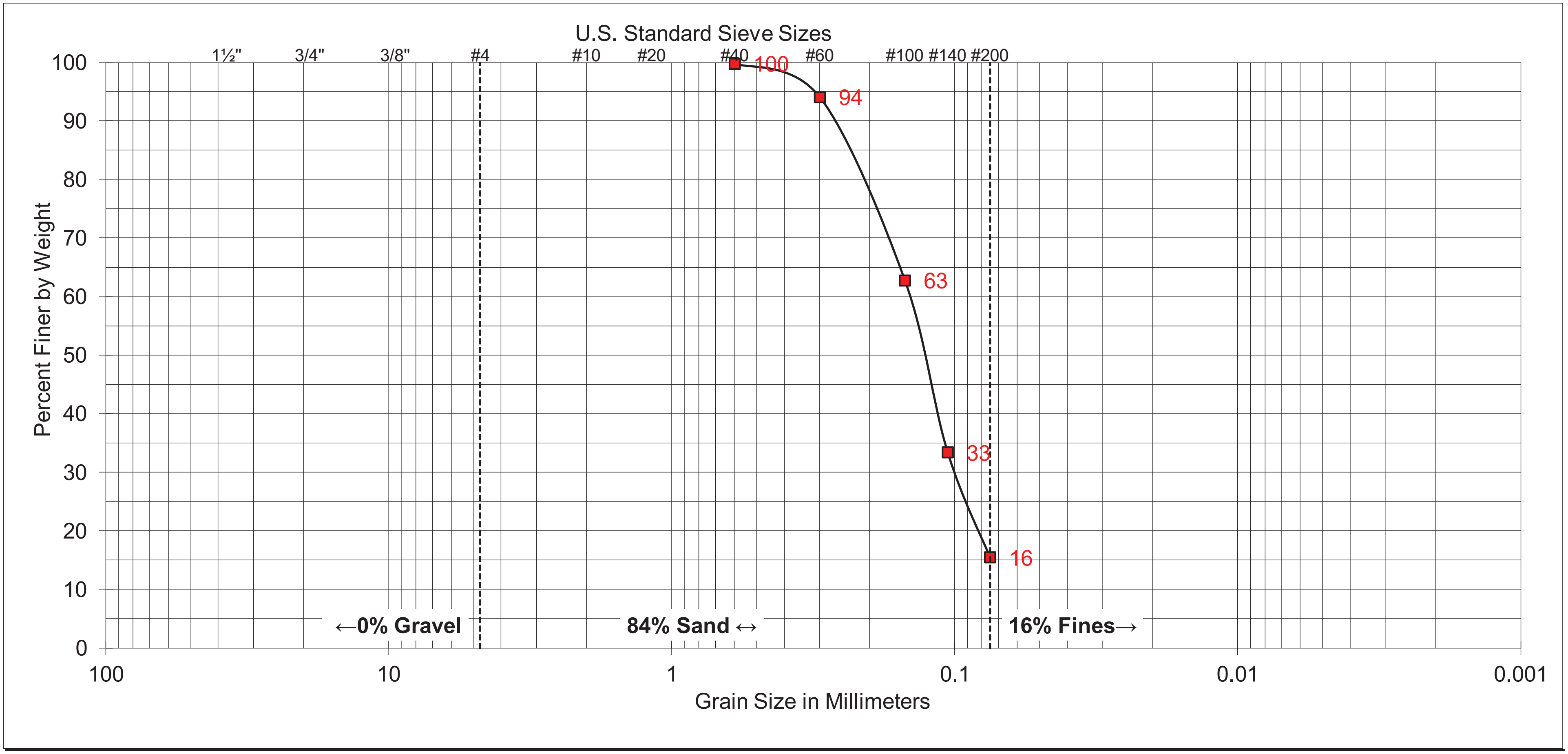
ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.6



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

EXPLORATION ID: A-23-016

SAMPLE DEPTH: 0.5-5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

ATTERBERG LIMITS

LIQUID LIMIT: --

PLASTIC LIMIT: --

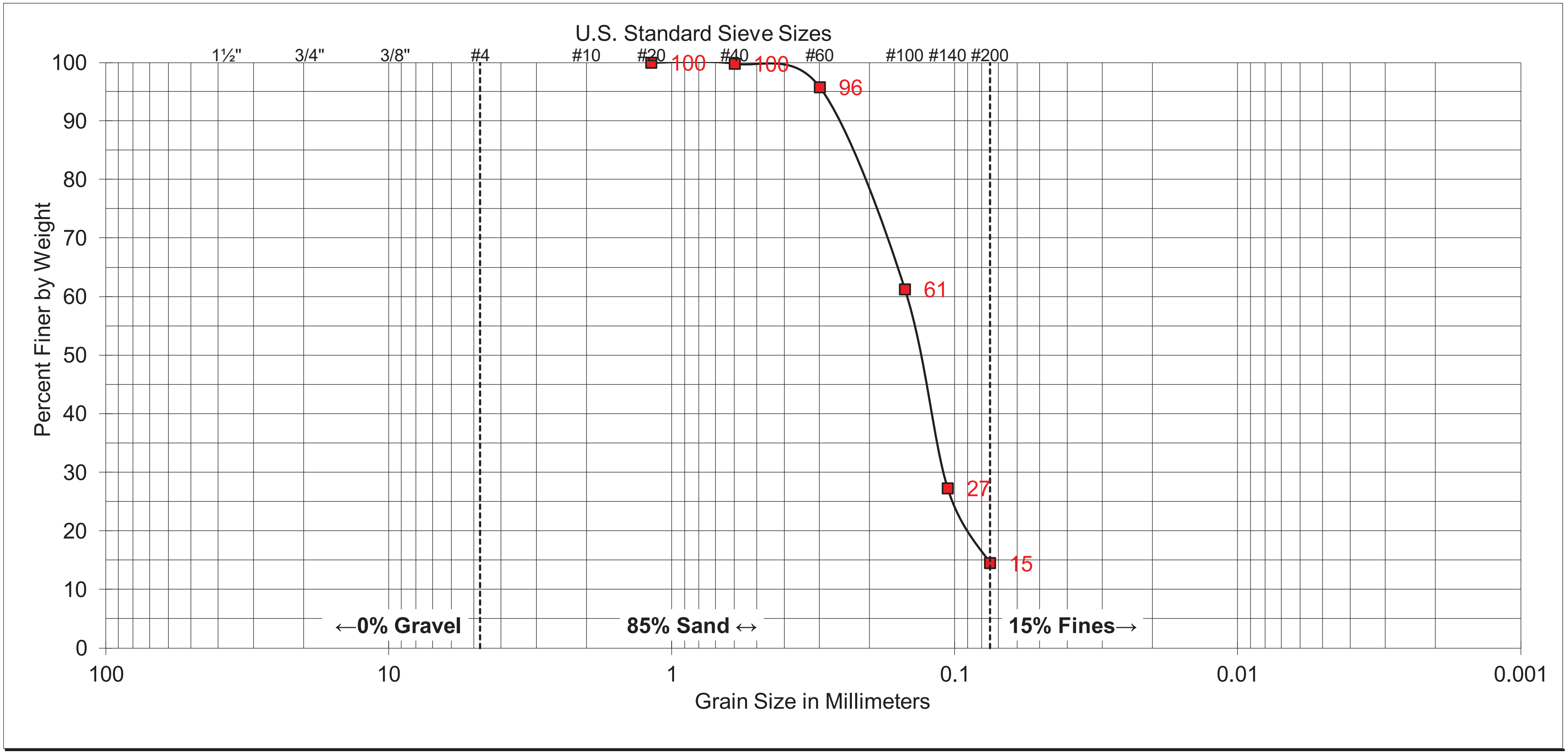
PLASTICITY INDEX: --

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.7



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	A-23-016
SAMPLE DEPTH:	40-41.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

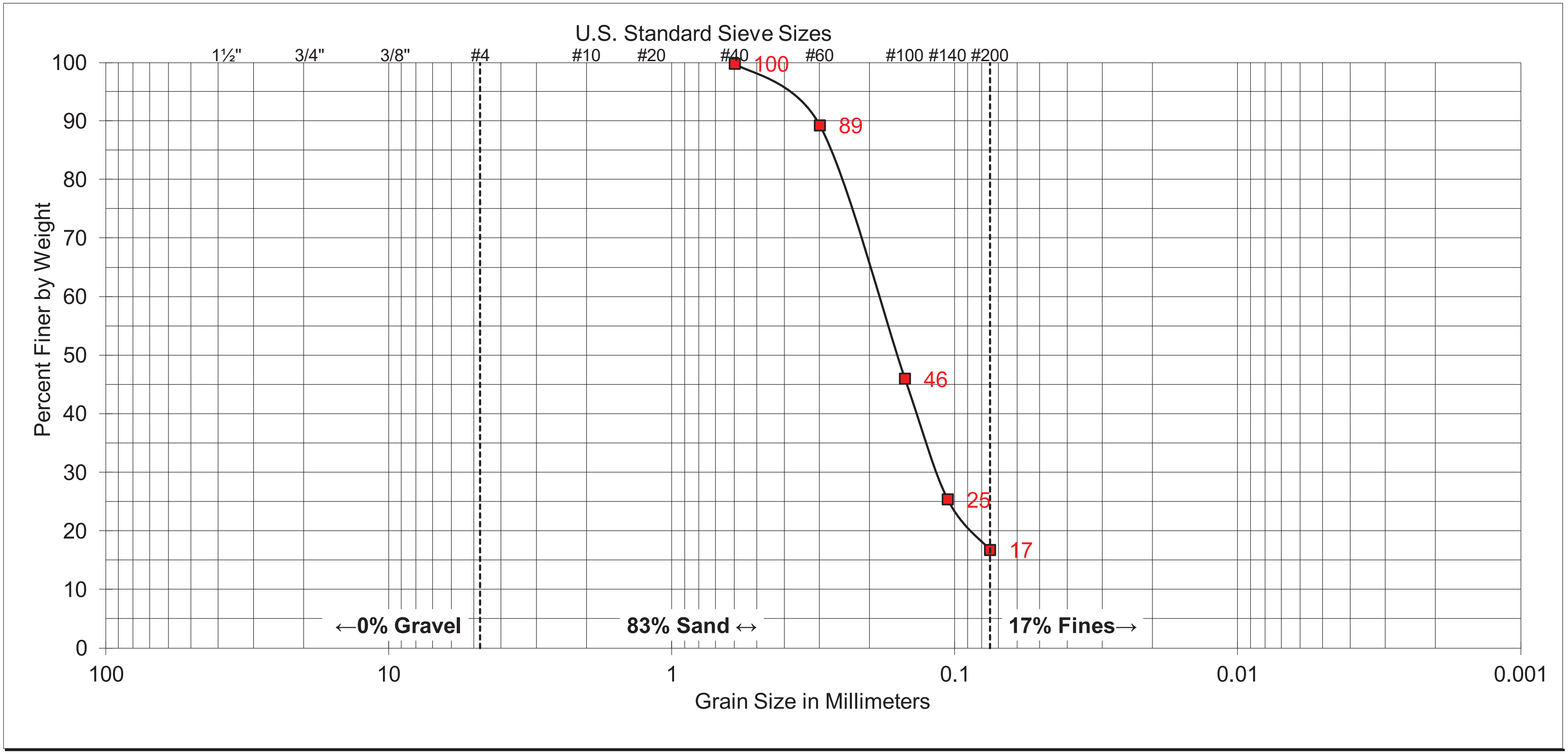
ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.8



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	R-23-001
SAMPLE DEPTH:	21-21.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

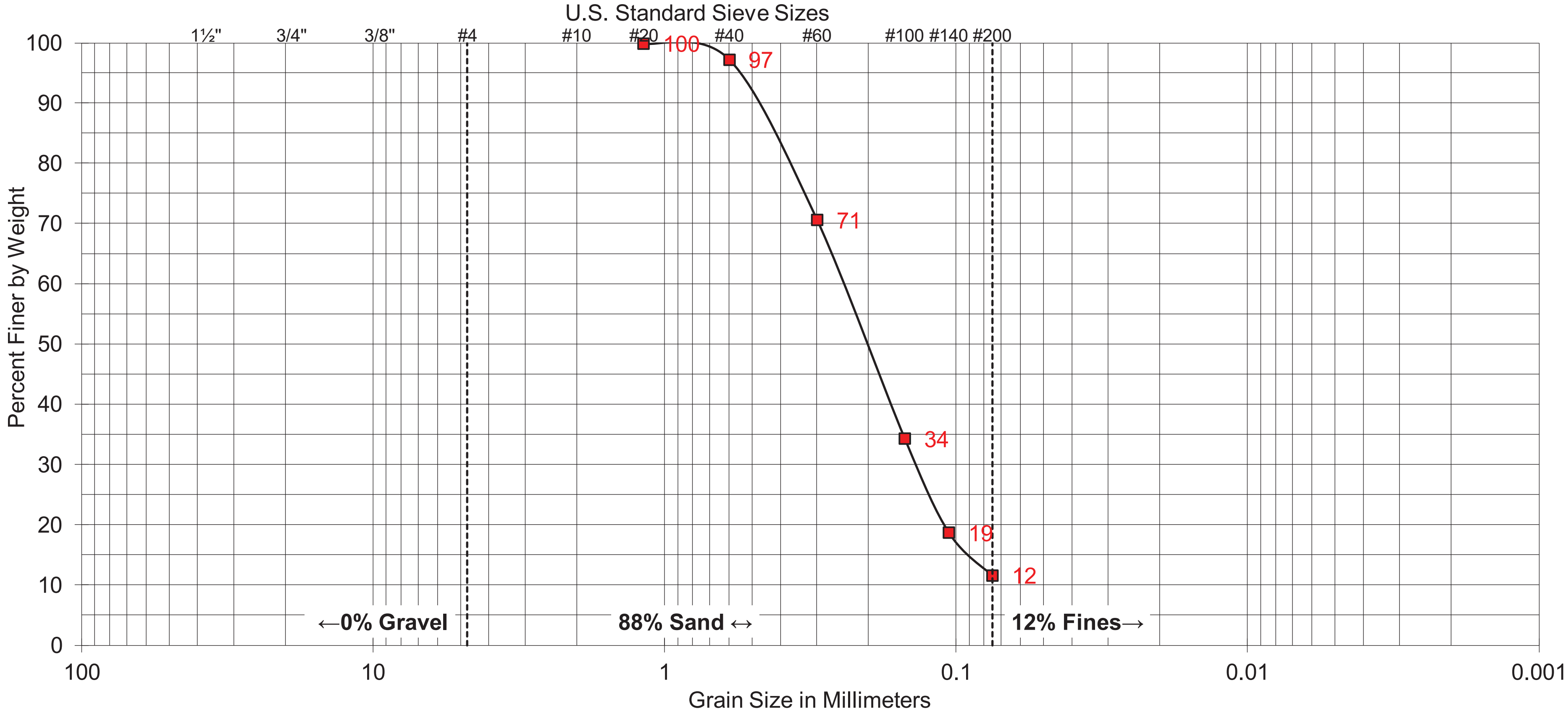
ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.9



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

EXPLORATION ID: R-23-001

SAMPLE DEPTH: 31-31.5'

UNIFIED SOIL CLASSIFICATION: SP-SM

DESCRIPTION: POORLY GRADED SAND WITH SILT

ATTERBERG LIMITS

LIQUID LIMIT: --

PLASTIC LIMIT: --

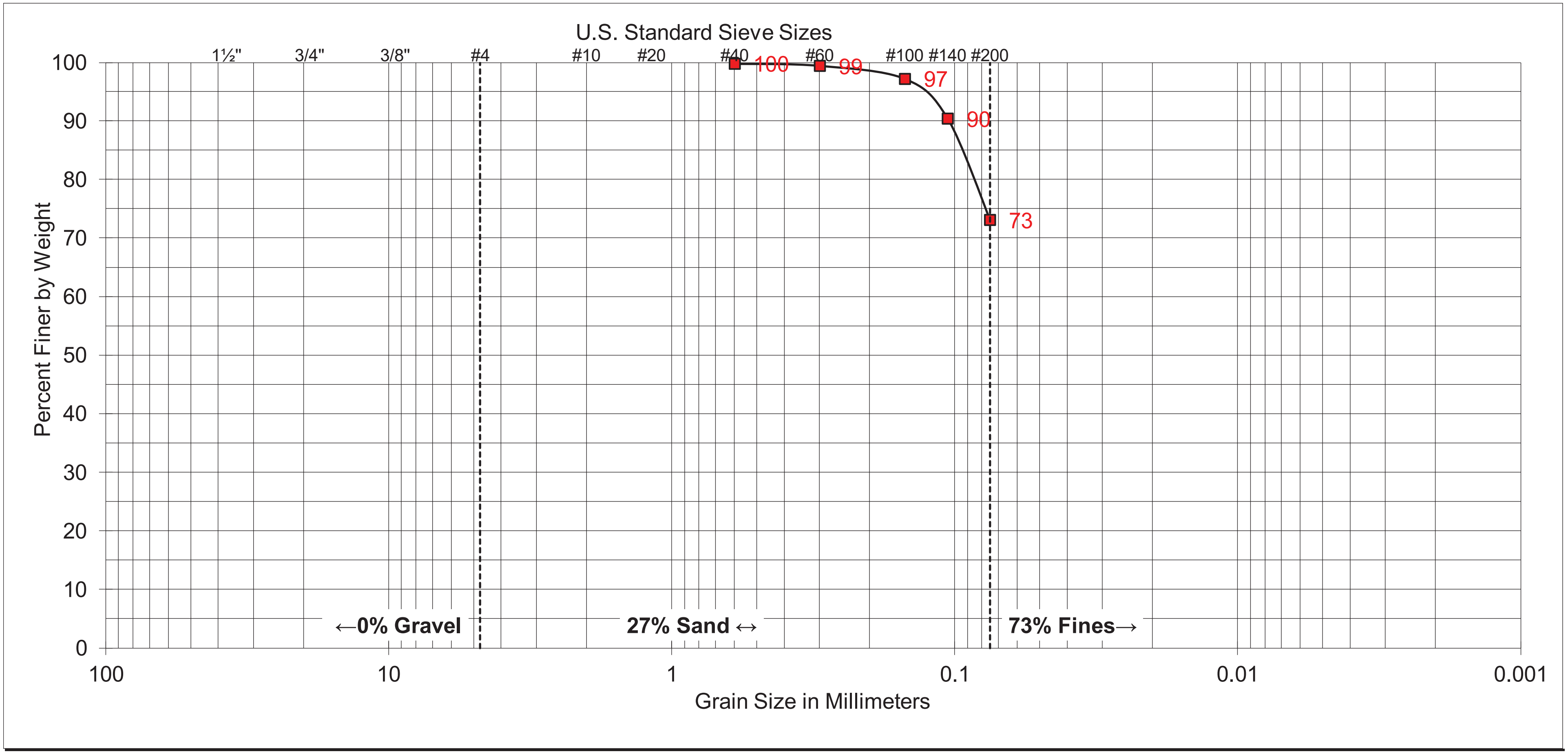
PLASTICITY INDEX: --

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.10



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

EXPLORATION ID:	R-23-001
SAMPLE DEPTH:	61-61.5'

UNIFIED SOIL CLASSIFICATION: ML

DESCRIPTION: SILT WITH SAND

ATTERBERG LIMITS

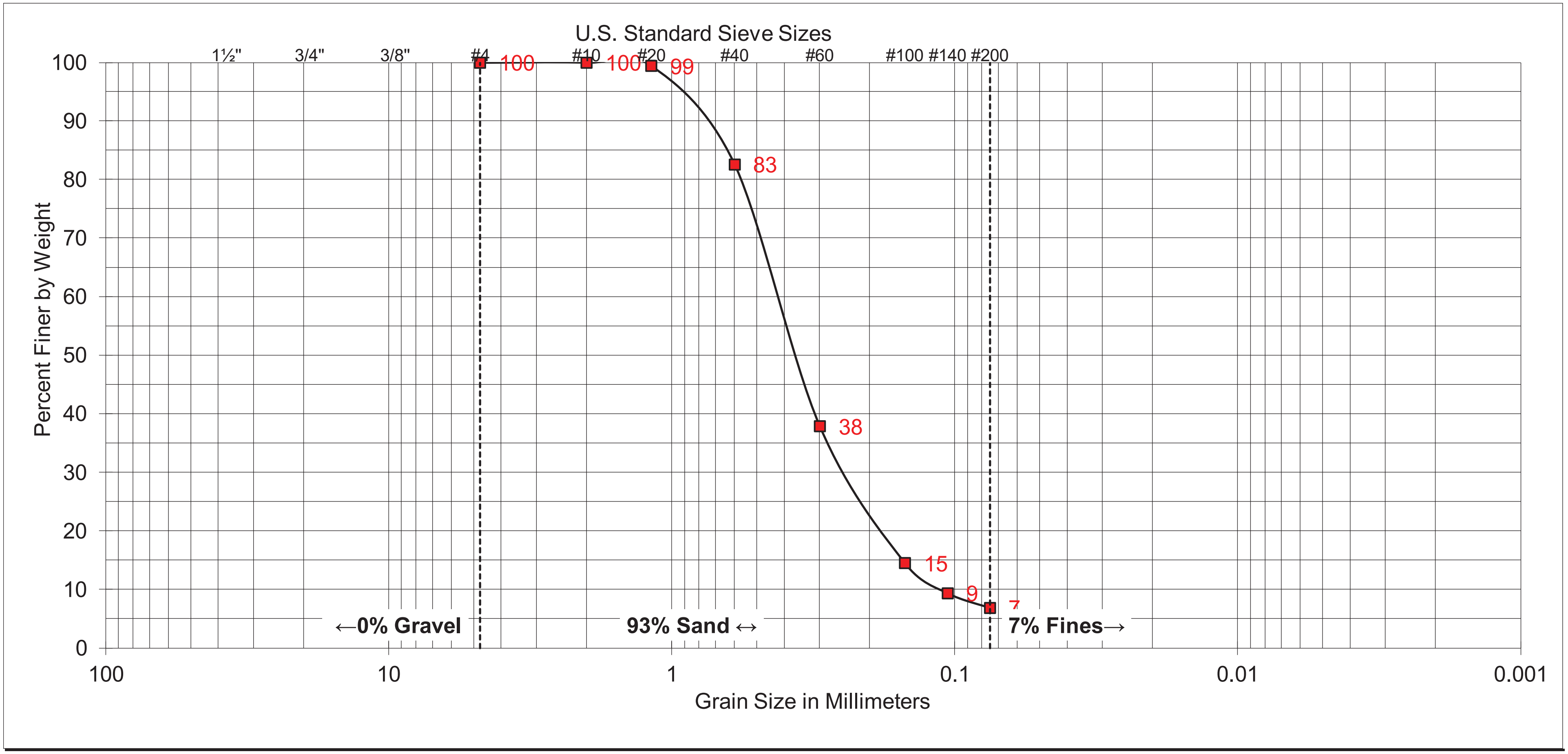
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.11



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE

EXPLORATION ID:	R-23-001
SAMPLE DEPTH:	91-91.5'

UNIFIED SOIL CLASSIFICATION: SP-SM

DESCRIPTION: POORLY GRADED SAND WITH SILT

ATTERBERG LIMITS

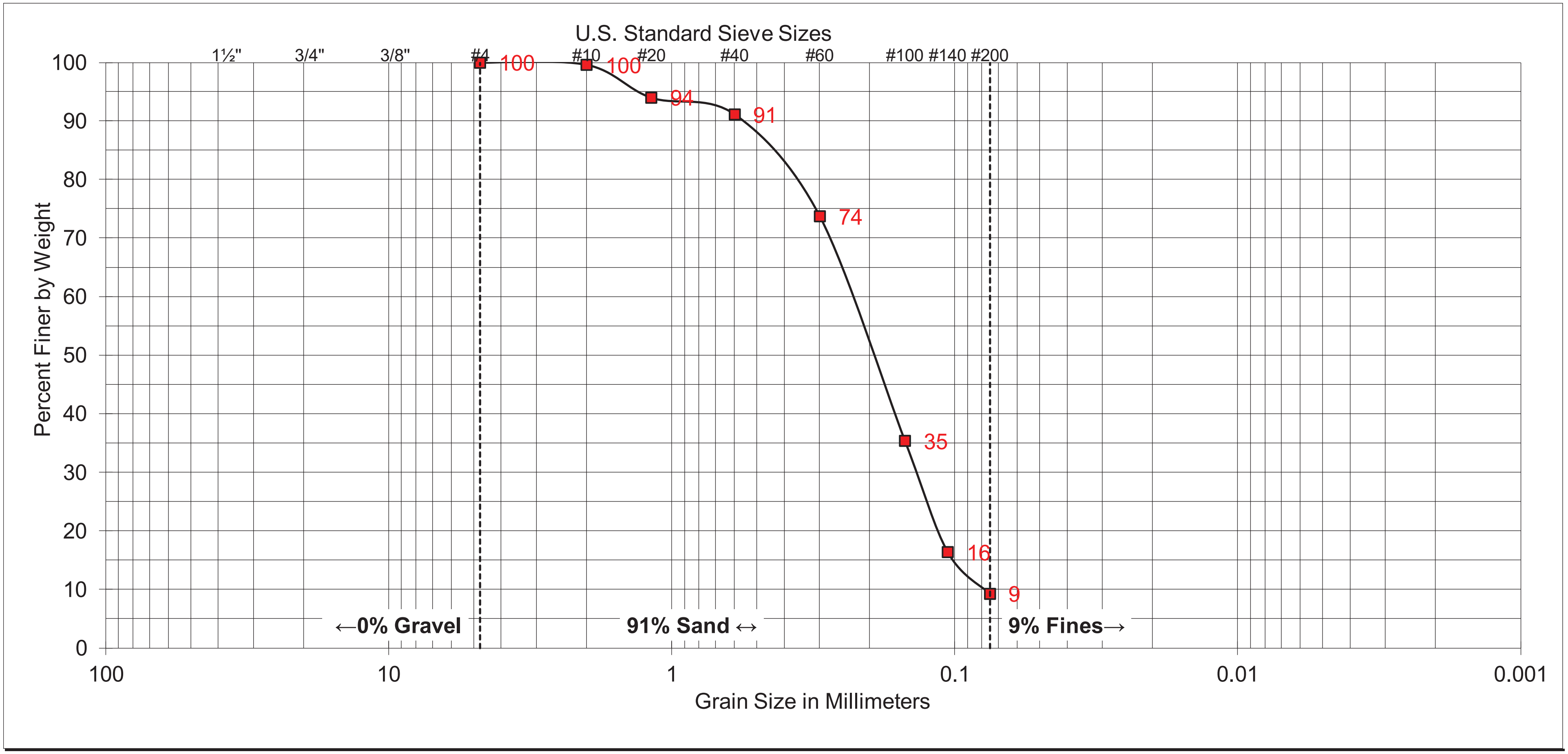
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.12



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

EXPLORATION ID: R-23-002

SAMPLE DEPTH: 25-26.5'

UNIFIED SOIL CLASSIFICATION: SP-SM

DESCRIPTION: POORLY GRADED SAND WITH SILT

ATTERBERG LIMITS

LIQUID LIMIT: --

PLASTIC LIMIT: --

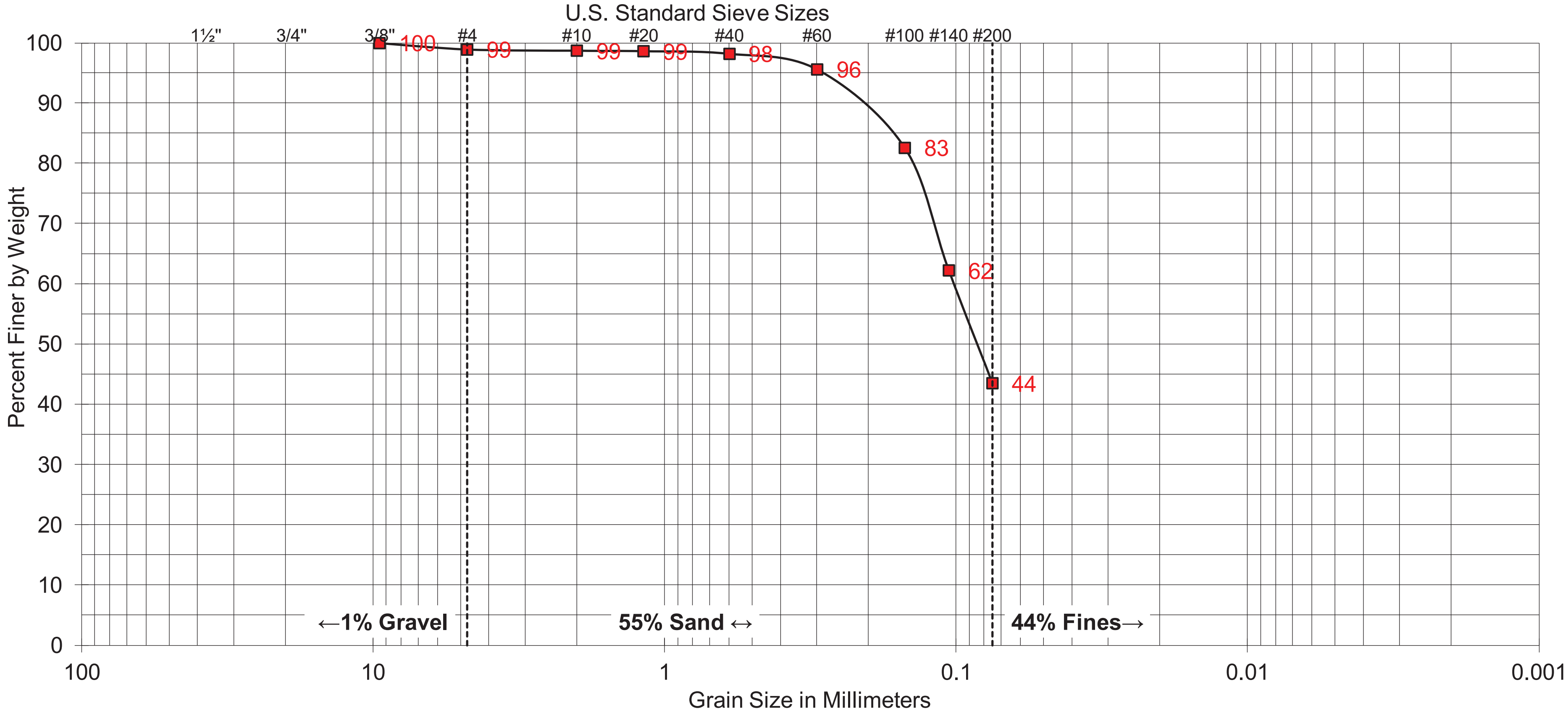
PLASTICITY INDEX: --

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.13



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	R-23-002
SAMPLE DEPTH:	55-56.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

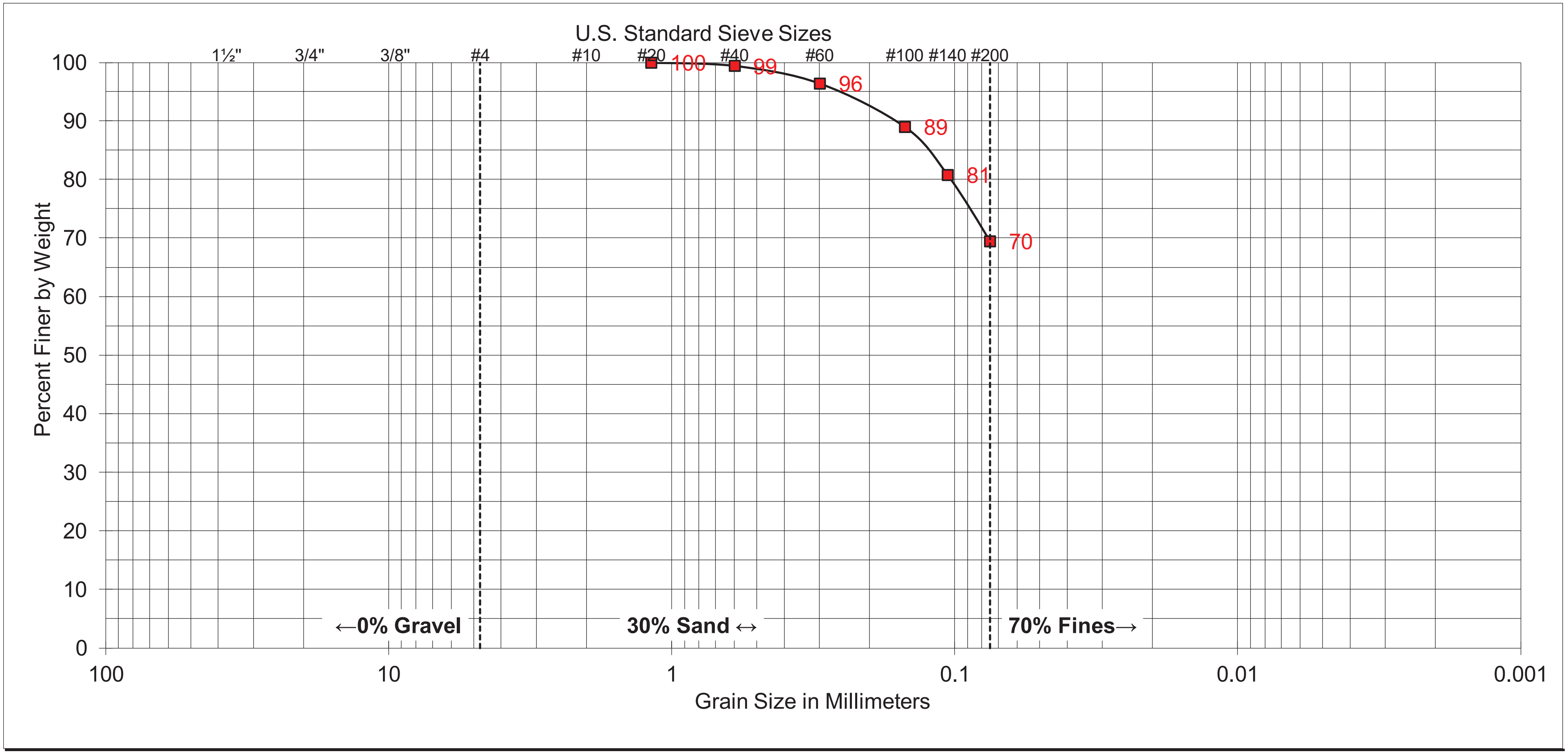
ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.14



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	R-23-002
SAMPLE DEPTH:	66-66.5'

UNIFIED SOIL CLASSIFICATION: ML

DESCRIPTION: SANDY SILT

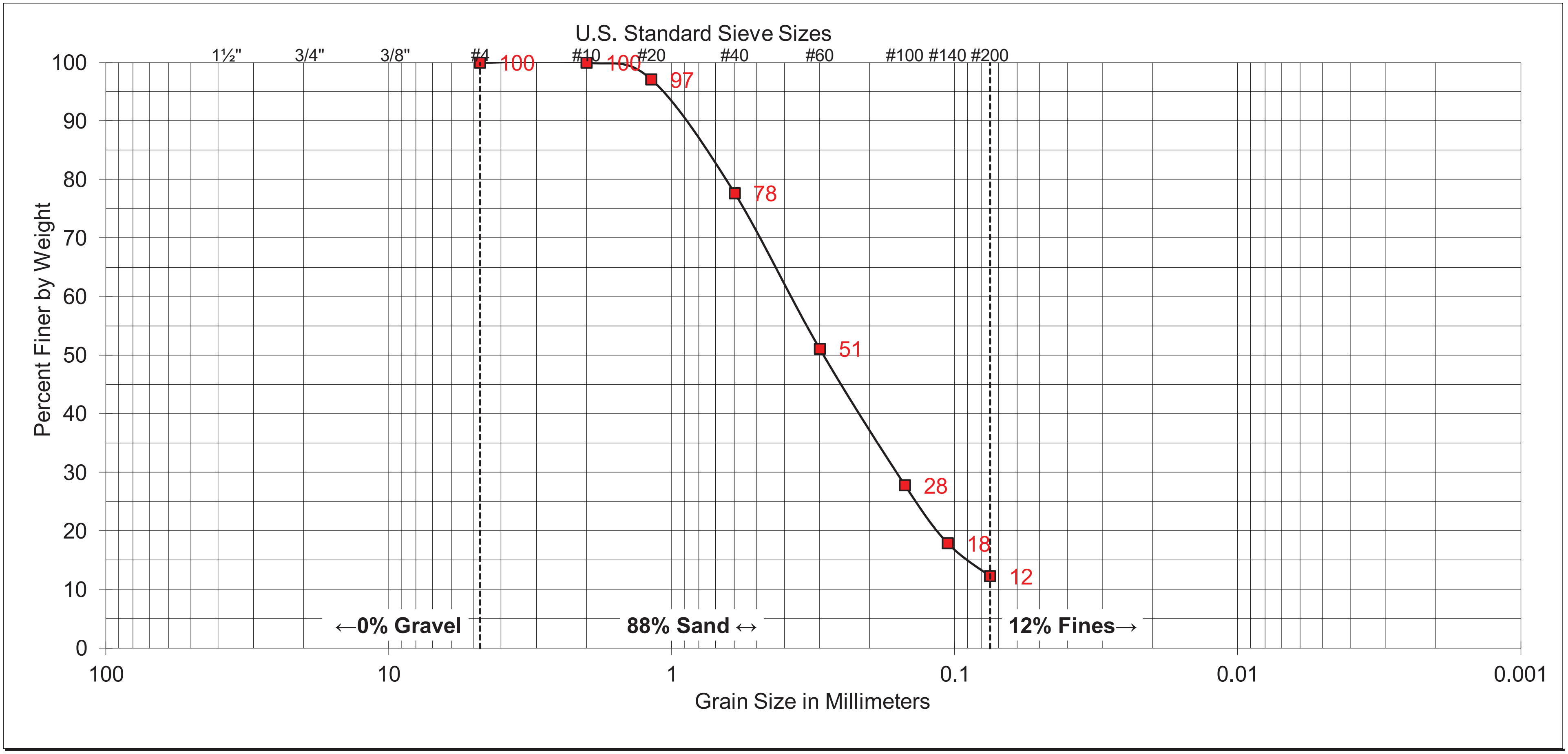
ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

FIGURE B-1.15



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION ID:	R-23-002
SAMPLE DEPTH:	81-81.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

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

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD760

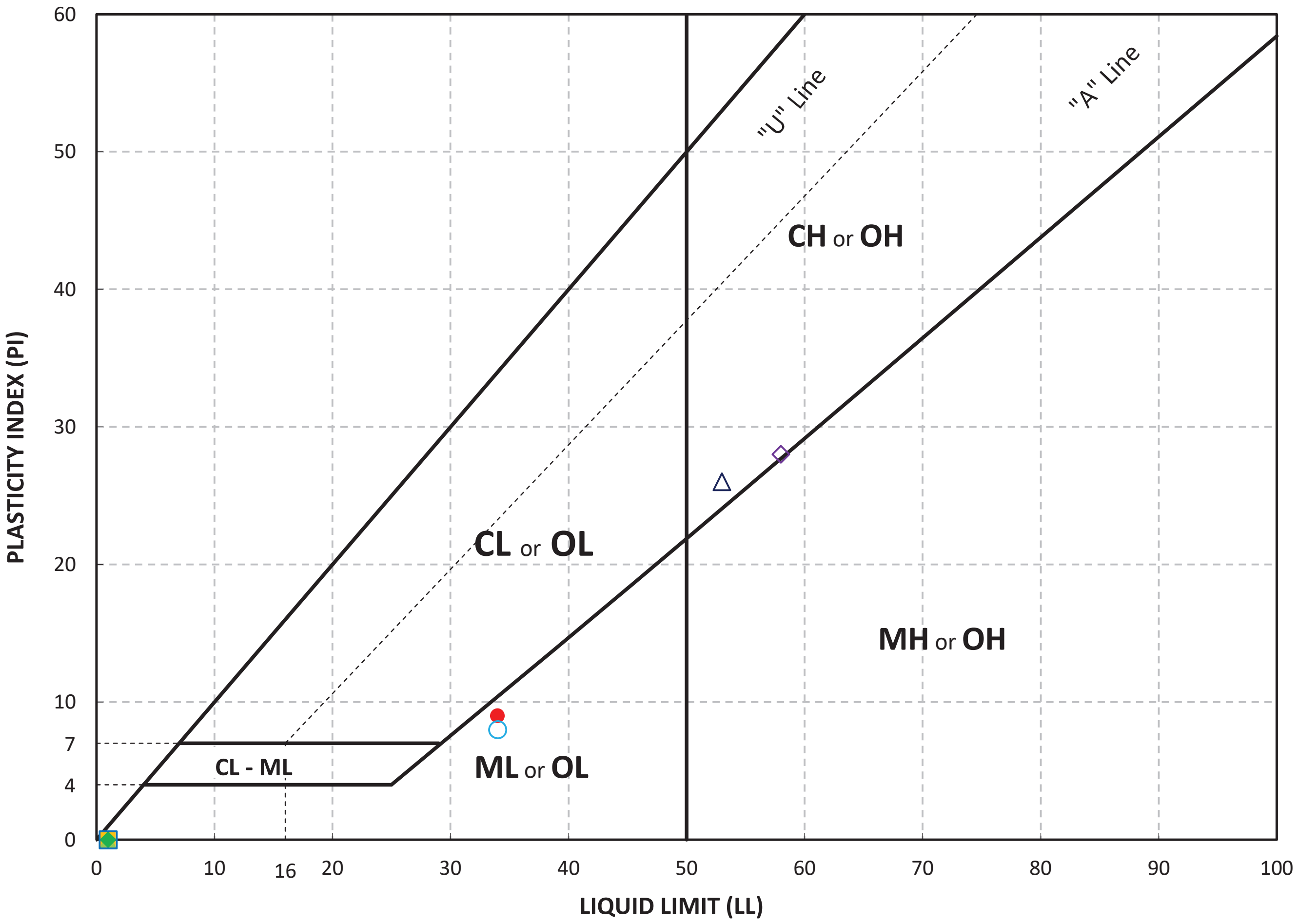
FIGURE B-1.16

PERCENT PASSING THE NO. 200 SIEVE
(ASTM D1140)

SAMPLE	DESCRIPTION	PERCENT PASSING THE NO. 200 (%)
A-23-011 @ 11’ – 11.5’	SILT with SAND (ML)	73
A-23-012 @ 21’ – 21.5’	SILTY SAND (SM)	38
A-23-013 @ 5’ – 6.5’	SILTY SAND (SM)	13
A-23-014 @ 20’ – 21.5’	Poorly Graded SAND with SILT (SP-	8
A-23-016 @ 25’ – 27.5’	SANDY SILT (ML)	59
A-23-016 @ 35’ – 35.5’	SANDY Lean CLAY (CL)	60
R-23-001 @ 15’ – 16.5’	Poorly Graded SAND with SILT (SP-	16
R-23-001 @ 41’ – 41.5’	Fat CLAY (CH)	91
R-23-001 @ 45’ – 47.5’	Fat CLAY (CH)	91
R-23-001 @ 55’ – 56.5’	SILT (ML)	87
R-23-002 @ 15’ – 16.5’	SILTY SAND (SM)	46
R-23-002 @ 35’ – 36.5’	SANDY SILT (ML)	59
R-23-002 @ 45’ – 46.5’	SILT with SAND (ML)	79
R-23-002 @ 50’ – 52.5’	SILTY SAND (SM)	44

ATTERBERG LIMITS

(ASTM D4318)



SYMBOL	BORING NO.	DEPTH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL DESCRIPTION (USCS)
●	A-23-011	11' - 11.5'	34	25	9	SILT with SAND (ML)
■	A-23-012	21' - 21.5'	NP	NP	NP	SILTY SAND (SM)
▲	A-23-012	31' - 31.5'	NP	NP	NP	Poorly Graded SAND with SILT (SP-SM)
◆	A-23-014	20' - 21.5'	NP	NP	NP	Poorly Graded SAND with SILT (SP-SM)
○	A-23-016	25' - 27.5'	34	26	8	SANDY SILT (ML)
□	R-23-001	15' - 16.5'	NP	NP	NP	Poorly Graded SAND with SILT (SP-SM)
△	R-23-001	41' - 41.5'	53	27	26	Fat CLAY (CH)
◇	R-23-001	45' - 47.5'	58	30	28	Fat CLAY (CH)

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

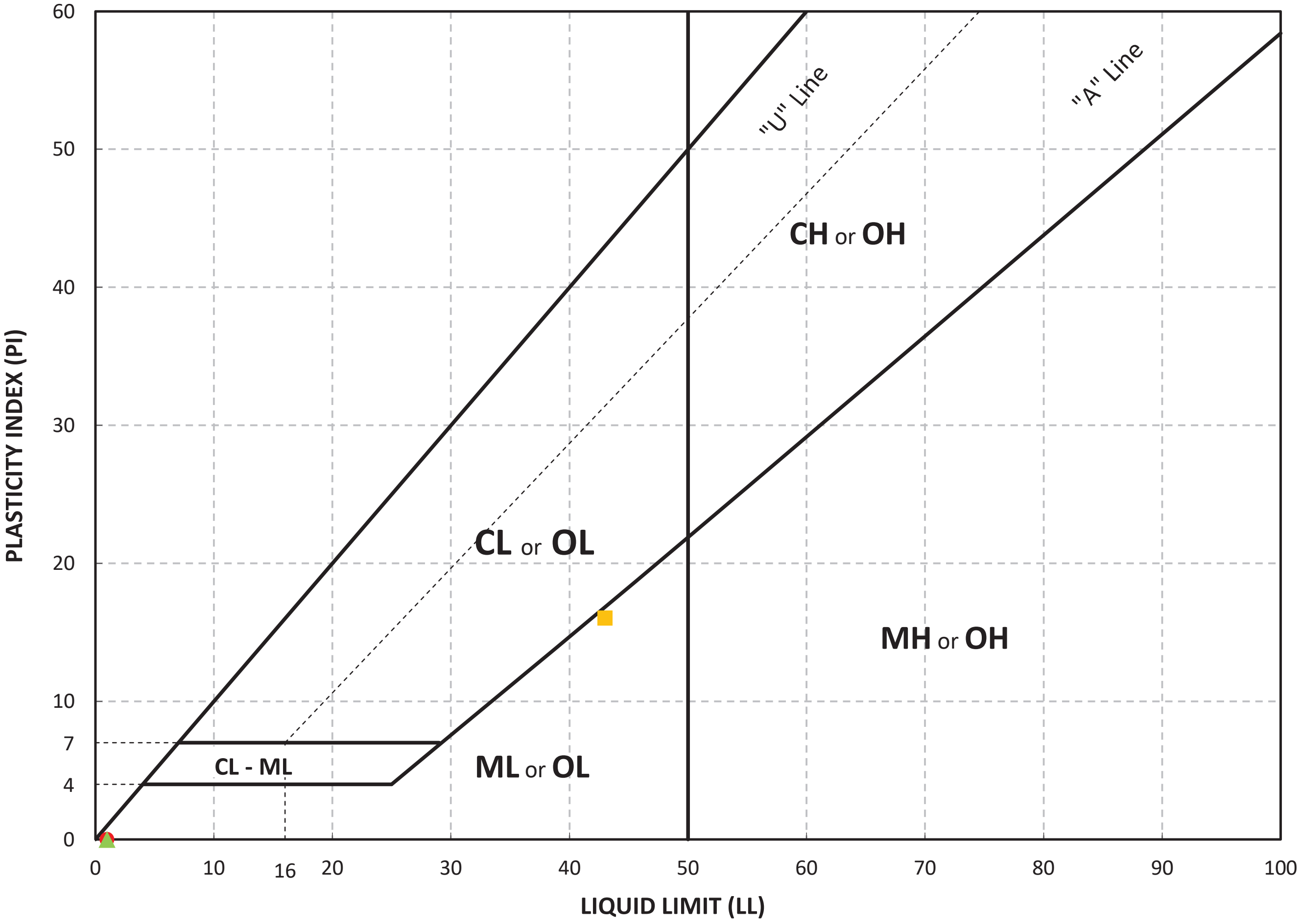
GROUP DELTA

LABORATORY TEST RESULTS

Project No. SD760
FIGURE B-2.1

ATTERBERG LIMITS

(ASTM D4318)



SYMBOL	BORING NO.	DEPTH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL DESCRIPTION (USCS)
●	R-23-002	35' - 36.5'	NP	NP	NP	SANDY SILT (ML)
■	R-23-002	45' - 46.5'	43	27	16	SILT with SAND (ML)
▲	R-23-002	50' - 52.5'	NP	NP	NP	SILTY SAND (SM)

Notes: (1) Unified Soil Classification System (USCS) per ASTM D2487

(2) NP = Non-Plastic per ASTM D4318

GROUP DELTA

LABORATORY TEST RESULTS

Project No. SD760

FIGURE B-2.2

EXPANSION TEST RESULTS
(ASTM D4829)

SAMPLE	DESCRIPTION	EXPANSION INDEX
A-23-011 @ 0.5’ – 5’	SILTY SAND (SM)	6
A-23-014 @ 0.5’ – 5’	CLAYEY SAND (SC)	13
A-23-015 @ 0.5’ – 5’	CLAYEY SAND (SC)	36

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

CORROSIVITY TEST RESULTS

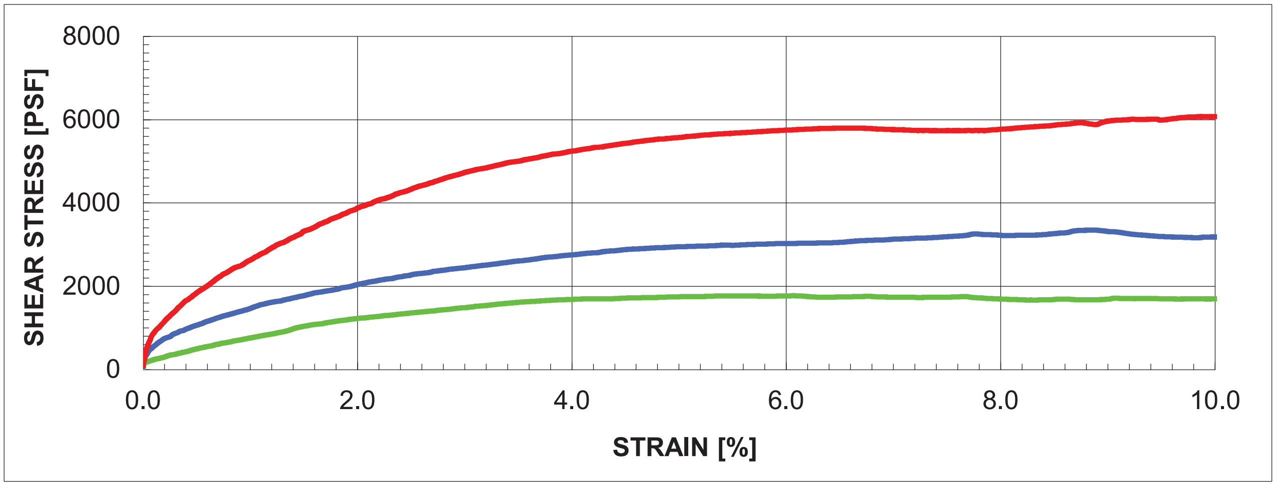
(ASTM D516, CTM 643)

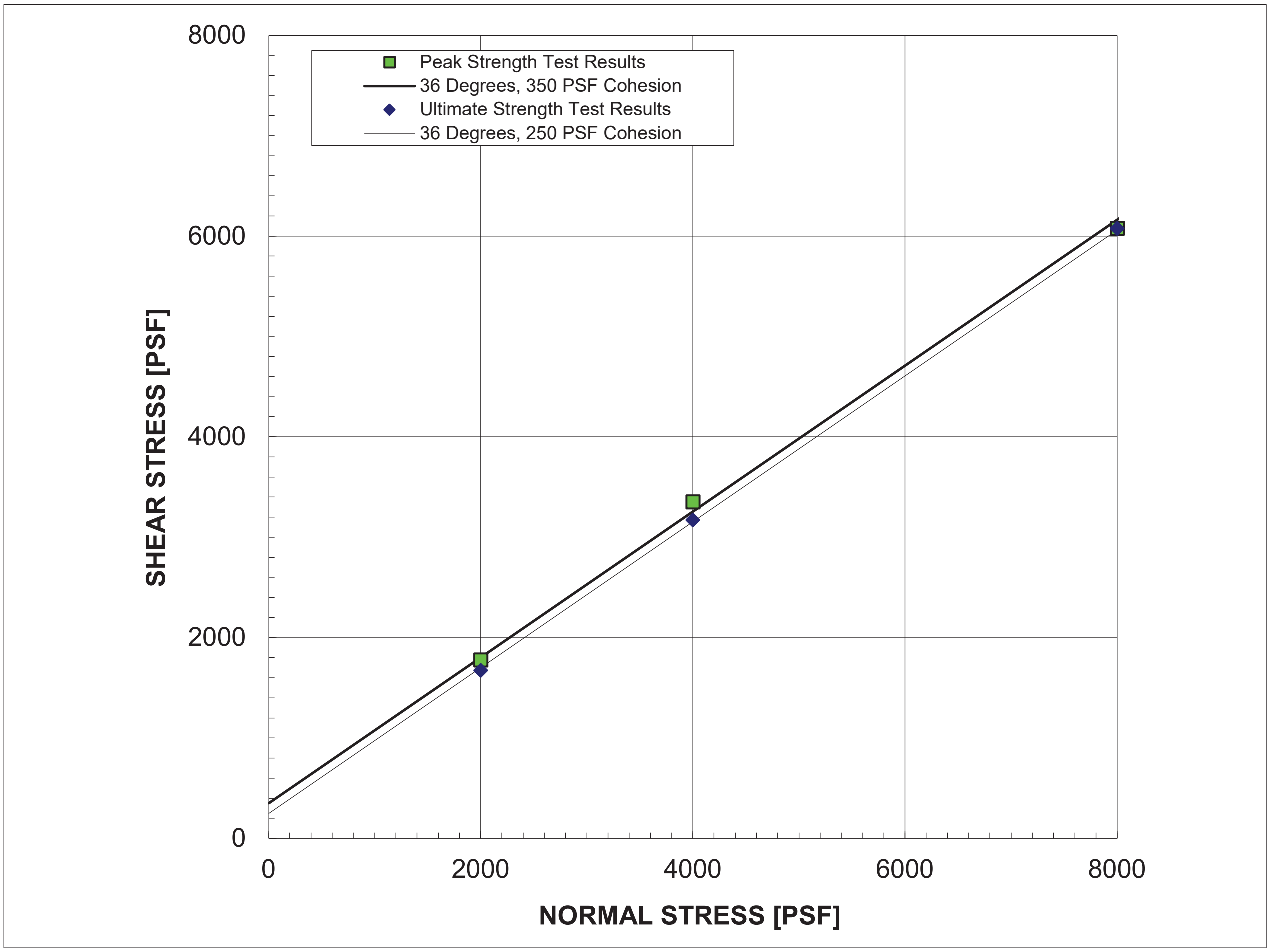
SAMPLE	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
A-23-011 @ 0.5' – 5'	8.15	7,962	<0.01	<0.01
A-23-015 @ 0.5' – 5'	8.33	1,387	0.01	<0.01
R-23-002 @ 21' – 21.5'	8.08	698	0.05	0.06

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





SAMPLE: R-23-001 @ 61' - 61.5'

Description:
SILT with SAND (ML)

STRAIN RATE: 0.0007 IN/MIN

(Sample was consolidated and drained)

	PEAK	**ULTIMATE**
ϕ'	36 °	36 °
C'	350 PSF	250 PSF

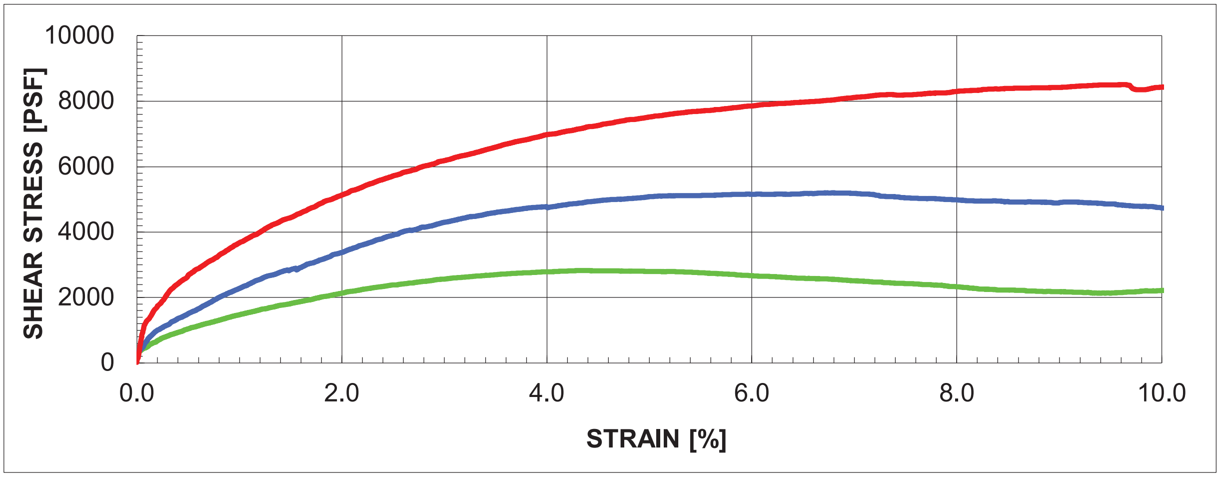
	IN-SITU	**AS-TESTED**
γ_d	86.7 PCF	86.7 PCF
w_c	36.2 %	35.0 %

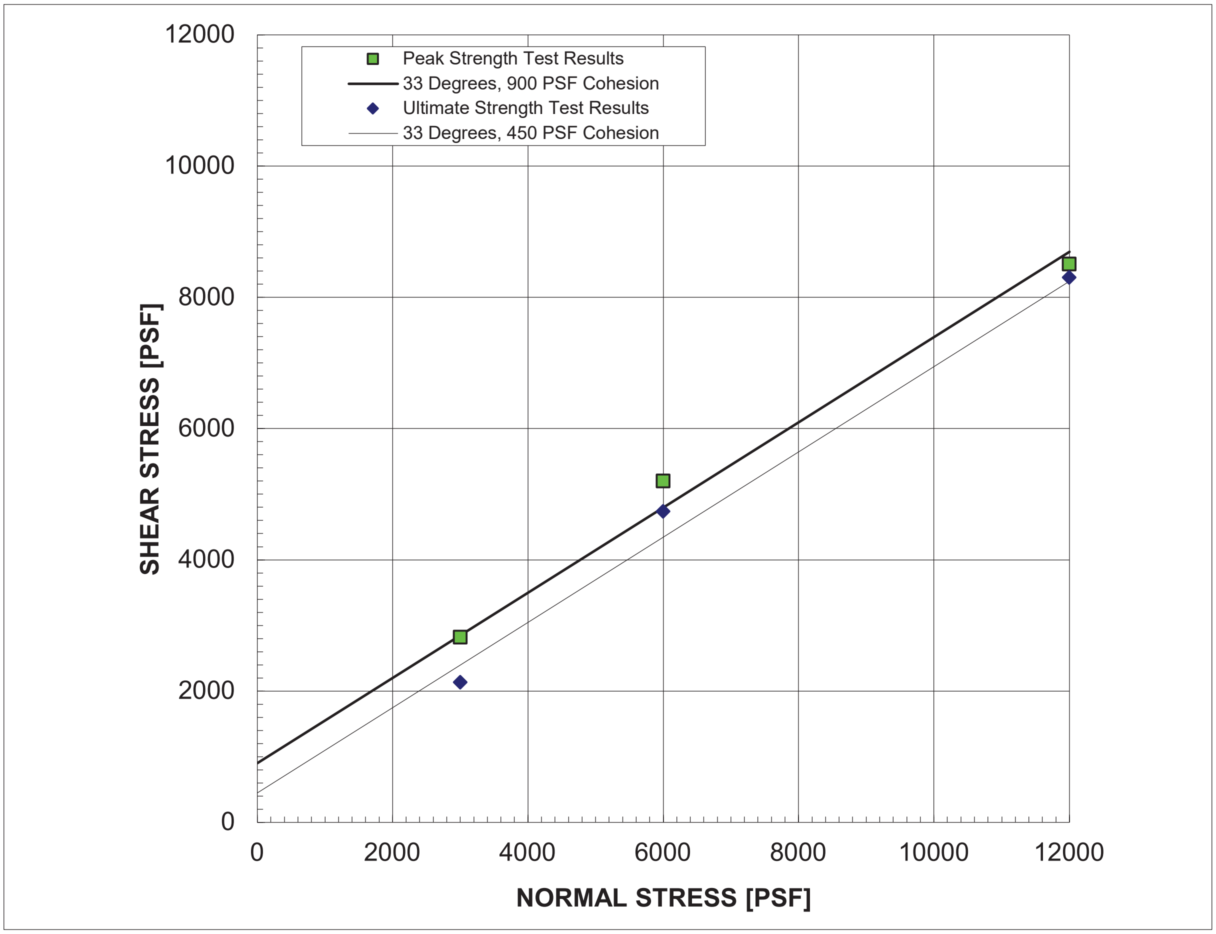
GROUP DELTA

DIRECT SHEAR TEST RESULTS

Project No. SD760

FIGURE B-5.1





SAMPLE: R-23-001 @ 91' - 91.5'

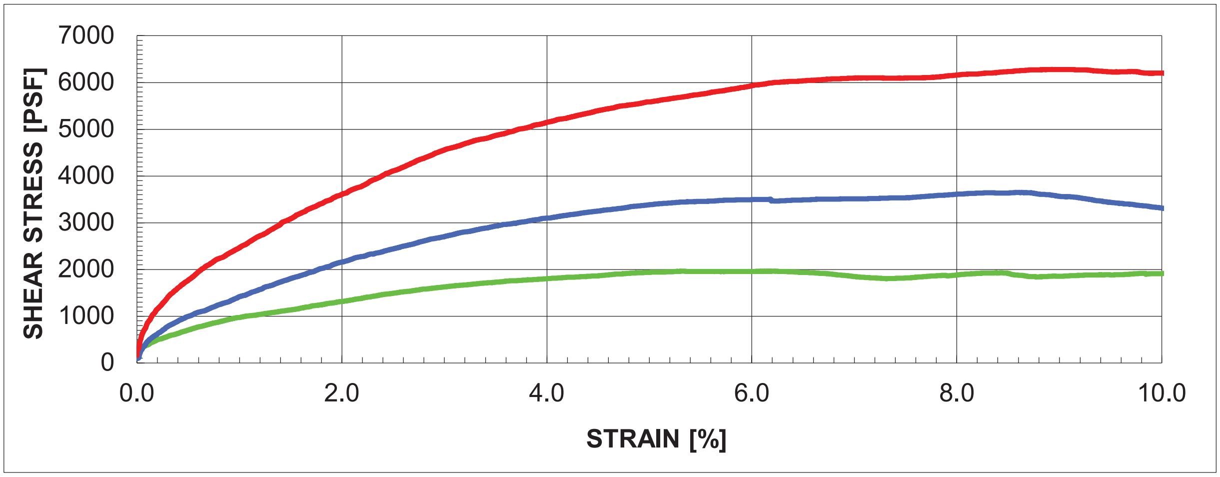
Description:
Poorly graded SAND with SILT (SP-SM)

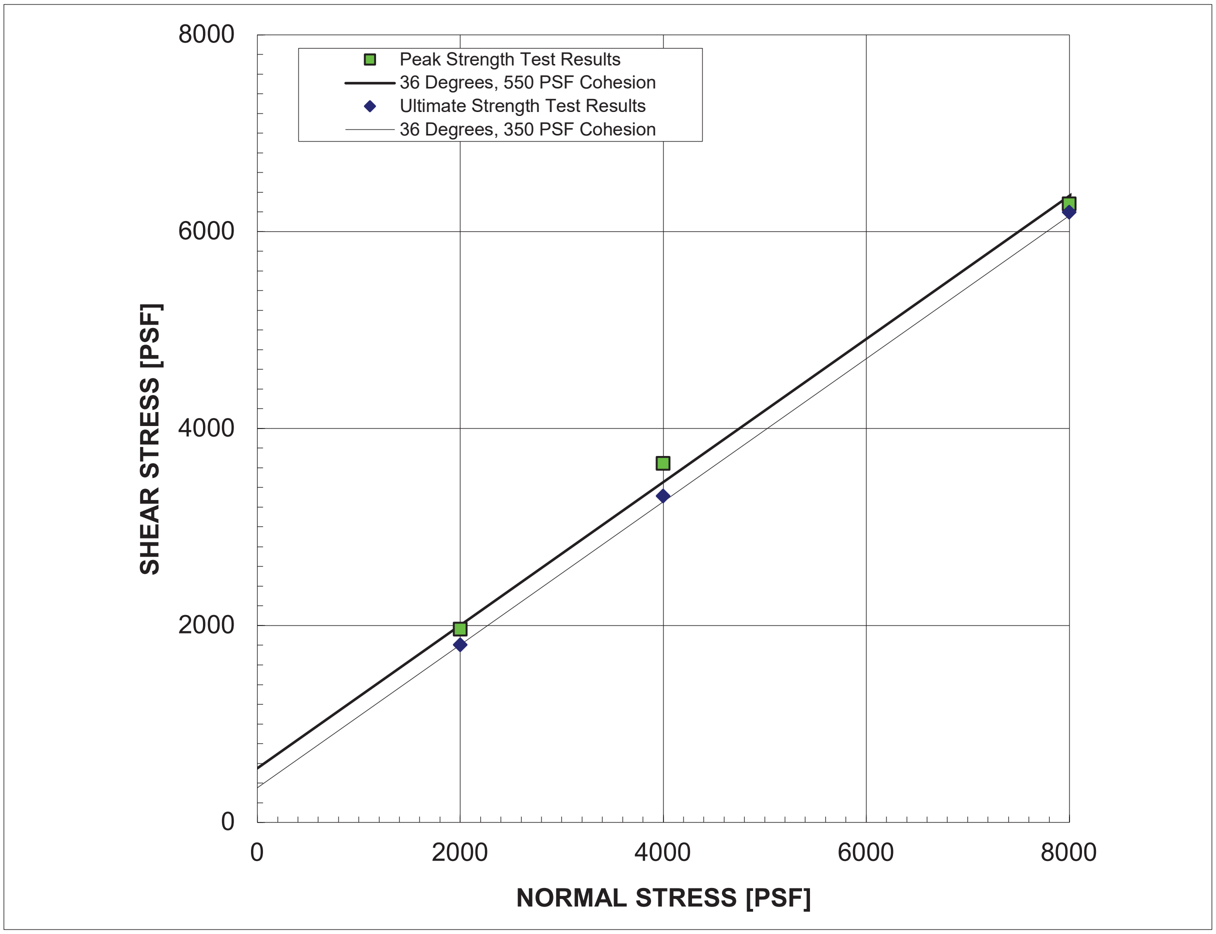
	PEAK	ULTIMATE
ϕ'	33 °	33 °
C'	900 PSF	450 PSF

STRAIN RATE: 0.0020 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	96.3 PCF	96.3 PCF
w_c	28.5 %	26.6 %

GROUP DELTA **DIRECT SHEAR TEST RESULTS**





SAMPLE: R-23-002 @ 66' - 66.5'

Description:
SANDY SILT (ML)

	PEAK	ULTIMATE
ϕ'	36 °	36 °
C'	550 PSF	350 PSF

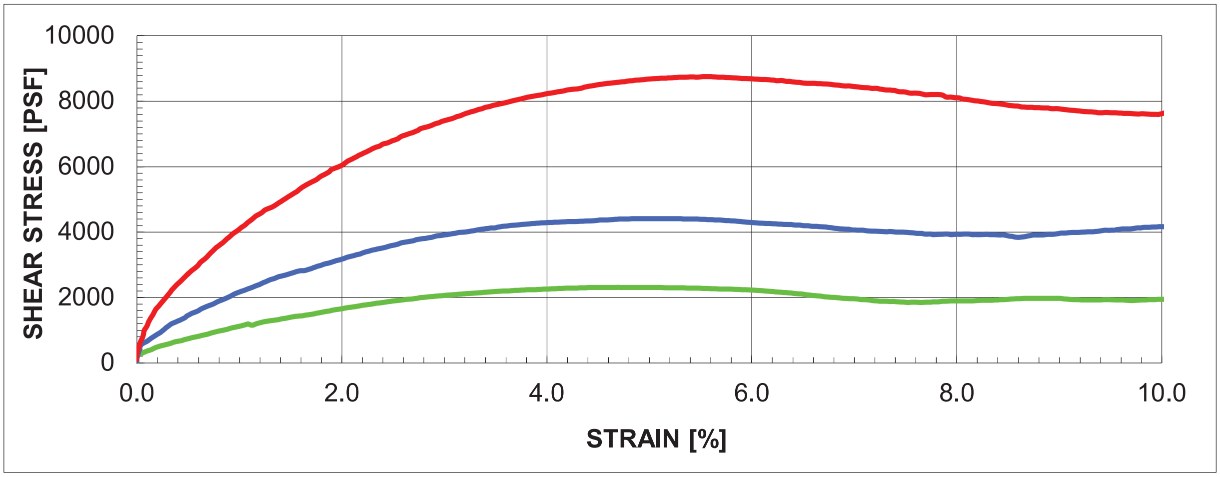
STRAIN RATE: 0.0008 IN/MIN
(Sample was consolidated and drained)

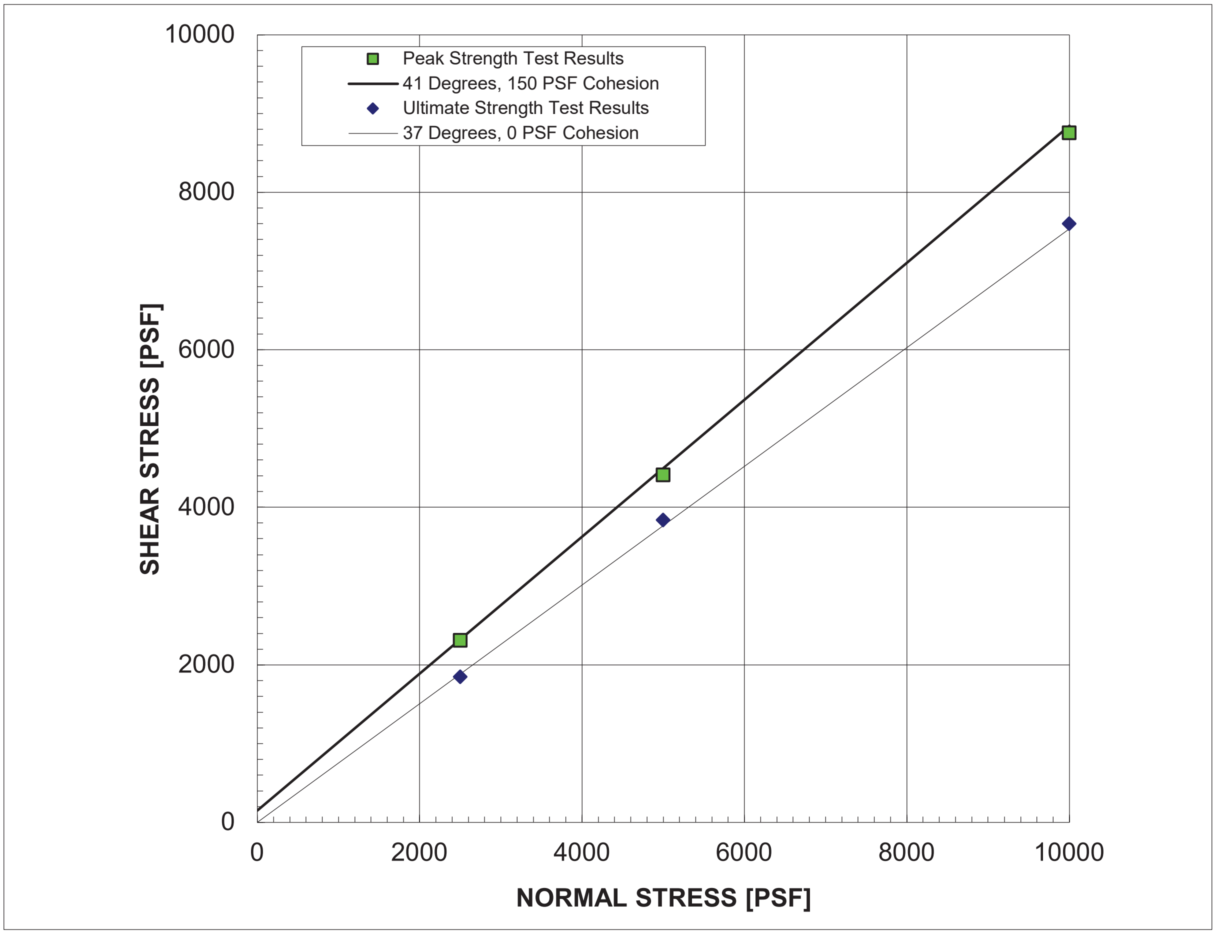
	IN-SITU	AS-TESTED
γ_d	87.5 PCF	87.5 PCF
w_c	38.5 %	29.0 %

GROUP DELTA

DIRECT SHEAR TEST RESULTS

Project No. SD760
FIGURE B-5.3





SAMPLE: R-23-002 @ 81' - 81.5'

Description:
SILTY SAND (SM)

	PEAK	ULTIMATE
φ'	41 °	37 °
C'	150 PSF	0 PSF
	IN-SITU	**AS-TESTED**
γ_d	102.8 PCF	102.8 PCF
w_c	21.3 %	23.7 %

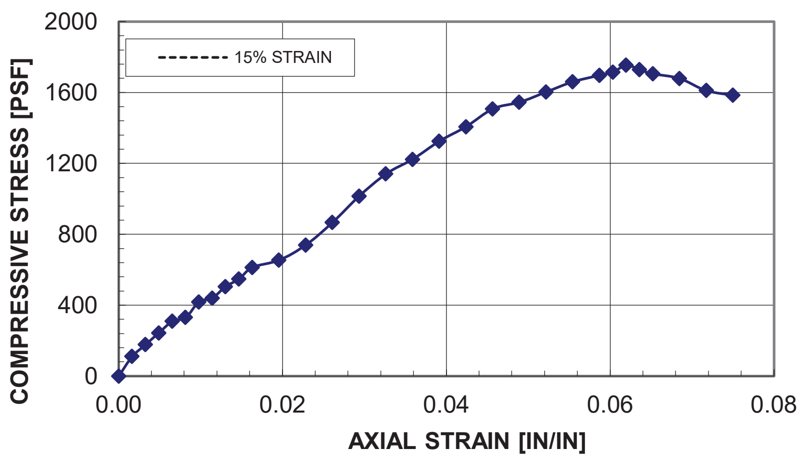
STRAIN RATE: 0.0040 IN/MIN
(Sample was consolidated and drained)

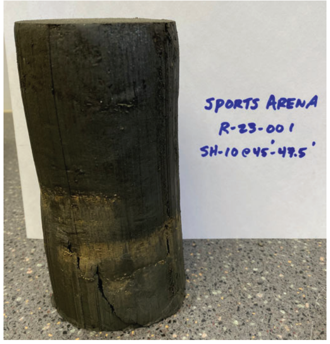
GROUP DELTA **DIRECT SHEAR TEST RESULTS**

Project No. SD760
FIGURE B-5.4

PROJECT: **Zephyr Sports Arena**
SAMPLE I.D.: **R-23-001 @ 45' - 47.5'**
DESCRIPTION: **Fat CLAY (CH)**
TEST METHOD: **ASTM D2166**
TESTED BY: **J. Krehbiel**
DATE: **2/27/23**

TYPE OF SAMPLE	Shelby Tube	
WET WT. OF SAMPLE	1080.96	[g]
INITIAL DIAM.	2.87	[in]
INITIAL HEIGHT	6.135	[in]
INITIAL AREA	6.469	[in^2]
INITIAL VOLUME	39.69	[in^3]
WET DENSITY	103.8	[pcf]
DRY WT. OF SAMPLE	713.82	[g]
WEIGHT OF WATER	367.1	[g]
INITIAL TOTAL MOISTURE	51.4	[%]
DRY DENSITY	68.5	[pcf]
L-D RATIO	2.1:1	
STRAIN RATE	1.66	[%/min]
STRAIN AT FAILURE	10.11	[%]
STRAIN AT FAILURE	0.620	[in]
15% STRAIN	0.920	[in]
FAILURE CRITERIA:	Yield	
COMP. STRENGTH:	1754	[psf]
SHEAR STRENGTH:	877	[psf]
SPEC. GRAVITY (Assumed)	2.85	
SATURATION:	92	[%]
FAILURE MODE:	semi-plastic	




SPORTS ARENA
R-23-001
SH-10@45'-47.5'

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	6.47	0.0
0.2	5.0	0.990	0.010	0.002	6.48	111.1
0.3	8.0	0.980	0.020	0.003	6.49	177.5
0.7	14.0	0.960	0.040	0.007	6.51	309.6
0.8	15.0	0.950	0.050	0.008	6.52	331.2
1.0	19.0	0.940	0.060	0.010	6.53	418.8
1.4	25.0	0.910	0.090	0.015	6.57	548.3
1.6	28.0	0.900	0.100	0.016	6.58	613.1
1.7	30.0	0.880	0.120	0.020	6.60	654.7
2.0	34.0	0.860	0.140	0.023	6.62	739.5
2.2	40.0	0.840	0.160	0.026	6.64	867.1
2.6	47.0	0.820	0.180	0.029	6.66	1015.5
2.9	53.0	0.800	0.200	0.033	6.69	1141.3
3.2	57.0	0.780	0.220	0.036	6.71	1223.3
3.5	62.0	0.760	0.240	0.039	6.73	1326.1
3.8	66.0	0.740	0.260	0.042	6.76	1406.8
4.2	71.0	0.720	0.280	0.046	6.78	1508.3
4.5	73.0	0.700	0.300	0.049	6.80	1545.5
4.9	76.0	0.680	0.320	0.052	6.83	1603.5
5.2	79.0	0.660	0.340	0.055	6.85	1661.0

GROUP DELTA

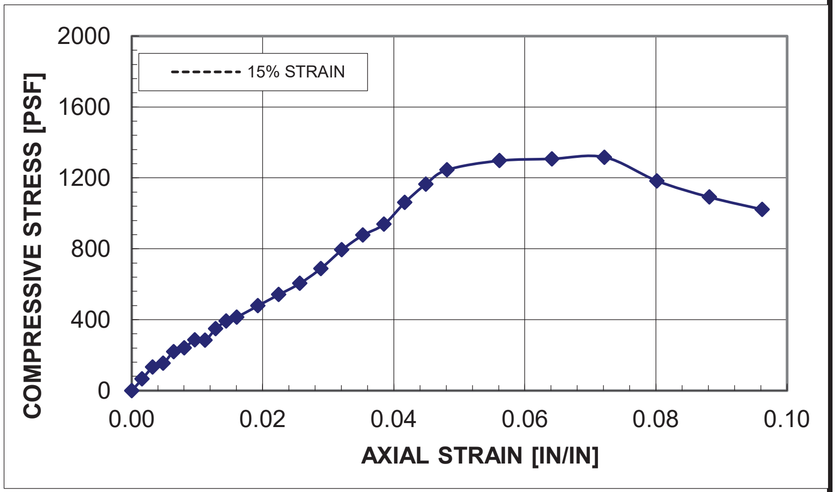
**UNCONFINED
COMPRESSIVE
STRENGTH**

Project No. SD760
FIGURE B-6.1

PROJECT: **Zephyr Sports Arena**
SAMPLE I.D.: **R-23-002 @ 50' - 52.5'**
DESCRIPTION: **SILTY SAND (SM)**

TEST METHOD: **ASTM D2166**
TESTED BY: **J. Krehbiel**
DATE: **2/27/23**

TYPE OF SAMPLE	Shelby Tube	
WET WT. OF SAMPLE	1195	[g]
INITIAL DIAM.	2.875	[in]
INITIAL HEIGHT	6.238	[in]
INITIAL AREA	6.492	[in^2]
INITIAL VOLUME	40.50	[in^3]
WET DENSITY	112.4	[pcf]
DRY WT. OF SAMPLE	878.58	[g]
WEIGHT OF WATER	316.4	[g]
INITIAL TOTAL MOISTURE	36.0	[%]
DRY DENSITY	82.7	[pcf]
L-D RATIO	2.2:1	
STRAIN RATE	1.62	[%/min]
STRAIN AT FAILURE	8.82	[%]
STRAIN AT FAILURE	0.550	[in]
15% STRAIN	0.936	[in]
FAILURE CRITERIA:	Yield	
COMP. STRENGTH:	1317	[psf]
SHEAR STRENGTH:	659	[psf]
SPEC. GRAVITY (Assumed)	2.85	
SATURATION:	89	[%]
FAILURE MODE:	semi-plastic	



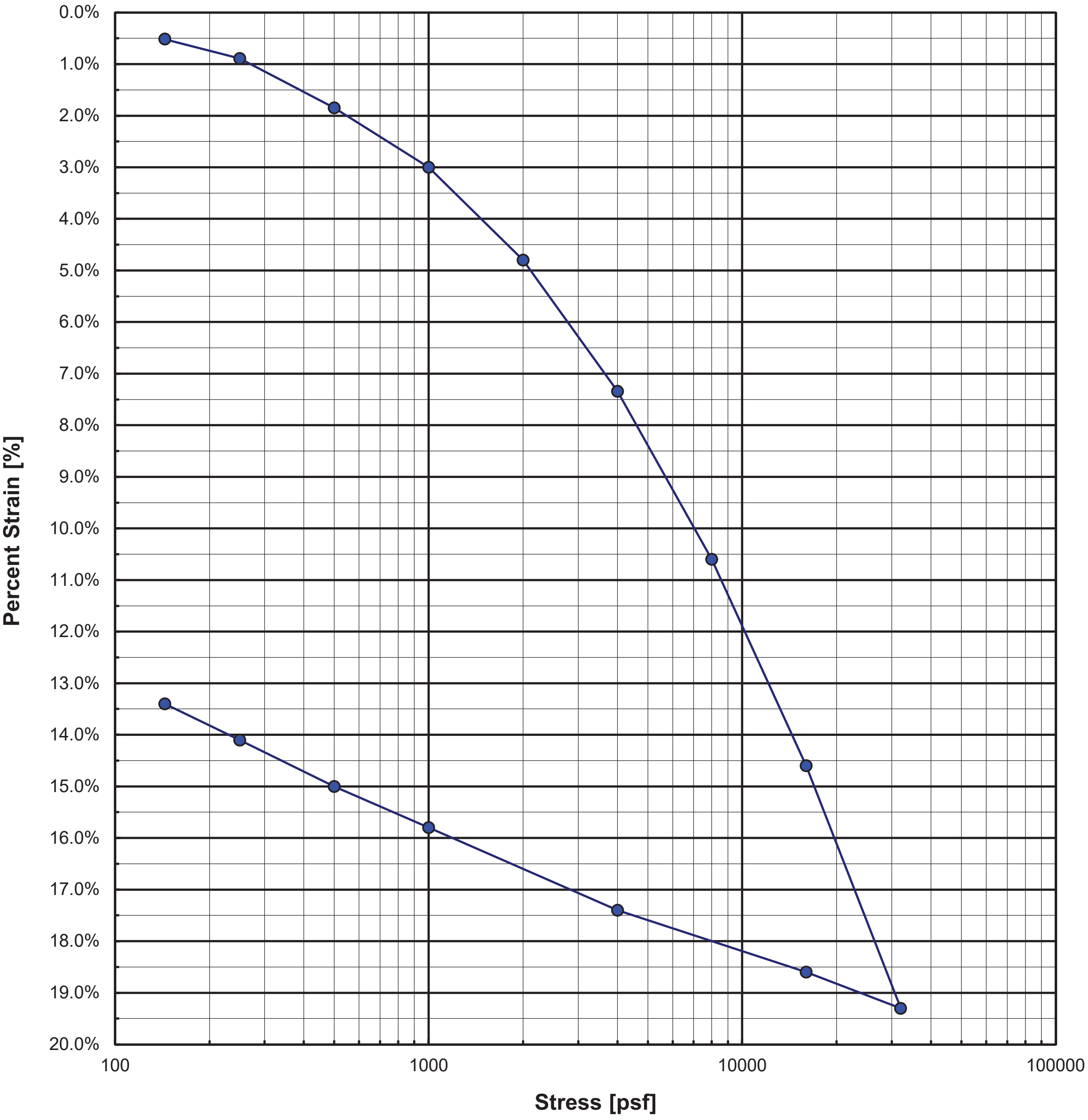
SPECIMEN AFTER FAILURE

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	6.49	0.0
0.1	3.0	0.990	0.010	0.002	6.50	66.4
0.3	6.0	0.980	0.020	0.003	6.51	132.7
0.7	10.0	0.960	0.040	0.006	6.53	220.4
0.8	11.0	0.950	0.050	0.008	6.54	242.0
1.0	13.0	0.940	0.060	0.010	6.55	285.6
1.5	18.0	0.910	0.090	0.014	6.59	393.5
1.7	19.0	0.900	0.100	0.016	6.60	414.7
1.8	22.0	0.880	0.120	0.019	6.62	478.6
2.0	25.0	0.860	0.140	0.022	6.64	542.1
2.3	28.0	0.840	0.160	0.026	6.66	605.2
2.6	32.0	0.820	0.180	0.029	6.68	689.3
2.9	37.0	0.800	0.200	0.032	6.71	794.4
3.2	41.0	0.780	0.220	0.035	6.73	877.4
3.5	44.0	0.760	0.240	0.038	6.75	938.4
2.9	50.0	0.740	0.260	0.042	6.77	1062.9
4.3	55.0	0.720	0.280	0.045	6.80	1165.2
4.6	59.0	0.700	0.300	0.048	6.82	1245.8
4.9	62.0	0.650	0.350	0.056	6.88	1298.1
5.2	63.0	0.600	0.400	0.064	6.94	1307.8

GROUP DELTA

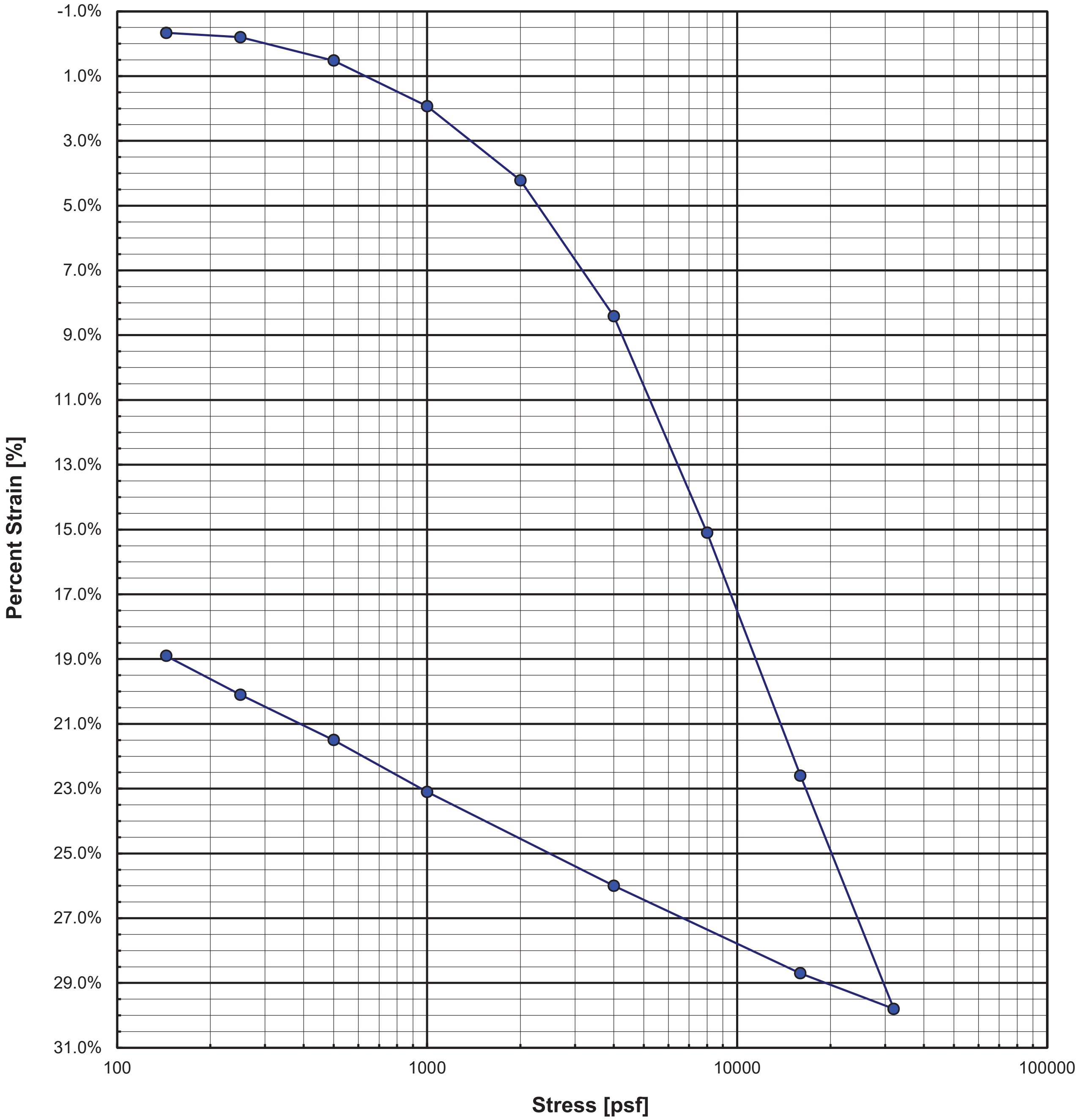
UNCONFINED COMPRESSIVE STRENGTH

Project No. SD760
FIGURE B-6.2



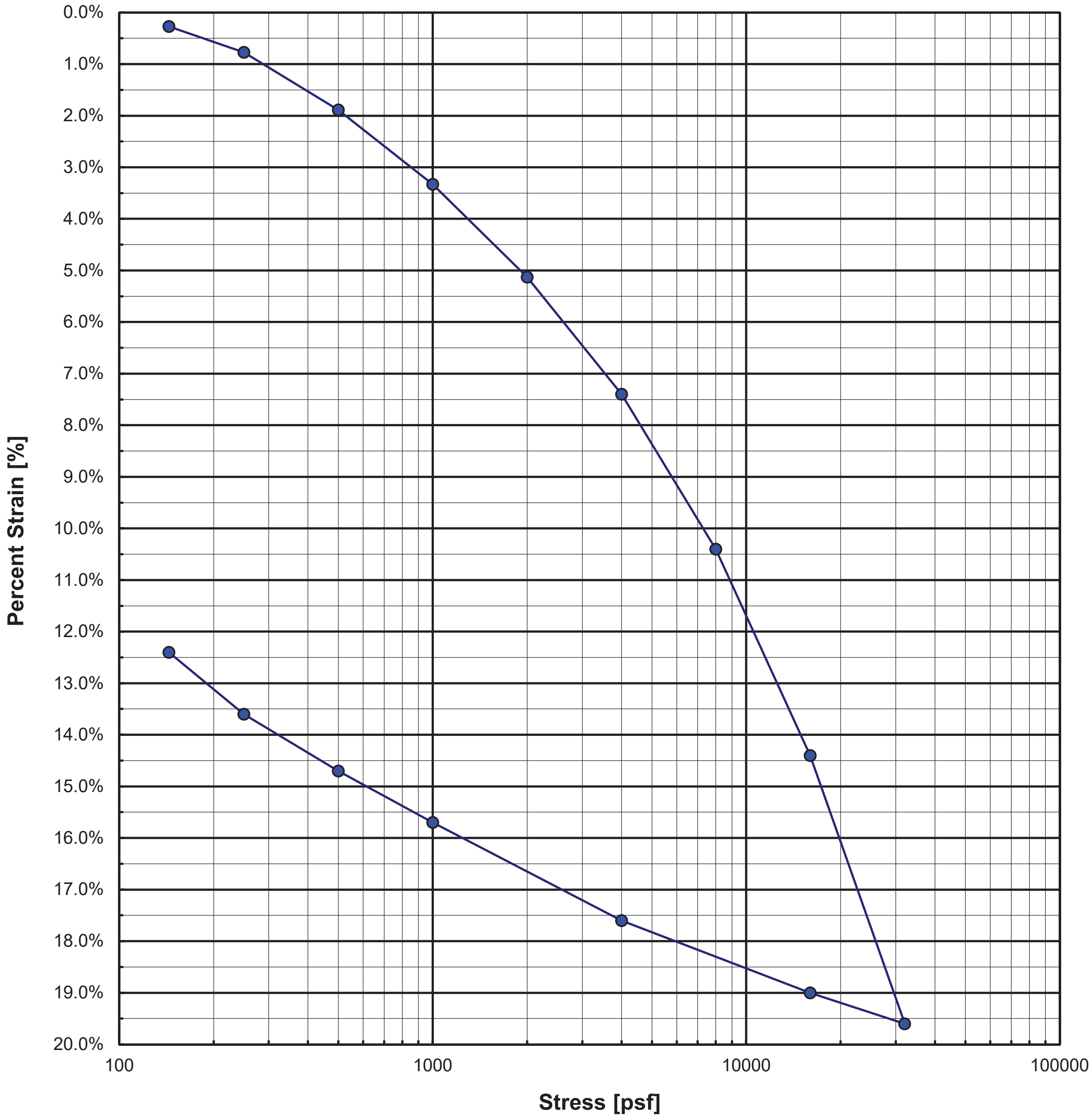
SAMPLE ID: A-23-016 @ 25' - 27.5' **DESCRIPTION:** SANDY SILT (ML)

INITIAL	FINAL	
1.0000	0.8660	SAMPLE HEIGHT [IN]
89.8	103.7	DRY DENSITY [PCF]
2.88	2.88	SPECIFIC GRAVITY (ASSUMED)
0.99	0.73	VOID RATIO (e)
33.3	25.5	WATER CONTENT [%]
96.6	100.0	DEGREE OF SATURATION [%]



SAMPLE ID: R-23-001 @ 45' - 47.5' **DESCRIPTION:** Fat CLAY (CH)

INITIAL	FINAL	
1.0000	0.8110	SAMPLE HEIGHT [IN]
65.5	80.8	DRY DENSITY [PCF]
2.86	2.86	SPECIFIC GRAVITY (ASSUMED)
1.73	1.21	VOID RATIO (e)
60.5	42.3	WATER CONTENT [%]
99.8	100.0	DEGREE OF SATURATION [%]



SAMPLE ID: R-23-002 @ 50' - 52.5' **DESCRIPTION:** SILTY SAND (SM)

INITIAL	FINAL	
1.0000	0.8760	SAMPLE HEIGHT [IN]
77.3	88.3	DRY DENSITY [PCF]
2.85	2.85	SPECIFIC GRAVITY (ASSUMED)
1.30	1.02	VOID RATIO (e)
38.7	35.7	WATER CONTENT [%]
85.1	100.0	DEGREE OF SATURATION [%]

GROUP DELTA

CONSOLIDATION RESULTS

Project No. SD760

FIGURE B-7.3

APPENDIX C
GEOTECHNICAL ANALYSES

APPENDIX C

GEOTECHNICAL ANALYSES

SOIL PARAMETERS

Several soil parameters were interpreted from our field in-situ testing and laboratory test results. These parameters were used in the following calculations that are discussed in the later portions of this appendix. The presence of mica, organics, and/or seashells can influence the geotechnical engineering characteristics of the fill and upper paralic estuarine deposits.

Hammer Energy-Corrected Blow Count (N_{60})

The Hammer Energy-Corrected Standard Penetration Test (SPT) Blow Count (N_{60}) was interpreted from our driven samples collected from the geotechnical borings and from the Cone Penetration Test (CPT) soundings. In the geotechnical borings, N_{60} was estimated using the methods described in Appendix A. In the CPT soundings, the N_{60} was estimated using a correlation included in the referenced publication (Robertson et al., 2012). Figure C-1.1 below provides a plot of the interpreted N_{60} versus elevation.

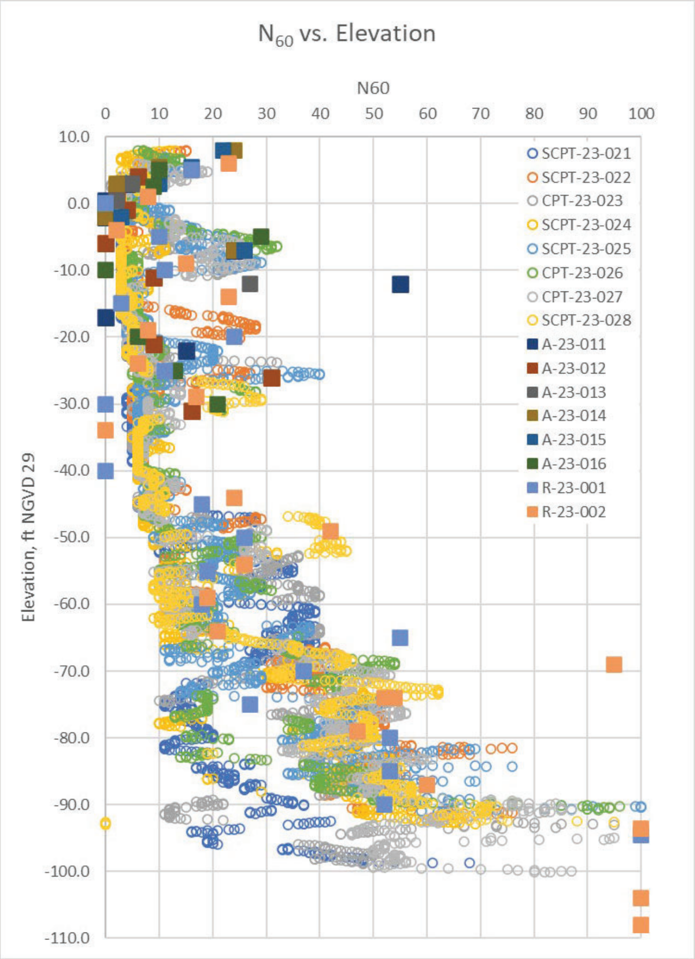


Figure C-1.1 – N_{60} versus Elevation

Effective Angle of Internal Friction (ϕ')

The effective angle of internal friction (ϕ'), or commonly known as friction angle, was measured in the laboratory by performing Direct Shear (DS) tests on partially intact samples collected from the geotechnical borings, as shown in Appendix B. It was also interpreted from our driven samples collected from the geotechnical borings and the Cone Penetration Test (CPT) soundings. In the geotechnical borings, ϕ' was estimated using a correlation to SPT blow count (AASHTO, 2012). In the CPT soundings, the ϕ' was estimated using a correlation included in the referenced publication (Robertson et al., 2012). Figure C-1.2 below provides a plot of the friction angle versus elevation.

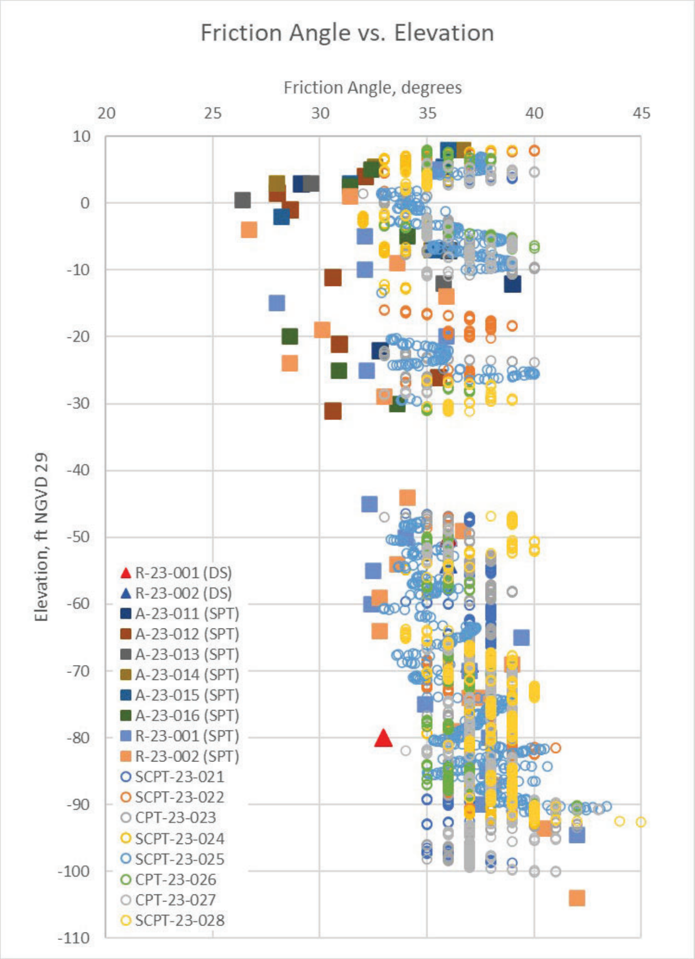


Figure C-1.2 – Friction Angle versus Elevation

Undrained Shear Strength (S_u)

The undrained shear strength (S_u) was measured in the laboratory by performing Unconfined Compressive (UC) strength tests on relatively undisturbed samples collected from the geotechnical borings, as shown in Appendix B. It was also interpreted from the Cone Penetration Test (CPT) soundings using a correlation included in the referenced publication and the computer program CPeT-IT (GeoLogismiki, 2023b; Robertson et al., 2012). Figure C-3 below provides a plot of the undrained shear strength versus elevation.

Figure C-1.3 – Undrained Shear Strength versus Elevation

LIQUEFACTION

The computer program CLiq (GeoLogismiki, 2023a) was used to perform liquefaction triggering calculations using several CPT-based methods, including those recommended by the NCEER Workshops (Robertson et al., 1997; 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):	0.74g
Earthquake Magnitude (Mw):	6.9
Design Groundwater Level:	+3 feet NGVD 29

The PGA_M was evaluated using the 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 analyses preliminarily adopt a Site Class D to evaluate the PGA used for liquefaction triggering. This may need to be reviewed and updated following the completion of a ground response study for the site. The controlling magnitude used in the liquefaction evaluation was selected by reviewing deaggregation results obtained from the USGS Unified Hazard Tool (USGS, 2023). The groundwater level was adopted as recommended in the *Design Groundwater Elevation* section of this report.

The analyses were performed using data collected from the CPT soundings performed at the site. The correlated CPT parameters were compared to the results of our field and laboratory testing collected from the geotechnical borings. The Soil Behavior Type (SBT) correlated from the CPT data was adjusted to best fit the observations, classifications, and material properties of the soils observed within the borings.

In accordance with Special Publication 117A (CGS, 2008) and general geotechnical engineering practices, the liquefaction analyses were limited to a depth of 60 feet to incorporate the potentially liquefiable layers that extend to depths of approximately 60 feet.

The liquefaction settlement analyses include depth weighting proposed by Cetin et al. (2009), which consists of a linear 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).

GEOTECHNICAL ANALYSES (Continued)

Our assessment of the potential for liquefaction triggering and estimate of the liquefaction-induced settlement interprets the following:

- Potentially liquefiable soils occur at the design groundwater table (+3 feet NGVD 29) and extends to about 60 feet below existing grades (-50 feet NGVD 29). The liquefiable soils are predominantly silty sand (USCS Symbol SM), sand (SP-SM), and non-plastic sandy silts (ML). In the upper 40 feet below existing grades (-30 feet NGVD 29), liquefiable materials generally occur as a thick, continuous layer that is occasionally interrupted by thin layers of non-liquefiable materials less than about three feet in thickness. Below a depth of 40 feet, liquefiable materials occur in relatively thin layers (about 5-foot thick or less) that are separated by non-liquefiable materials that range from about two to ten feet in thickness.
- Estimated settlements range from 7.5 to 10 inches in our calculations. Differential settlement over the common 30- to 40-foot column spacing is typically estimated to be one-half to two-thirds of the total settlement. Actual settlements realized in the field following a seismic event can vary significantly from calculations. Accordingly, design total and differential liquefaction induced settlements are also provided in the table below to account for the potential variability of actual liquefaction induced settlements compared to those that were calculated as a part of this evaluation.

ESTIMATED LIQUEFACTION-INDUCED SETTLEMENT

Exploration	Calculated Total Settlement [1,2] (Inches)	Calculated Differential Settlement [3] (Inches)	Design Total Settlement [1,2] (Inches)	Design Differential Settlement [3] (Inches)
SCPT-23-021	7.5	5	5.5 – 9.5	4 – 6.5
SCPT-23-022	10	6.5	7.5 – 12.5	5 – 8.5
CPT-23-023	8.5	5.5	6.5 – 10.5	4.5 – 7
SCPT-23-024	10	6.5	7.5 – 12.5	5 – 8.5
SCPT-23-025	8	5	6 – 10	4 – 6.5
CPT-23-026	8	5	6 – 10	4 – 6.5
CPT-23-027	7	4.5	5.5 – 9	3.5 – 6
SCPT-23-028	9	6	7 – 11.5	4.5 – 7.5

[1] Settlement is the combination of liquefaction-induced and seismic compaction. Estimated magnitude of seismic compaction insignificant.
[2] Settlement is a “free-field” estimate that does not consider: a) the shear strain due to foundation loading, and b) contribution of ejecta-related settlement.
[3] Differential settlement is measured over a common 30- to 40-foot column spacing.

STATIC SETTLEMENT

Compressible soils underlie the site. Most of these soils are sands, silty sands, and non-plastic sandy silts that should settle elastically with the initial fill and structure loading (i.e., short-term settlement). However, there are local zones of thick fat clay and plastic silt that should experience some time dependent consolidation settlement (i.e., long-term settlement). The fat clay has a high plasticity and we interpret it to be medium stiff and normally consolidated from consolidation test, unconfined compression test, in-situ moisture contents, and Plasticity Index data. The plastic silt has medium plasticity and we interpret it to me medium stiff to stiff and slightly over consolidated from consolidation test, unconfined compression test, in-situ moisture contents, and Plasticity Index data. The in-situ moisture contents are near or are above the Liquid Limit and the Liquidity Indices range from 0.7 to 2.0, which indicate relatively soft and high compressibility soils. The total static settlement estimated at each exploration location is the sum of the long-term and short-term settlements.

Settlement analyses were conducted using the soil profiles and groundwater conditions encountered in the recent explorations and laboratory test data. The settlement magnitude and areal distribution were estimated with conventional elastic and consolidation soil mechanics methods. SPT and CPT correlations to elastic modulus were used to evaluate compressibility parameters for granular soils and non-plastic silts, and consolidation test results were used to evaluate consolidation parameters in clay and plastic silts. The analyses utilize the Boussinesq method for estimating the loading stress attenuation with depth. Settlement is neglected below the depth where the loading stress is less than 10 percent of the in-situ effective stress. The settlement parameters evaluated in these analyses do not consider increases in stiffness due to ground improvement or remedial grading and are therefore conservative in nature.

Most of the long-term settlement should occur in a relatively short time following initial loading. The zones of clay and plastic silt are usually surrounded by sand or silty sand, which should allow horizontal drainage to more quickly dissipate the excess porewater pressures that develop from loading. Estimated durations for substantial completion were not provided for the CPT locations because it is not part of the method. However, based on the interpreted thicknesses of the fine-grained layers within the CPT soundings, the settlement durations should be similar to those evaluated for the boring locations (R-23-001 and R-23-002).

The following table below provides the estimated short-term, long-term, and total static settlement and the durations of the long-term settlement assuming that a new fill thickness of three feet over a 250- by 250-foot area is placed in the vicinity of the exploration.

GEOTECHNICAL ANALYSES (Continued)

ESTIMATED STATIC SETTLEMENT FROM 3-FOOT-THICK FILL PLACEMENT

Exploration	Short-Term Elastic Settlement (Inches)	Long-Term Consolidation Settlement (Inches)	Total Static Settlement (Inches)	Duration for Substantial Completion [1] (Months)
SCPT-23-021	1.0	1.5	2.5	-- [2]
SCPT-23-022	0.5	1.0	1.5	-- [2]
CPT-23-023	0.5	1.0	1.5	-- [2]
SCPT-23-024	1.0	1.0	2.0	-- [2]
SCPT-23-025	0.5	1.0	1.5	-- [2]
CPT-23-026	0.5	1.0	1.5	-- [2]
CPT-23-027	0.5	1.0	1.5	-- [2]
SCPT-23-028	0.5	1.5	2.0	-- [2]
R-23-001	1.5	1.0	2.5	8 - 12
R-23-002	1.0	0.5	1.5	2 - 3

[1] Duration for substantial completion is the time to reaching 90% of the estimated long-term consolidation settlement.

[2] Duration for substantial completion is not part of the CPT-based static settlement method.

The following table below provides the estimated short-term, long-term, and total static settlement and the durations of the long-term settlement assuming a new 10-foot square shallow foundation embedded two feet below finished grade with a bearing pressure of 1,000 psf is placed in the vicinity of the exploration.

ESTIMATED STATIC SETTLEMENT FROM 10-FOOT SQUARE SHALLOW FOUNDATION WITH AN ALLOWABLE BEARING PRESSURE OF 1,000 PSF

Exploration	Short-Term Elastic Settlement (Inches)	Long-Term Consolidation Settlement (Inches)	Total Static Settlement (Inches)	Duration for Substantial Completion [1] (Months)
SCPT-23-021	0.5	0.5	1.0	-- [2]
SCPT-23-022	<0.5	<0.5	0.5	-- [2]
CPT-23-023	0.5	0.5	1.0	-- [2]
SCPT-23-024	0.5	0.5	1.0	-- [2]
SCPT-23-025	<0.5	<0.5	0.5	-- [2]
CPT-23-026	<0.5	<0.5	0.5	-- [2]
CPT-23-027	0.5	0.5	1.0	-- [2]
SCPT-23-028	0.5	0.5	1.0	-- [2]
R-23-001	<0.5	<0.5	0.5	<1
R-23-002	<0.5	<0.5	0.5	<1

[1] Duration for substantial completion is the time to reaching 90% of the estimated long-term consolidation settlement.

[2] Duration for substantial completion is not part of the CPT-based static settlement method.

The assessment of settlement and duration is based on engineering analyses using data obtained from widely spaced explorations, where subsurface conditions could vary significantly across the site. Due to these uncertainties, the estimated settlement and duration could vary across relatively short distances. Settlement monitoring is recommended to confirm these estimates and to plan the timing for construction of settlement sensitive improvements.

DEEP FOUNDATIONS

18- and 24-inch diameter Drilled Displacement Piles (DDP) were evaluated for axial and lateral capacity. DDP displace the soil using a drill tool that is often proprietary to the Piling Contractor and do not generate spoil. The DDP recommendations assume the following:

DDP Assumptions

- Finished Floor Elevation (FFE): +11 feet NGVD 29
- Typical Pile Cutoff Elevation: +7 feet NGVD 29 [4 feet below FFE]
- Pile Diameter: 18 and 24 inches
- Pile Configuration: Single

Geotechnical Conditions

- Average Existing Grade (AEG) Elevation: +10 feet NGVD 29
- Design Groundwater Elevation: +3 feet NGVD 29 [7 feet below AEG]
- Fill: +10 feet to +0 feet NGVD 29 [0 to 10 feet below AEG]
- Upper Paralic Estuarine Deposits: +0 to -50 feet NGVD 29 [10 to 60 feet below AEG]
- Lower Paralic Estuarine Deposits: -50 to -92 feet NGVD 29 [60 to 102 feet below AEG]
- Old Paralic Deposits (Qop): -92 feet NGVD 29 and deeper [102 feet below AEG and deeper]

Axial Capacity

Figures C-2.1 to C-2.4, Allowable Vertical Pile Capacity present downward and upward allowable pile capacities versus embedment depth for 18- and 24-inch diameter DDP. These allowable capacities may be increased by one-third for short-term wind and seismic loads. Figures C-2.5 to C-2.8, Ultimate Vertical Pile Capacity present downward and upward ultimate pile capacities versus embedment depth for 18- and 24-inch diameter DDP. The ultimate downward capacities are adjusted for downdrag loads, which are discussed further in the following section. The estimated capacities assume methods of pile installation that do not compromise shaft resistance and end bearing.

The axial pile group efficiency in compression is 1.0 assuming that piles are installed with a minimum spacing of three pile diameters (3D), center-to-center (CTC). DDP should have a minimum embedment of 25 feet into the Lower Paralic Estuarine Deposits (minimum tip elevation of -75 feet NGVD 29 corresponding to a minimum pile length of approximately 82 feet).

Seismic Settlement and Downdrag

In accordance with ASCE 7-16, the Structural Engineer should include the following liquefaction settlement-induced downdrag. Note that the Net Ultimate Vertical Pile Capacity per ASCE 7-16 is the ultimate vertical pile capacity less the corresponding downdrag load from the table below presented in Figures C-2.5 through C-2.8.

ESTIMATED LIQUEFACTION-INDUCED DOWNDRAG

Pile Diameter, inches	**Downdrag Load, Kips**	
	West (Residential)	**East (New Sports Arena)**
18	130	145
24	165	190

Lateral Capacity

Resistance to lateral loads can be estimated using the passive soil pressure against the pile caps and grade beams above groundwater and the bending resistance of the piles. We do not recommend using friction between pile caps or grade beams and the underlying soil due to the potential for long-term and liquefaction-induced settlement that may reduce the contact between the concrete and soil. The use of passive soil resistance assumes the following:

- The remedial earthwork is completed as recommended in this report.
- There is infinite level ground surrounding the foundations.
- The design groundwater elevation stated in this report.
- The pile caps and grade beams are not deeper than stated in this report.

Passive soil resistance may be estimated using an equivalent fluid weight of 250 pcf for grade beams and pile caps above groundwater that are poured neat against properly compacted fill. This passive pressure is allowable and assumes a factor of safety of 1.5. The upper 12 inches of material in areas without concrete slabs or pavement should not be included in the estimation of passive resistance.

GEOTECHNICAL ANALYSES (Continued)

If passive pressure is used in combination with the bending resistance of piles, the selected passive resistance should be compatible with the deflection of the pile or pile groups providing resistance. To evaluate the lateral displacement of a pile cap under loading, a Passive Force versus Lateral Displacement curve is presented for embedded pile caps 4 feet thick (with 3 feet of embedment) in Figure C-3.1. These recommendations assume remedial earthwork is performed as recommended in this report. Group Delta should be contacted for revised recommendations if the pile caps are deeper than stated in this report.

Lateral capacity of 18- and 24-inch diameter DDP was computed using the computer program LPILE (Ensoft, 2019) using the p-y method. LPILE analyses were performed assuming free and fixed head conditions and pile head deflections of 0.5-, 1-, and 1.5-inch. The DDP were modeled using an elastic section with a cracked moment of inertia (50 percent of the gross moment of inertia), and an axial load of 150 kips. A minimum 28-day compressive strength of 4,000 psi was assumed for the concrete, corresponding to a concrete elastic modulus of approximately 3,600 kips per square inch (ksi). The following preliminary soil parameters were adopted for the lateral pile analyses.

PRELIMINARY LPILE SOIL PARAMETERS

Elevation (ft, NGVD 29)	Depth Below Pile Head (ft)	Layer Unit Description	Liquefiable (Yes or No)	Liquefiable Layer P-Multiplier	LPILE p-y Curve Soil Type	Unit Weight [pcf]	Friction Angle [degrees]	Undrained Strength [psf]
+10 to +3	-3 to 4	Fill	No	N/A	Sand (Reese)	120	32	--
+3 to -5	4 to 12	Fill	Yes	0.05	Sand (Reese)	58	29	--
-5 to -15	12 to 22	Upper Paralic	Yes	0.15	Sand (Reese)	59	30	--
-15 to -30	22 to 37	Upper Paralic	Yes	0.12	Sand (Reese)	58	30	
-30 to -42	37 to 49	Upper Paralic	No	N/A	Stiff Clay w/ Free Water (Reese)	44	--	800
-42 to -50	49 to 57	Upper Paralic	Yes	0.15	Sand (Reese)	57	29	--
-50 to -60	57 to 67	Lower Paralic	No	N/A	Sand (Reese)	58	33	--
-60 to -92	67 to 99	Lower Paralic	No	N/A	Sand (Reese)	58	36	--
-92 and below	99 and below	Old Paralic	No	N/A	Sand (Reese)	58	40	--

We performed analyses using p-multipliers (p_m) of 1.0 and 0.5 to evaluate two potential pile arrangement configurations. A p_m of 1.0 assumes piles are arranged singly or are in groups that have a minimum spacing of 8D, center-to-center. A p_m of 0.5 assumes piles are arranged in groups and are spaced closer than 8D. The table below should be used to evaluate the applicable p_m for specific piles in a group based on the spacing of the piles and the number of rows in the group. To evaluate the capacity of the piles in a group, the capacity of the pile may be linearly interpolated between the values provided for a p_m of 1.0 and 0.5.

P-MULTIPLIERS

Pile CTC Spacing (in the Direction of Loading)	P-Multipliers		
	Row 1	Row 2	Row 3 or Higher
3.0*D	0.75	0.55	0.40
5.0*D	1.00	0.85	0.70
7.0*D	1.00	1.00	0.90

Deflections, maximum shear forces, and bending moments for 18- and 24-inch DDP were calculated using the parameters above for liquefied conditions (see Figures C-3.2 through C-3.9). The table below summarizes the estimated maximum shear at the pile head for each of the pile diameters, fixity conditions, pile head deflection, and p_m that were evaluated. The estimated maximum shear values are unfactored and are considered ultimate values.

ESTIMATED MAXIMUM SHEAR FORCE AT PILE HEAD – 18-INCH DIAMETER DDP

Pile Head Fixity Condition	P-Multiplier, p_m	Maximum Shear at Pile Head (kips)		
		Pile Head Deflection (inches)		
		0.5	*1.0*	*1.5*
Fixed Head	1.0	35.5	49.6	54.7
	0.5	19.8	28.1	31.7
Free Head	1.0	22.1	32.3	35.4
	0.5	12.1	17.7	19.0

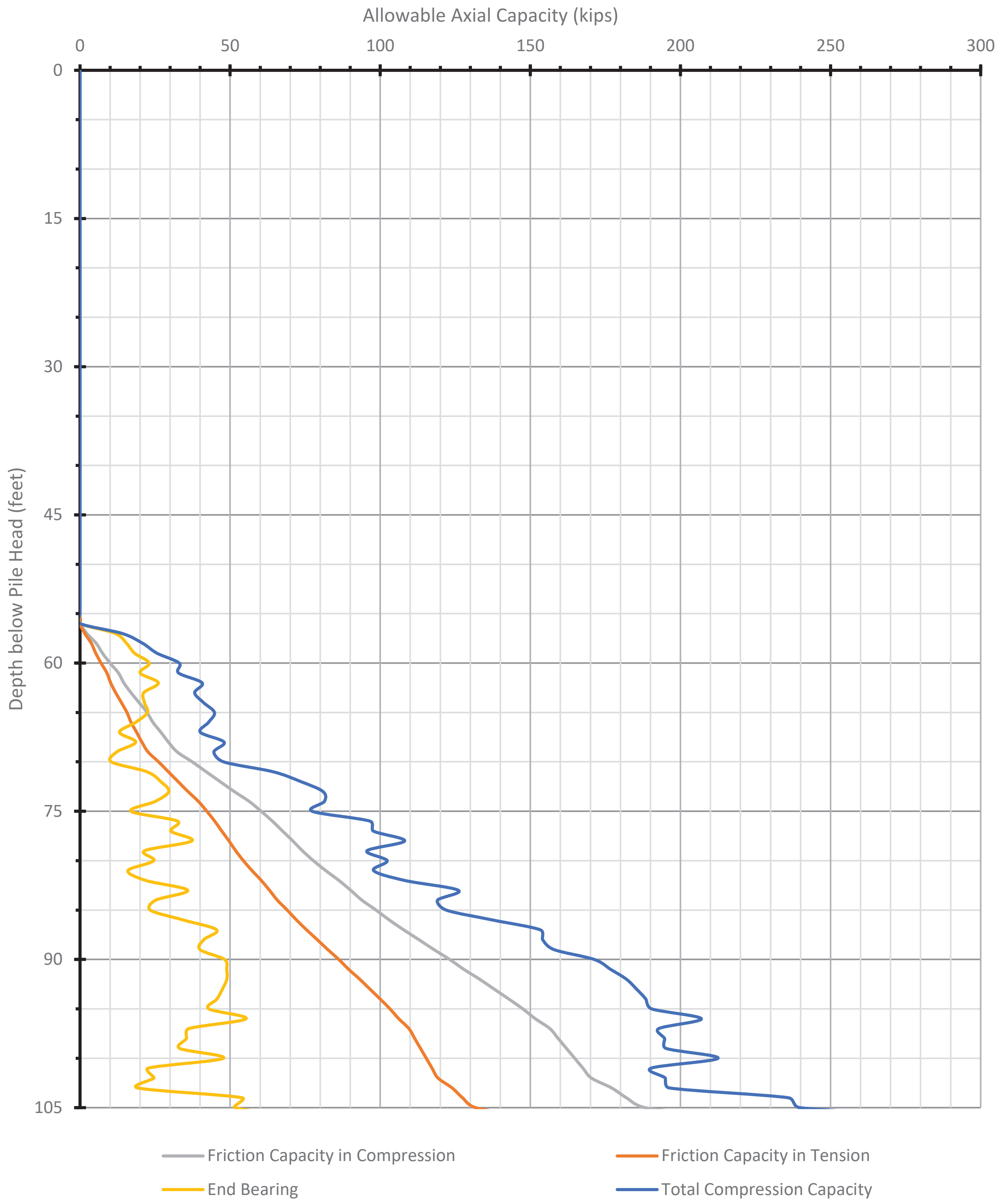
APPENDIX C

GEOTECHNICAL ANALYSES (Continued)

ESTIMATED MAXIMUM SHEAR FORCE AT PILE HEAD – 24-INCH DIAMETER DDP

Pile Head Fixity Condition	**P-Multiplier, p_m**	**Maximum Shear at Pile Head (kips)**		
		Pile Head Deflection (inches)		
		0.5	*1.0*	*1.5*
Fixed Head	1.0	49.3	73.7	83.6
	0.5	28.0	42.5	49.6
Free Head	1.0	31.9	45.8	52.4
	0.5	17.4	25.4	29.0

MIDWAY RISING SPORTS ARENA COMPLEX (WEST)
18-INCH DIAMETER DRILLED DISPLACEMENT PILES



PROJECT NAME

SPORTS ARENA COMPLEX
MIDWAY RISING

FIGURE NAME

AXIAL PILE CAPACITY VERSUS
DEPTH - ALLOWABLE (WEST)

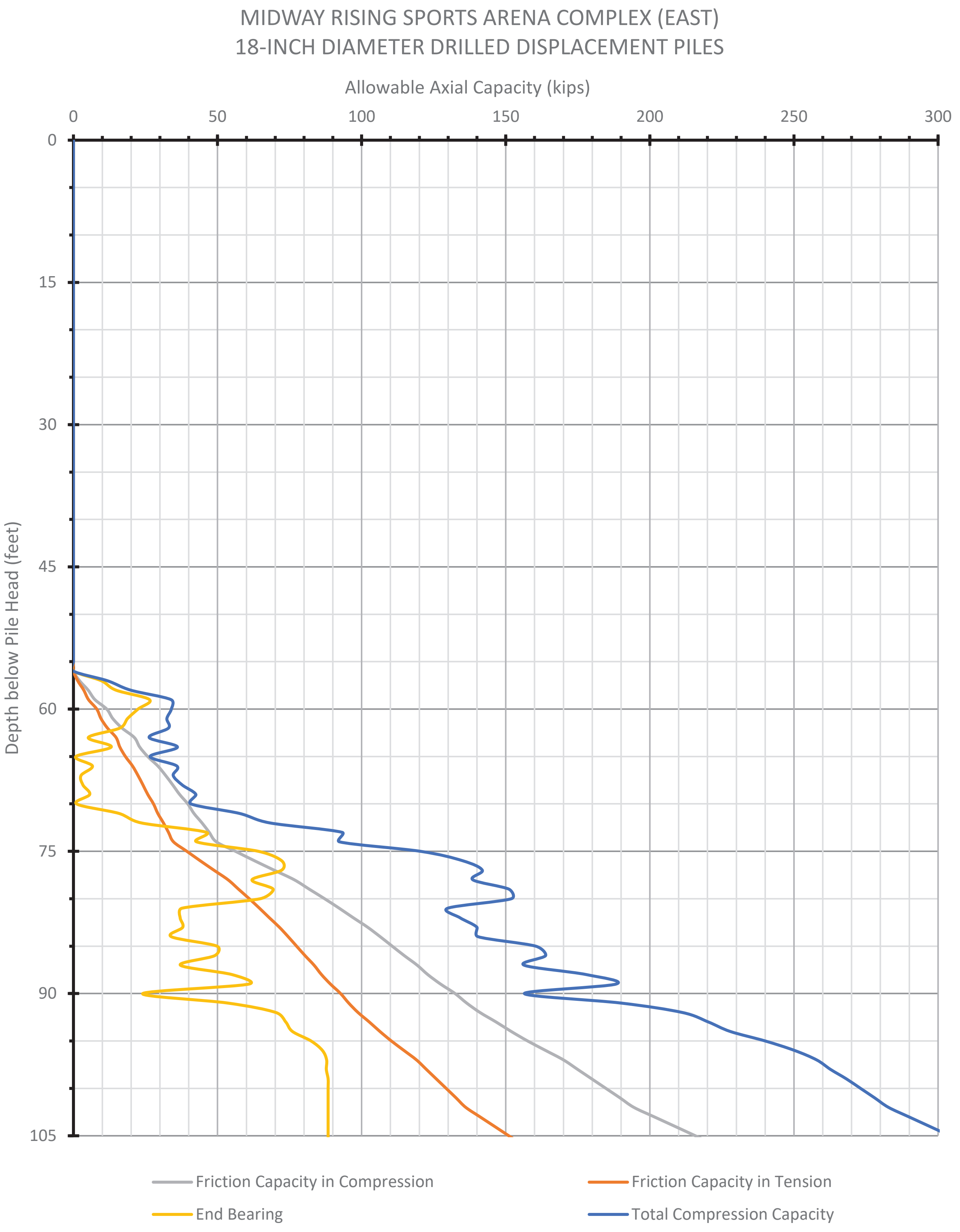
GROUP DELTA

PROJECT NUMBER

SD760

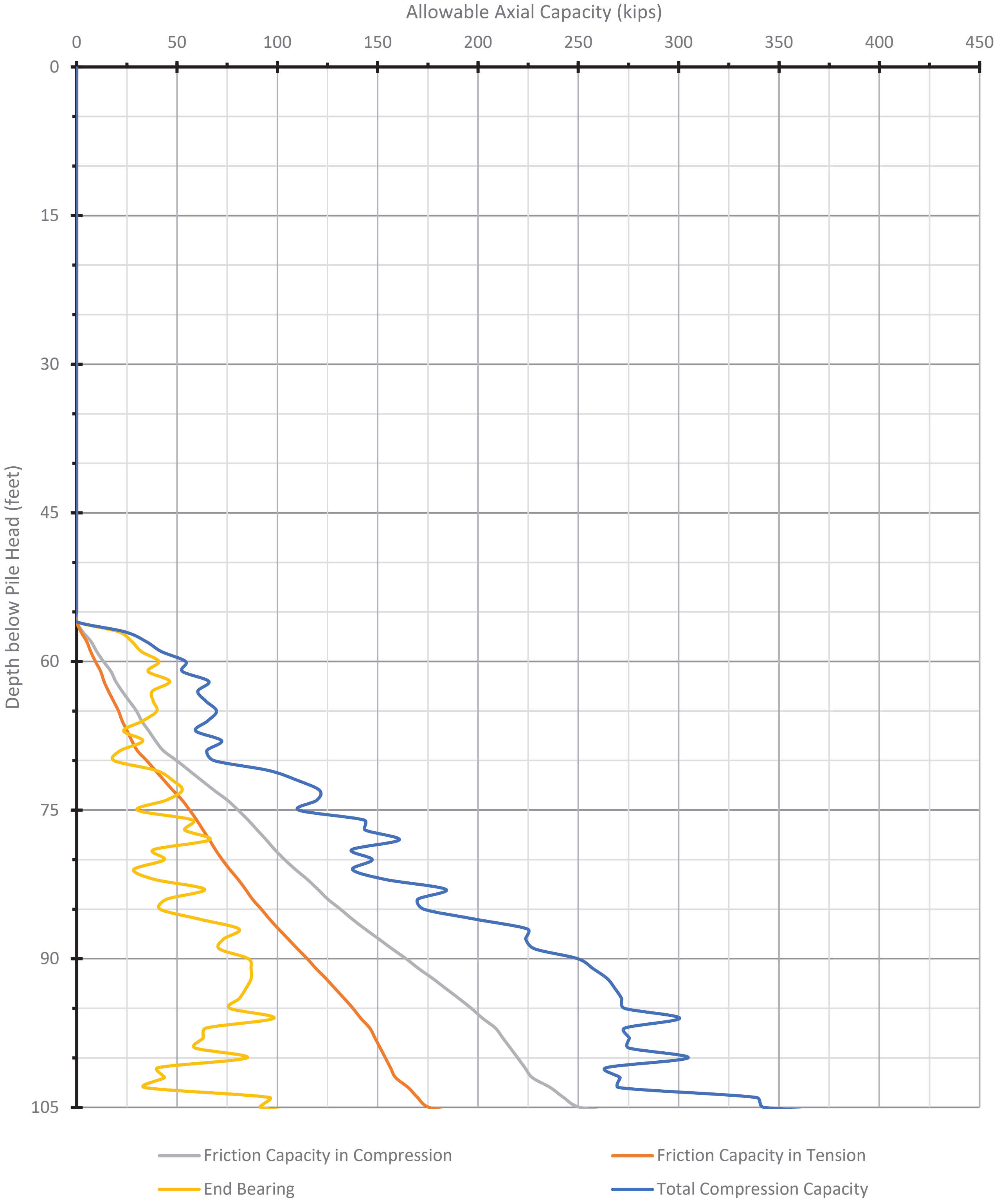
FIGURE NUMBER

C-2.1



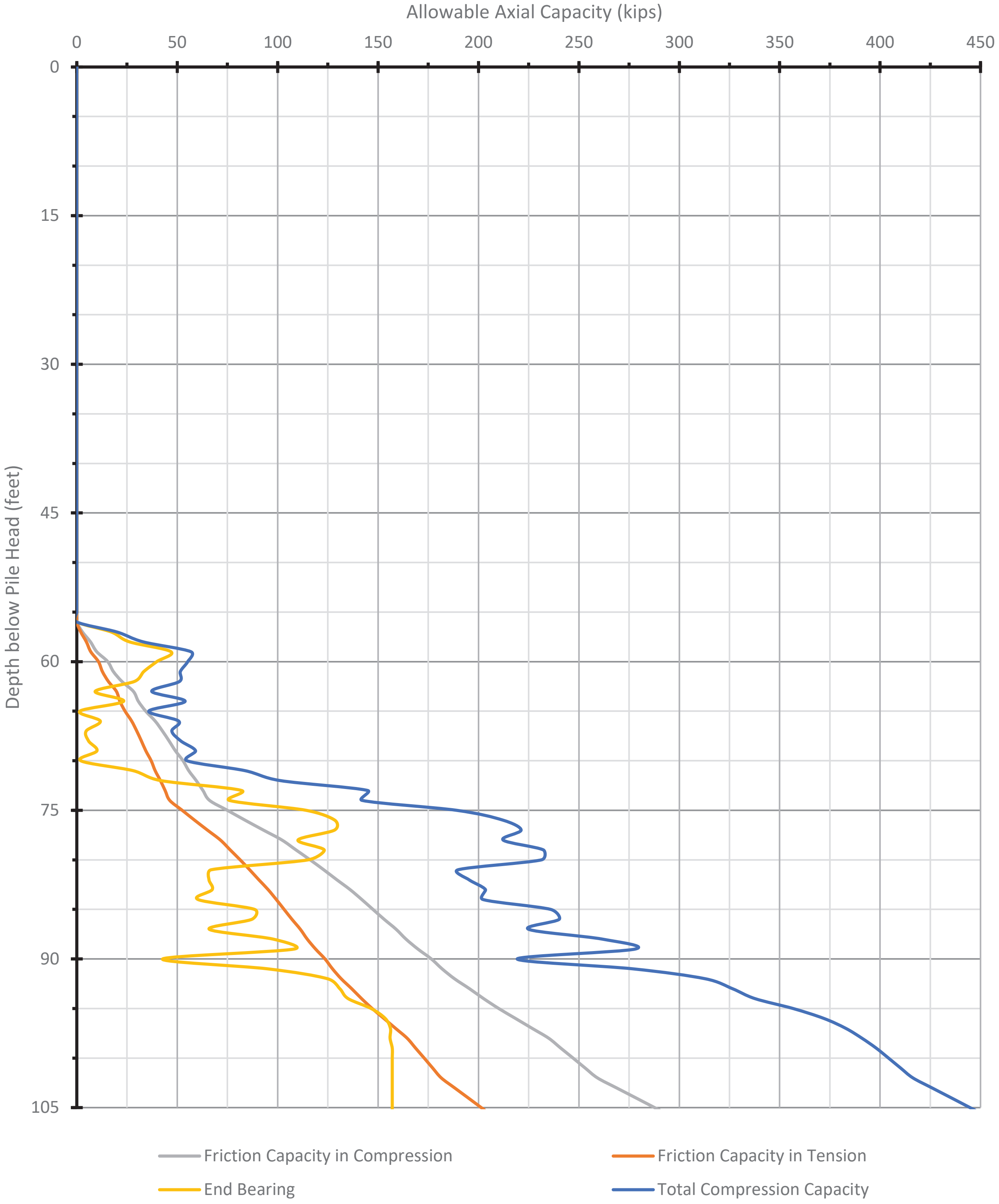
PROJECT NAME	FIGURE NAME		
SPORTS ARENA COMPLEX MIDWAY RISING	AXIAL PILE CAPACITY VERSUS DEPTH - ALLOWABLE (EAST)	GROUP DELTA	
		PROJECT NUMBER SD760	FIGURE NUMBER C-2.2

MIDWAY RISING SPORTS ARENA COMPLEX (WEST)
24-INCH DIAMETER DRILLED DISPLACEMENT PILES



PROJECT NAME	FIGURE NAME		
SPORTS ARENA COMPLEX MIDWAY RISING	AXIAL PILE CAPACITY VERSUS DEPTH - ALLOWABLE (WEST)	GROUP DELTA	
		PROJECT NUMBER SD760	FIGURE NUMBER C-2.3

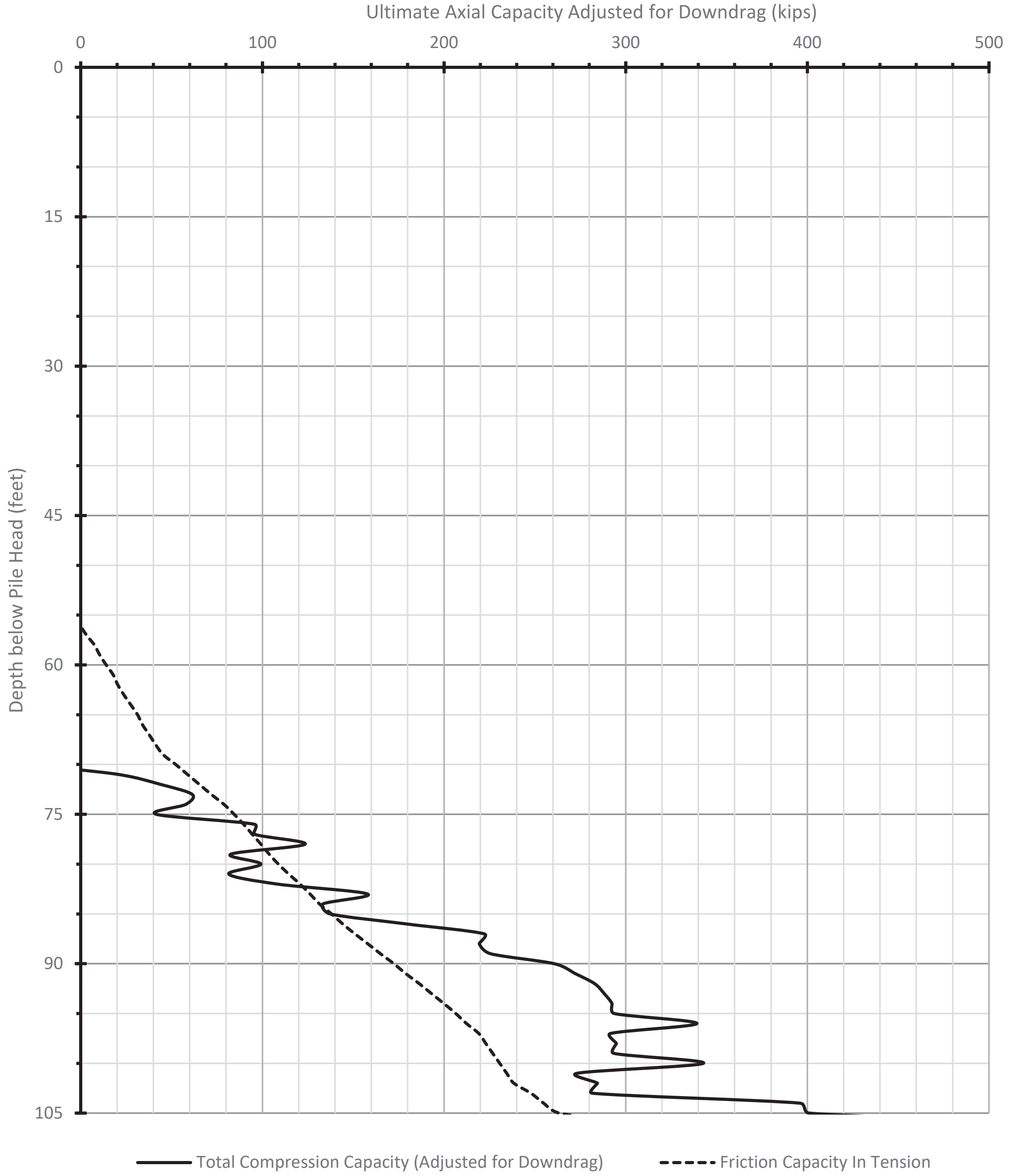
MIDWAY RISING SPORTS ARENA COMPLEX (EAST)
24-INCH DIAMETER DRILLED DISPLACEMENT PILES



PROJECT NAME	FIGURE NAME		
SPORTS ARENA COMPLEX MIDWAY RISING	AXIAL PILE CAPACITY VERSUS DEPTH - ALLOWABLE (EAST)	PROJECT NUMBER SD760	FIGURE NUMBER C-2.4

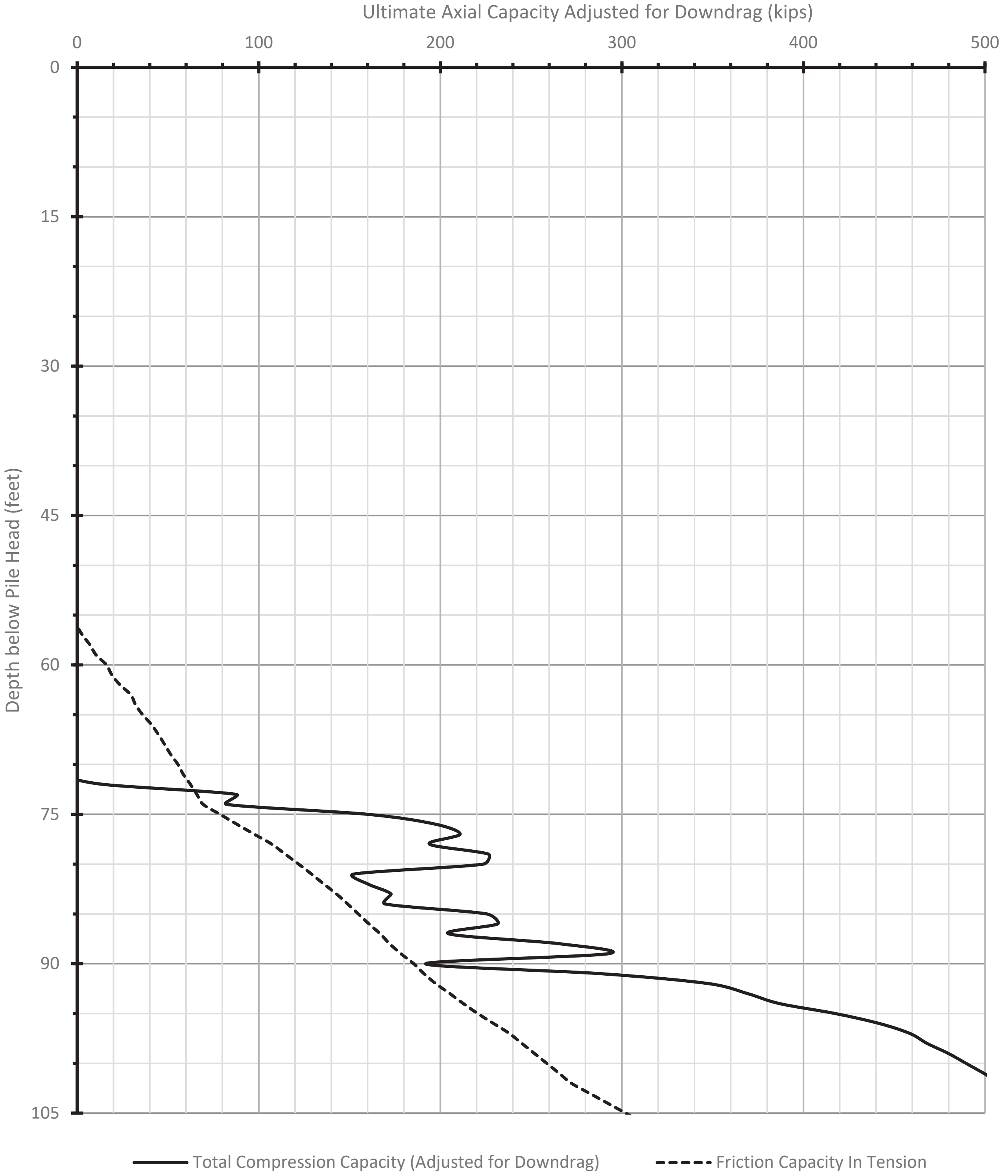
GROUP DELTA

MIDWAY RISING SPORTS ARENA COMPLEX (WEST)
18-INCH DIAMETER DRILLED DISPLACEMENT PILES



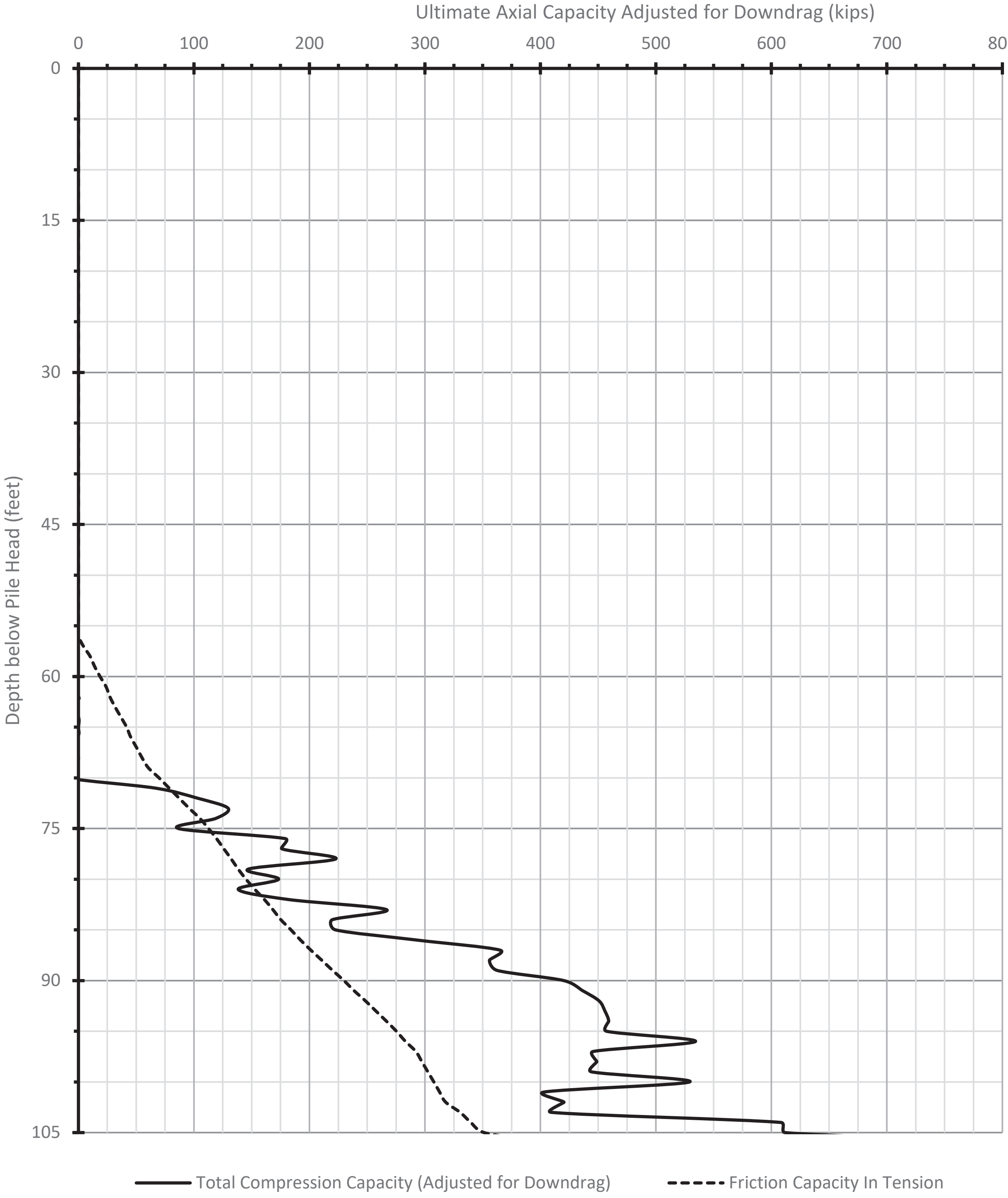
PROJECT NAME	FIGURE NAME	GROUP DELTA	
SPORTS ARENA COMPLEX MIDWAY RISING	AXIAL PILE CAPACITY VERSUS DEPTH - ULTIMATE (WEST)	PROJECT NUMBER SD760	FIGURE NUMBER C-2.5

MIDWAY RISING SPORTS ARENA COMPLEX (EAST)
18-INCH DIAMETER DRILLED DISPLACEMENT PILES

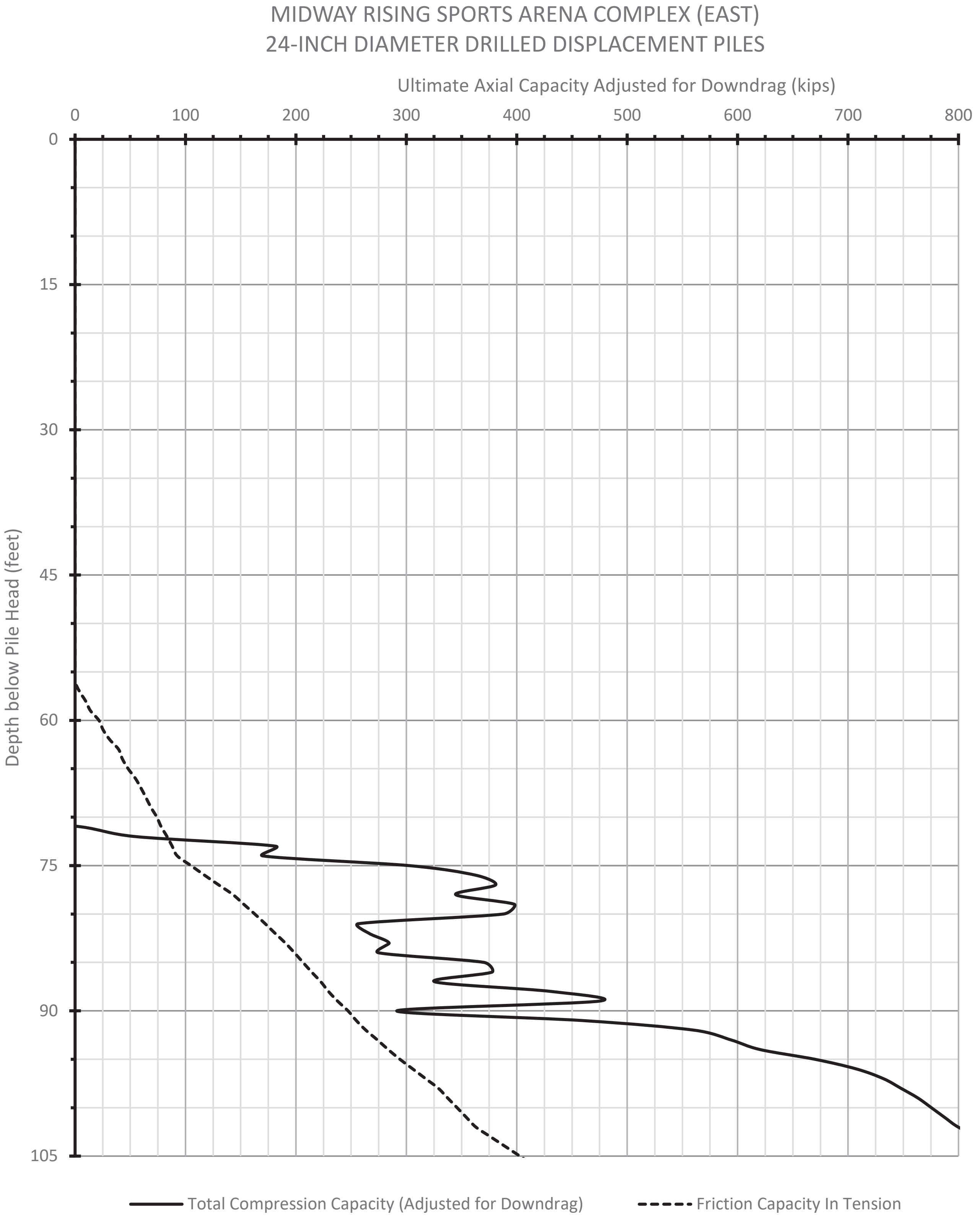


PROJECT NAME	FIGURE NAME	GROUP DELTA	
SPORTS ARENA COMPLEX MIDWAY RISING	AXIAL PILE CAPACITY VERSUS DEPTH - ULTIMATE (EAST)	PROJECT NUMBER SD760	FIGURE NUMBER C-2.6

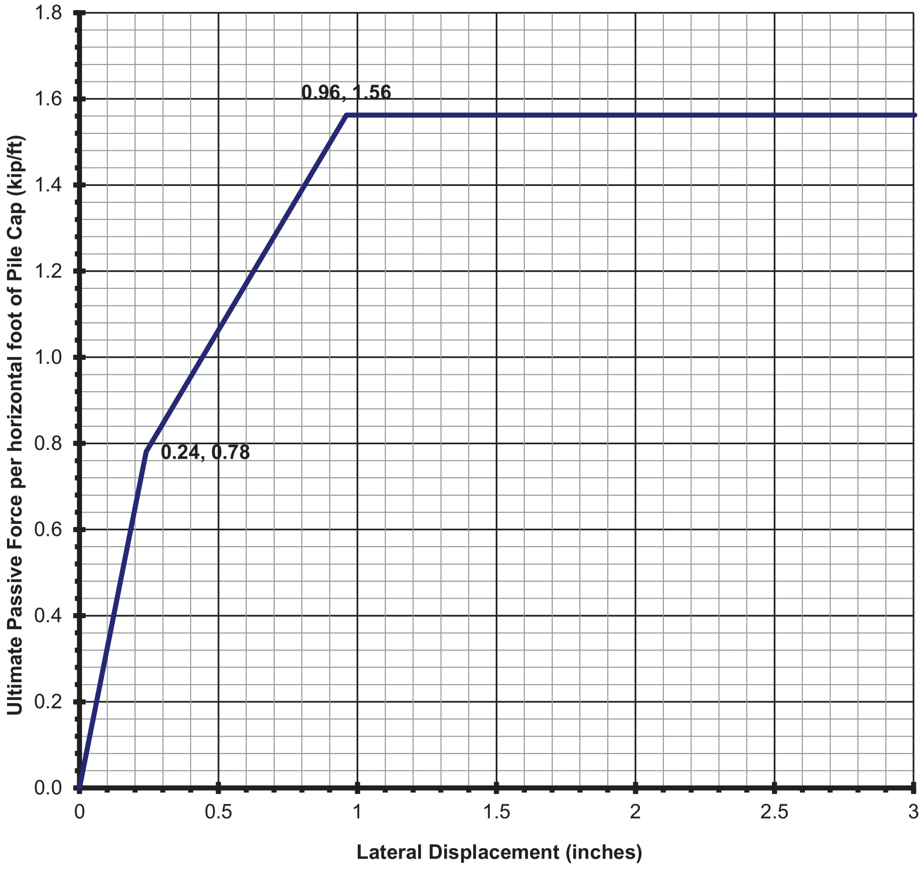
MIDWAY RISING SPORTS ARENA COMPLEX (WEST)
24-INCH DIAMETER DRILLED DISPLACEMENT PILES

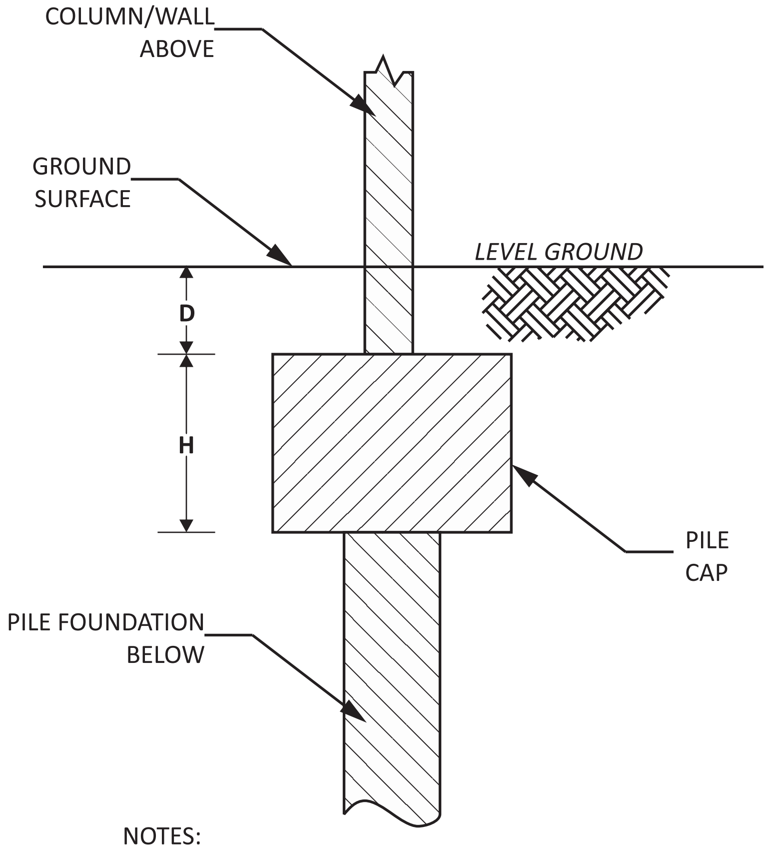


PROJECT NAME	FIGURE NAME	GROUP DELTA	
SPORTS ARENA COMPLEX MIDWAY RISING	AXIAL PILE CAPACITY VERSUS DEPTH - ULTIMATE (WEST)	PROJECT NUMBER SD760	FIGURE NUMBER C-2.7



PROJECT NAME	FIGURE NAME	GROUP DELTA	
SPORTS ARENA COMPLEX MIDWAY RISING	AXIAL PILE CAPACITY VERSUS DEPTH -ULTIMATE (EAST)	PROJECT NUMBER SD760	FIGURE NUMBER C-2.8





NOTES:

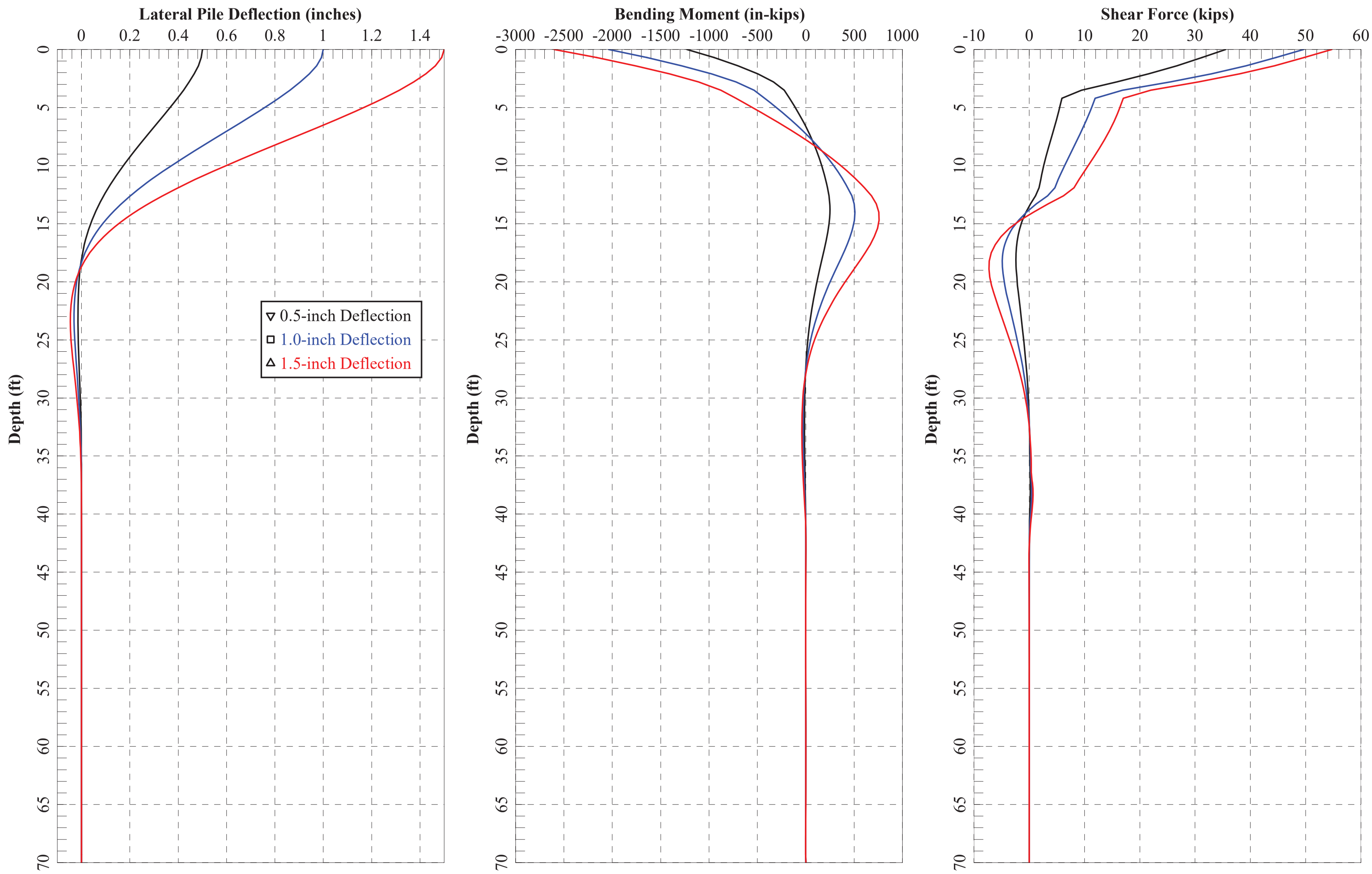
1. SOIL COVER DEPTH, ‘D’ = 1.0 FEET.
2. PILE CAP THICKNESS, ‘H’ = 2.0 FEET.
3. PASSIVE RESISTANCE ASSUMES GRANULAR COMPACTED FILL SOIL MATERIALS ABOVE GROUNDWATER PREPARED AS RECOMMENDED IN THE PRELIMINARY REPORT OF GEOTECHNICAL INVESTIGATION.

REFERENCES: TRILINEAR CURVE FOR IDEALIZED FORCE-DEFLECTION BEHAVIOR OF PILE CAP BASED ON CALTRANS "GUIDELINES ON FOUNDATION LOADING AND DEFORMATION DUE TO LIQUEFACTION INDUCED LATERAL SPREADING," FEBRUARY, 2011.
NAVFAC - DESIGN MANUAL 7.02, "FOUNDATIONS & EARTH STRUCTURES," REVALIDATED BY CHANGE SEPTEMBER 1, 1986, SECTION 2, FIGURE 1, PAGE 7.2-60.

NOT TO SCALE

PROJECT NAME	FIGURE NAME	GROUP DELTA	
SPORTS ARENA COMPLEX MIDWAY RISING	ULTIMATE PASSIVE FORCE VERSUS LATERAL DISPLACEMENT FOR EMBEDDED PILE CAPS	PROJECT NUMBER SD760	FIGURE NUMBER C-3.1

Midway Rising Sports Arena Complex - 18 in DDP - Fixed Head - Pm = 1.0 - Liquefied Condition



PROJECT NAME

SPORTS ARENA COMPLEX
MIDWAY RISING

FIGURE NAME

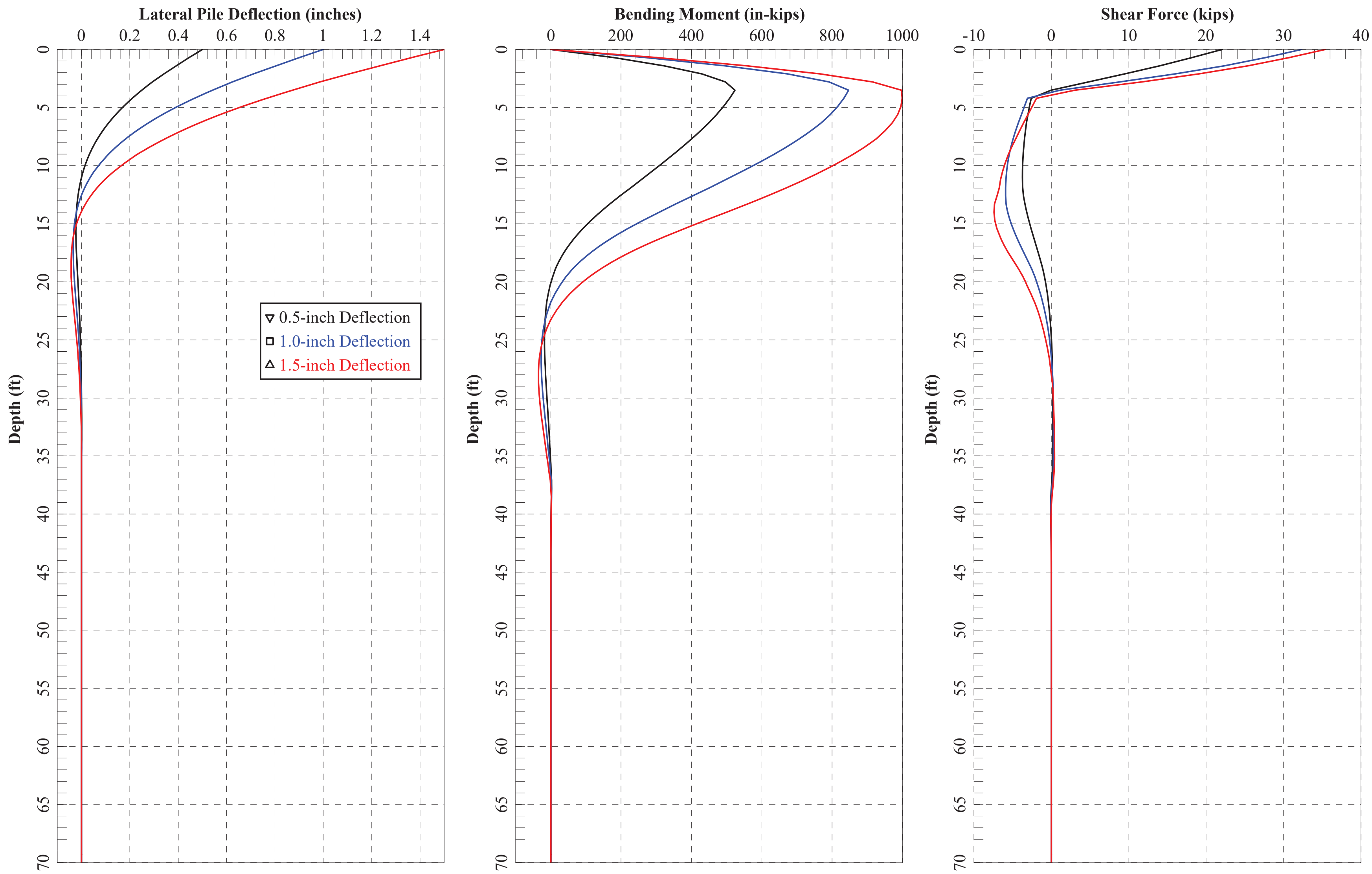
LATERAL PILE DEFLECTION, SHEAR,
AND MOMENT VERSUS DEPTH

GROUP DELTA

PROJECT NUMBER
SD760

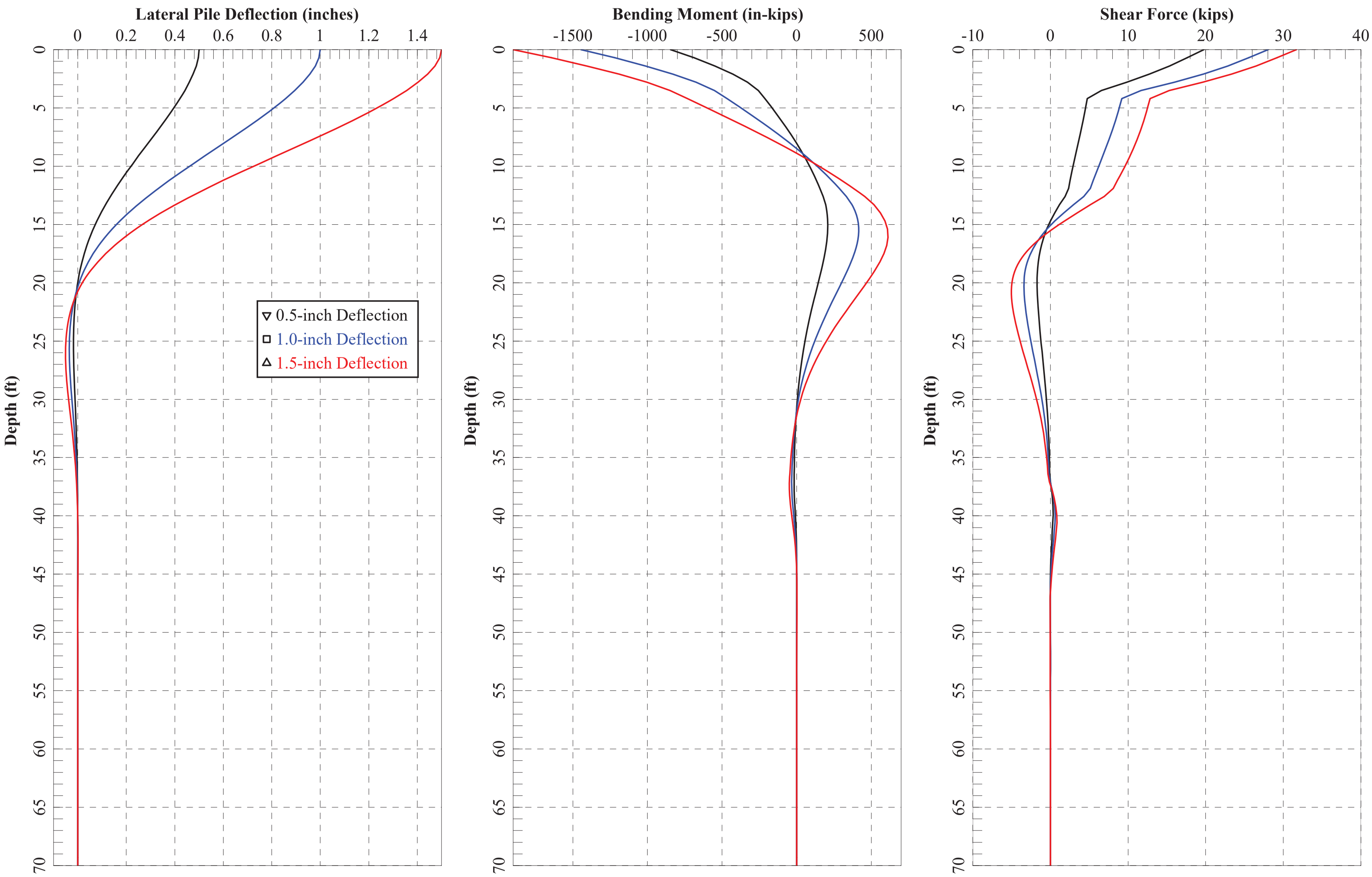
FIGURE NUMBER
C-3.2

Midway Rising Sports Arena Complex - 18 in DDP - Free Head - Pm = 1.0 - Liquefied Condition



PROJECT NAME	FIGURE NAME	GROUP DELTA	
SPORTS ARENA COMPLEX MIDWAY RISING	LATERAL PILE DEFLECTION, SHEAR, AND MOMENT VERSUS DEPTH	PROJECT NUMBER SD760	FIGURE NUMBER C-3.3

Midway Rising Sports Arena Complex - 18 in DDP - Fixed Head - Pm = 0.5 - Liquefied Condition



PROJECT NAME

SPORTS ARENA COMPLEX
MIDWAY RISING

FIGURE NAME

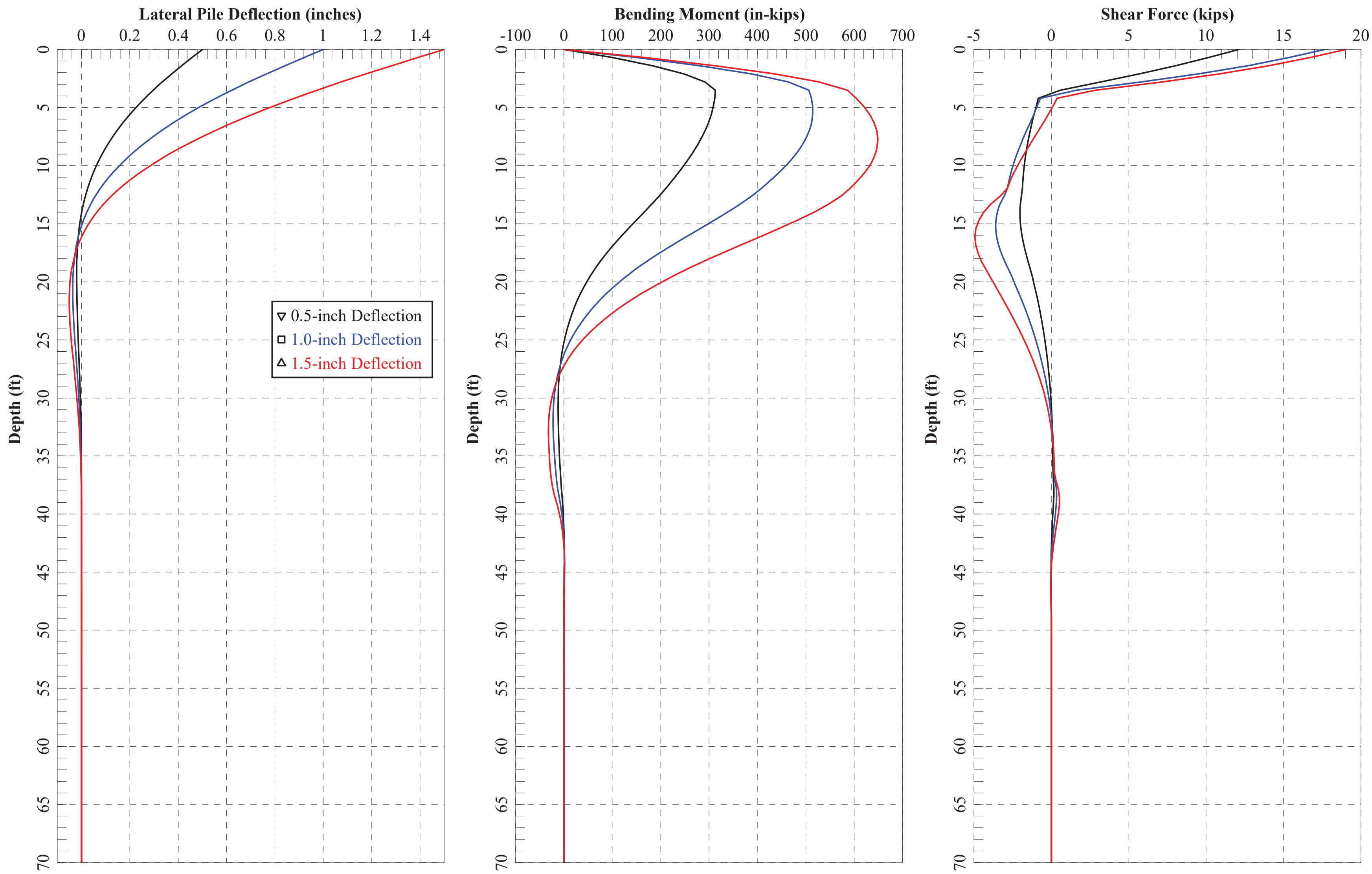
LATERAL PILE DEFLECTION, SHEAR,
AND MOMENT VERSUS DEPTH

GROUP DELTA

PROJECT NUMBER
SD760

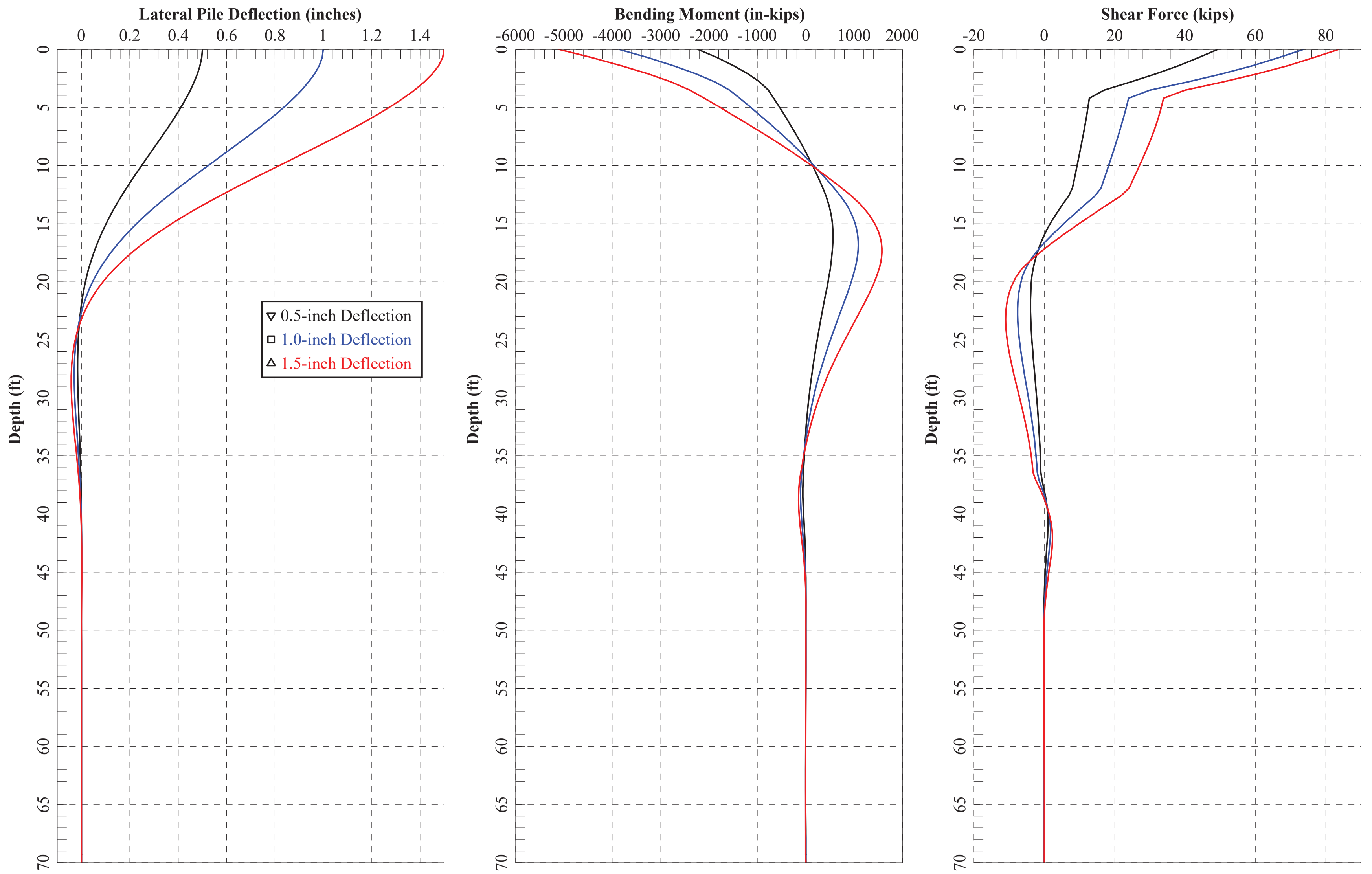
FIGURE NUMBER
C-3.4

Midway Rising Sports Arena Complex - 18 in DDP - Free Head - Pm = 0.5 - Liquefied Condition

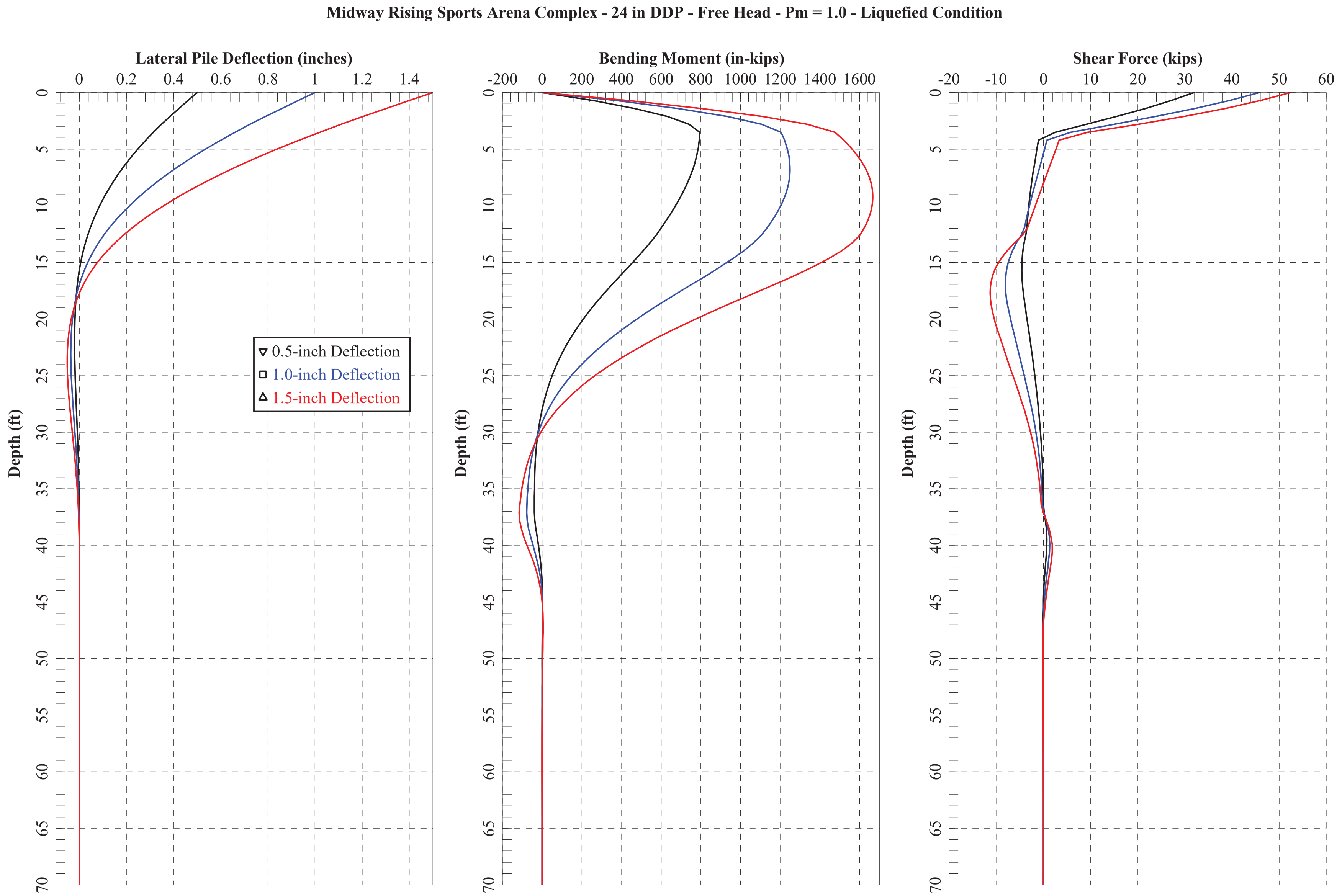


PROJECT NAME	FIGURE NAME		
SPORTS ARENA COMPLEX MIDWAY RISING	LATERAL PILE DEFLECTION, SHEAR, AND MOMENT VERSUS DEPTH	GROUP DELTA	
		PROJECT NUMBER SD760	FIGURE NUMBER C-3.5

Midway Rising Sports Arena Complex - 24 in DDP - Fixed Head - Pm = 1.0 - Liquefied Condition



PROJECT NAME	FIGURE NAME		
SPORTS ARENA COMPLEX MIDWAY RISING	LATERAL PILE DEFLECTION, SHEAR, AND MOMENT VERSUS DEPTH	GROUP DELTA	
		PROJECT NUMBER	FIGURE NUMBER
		SD760	C-3.6



PROJECT NAME

SPORTS ARENA COMPLEX
MIDWAY RISING

FIGURE NAME

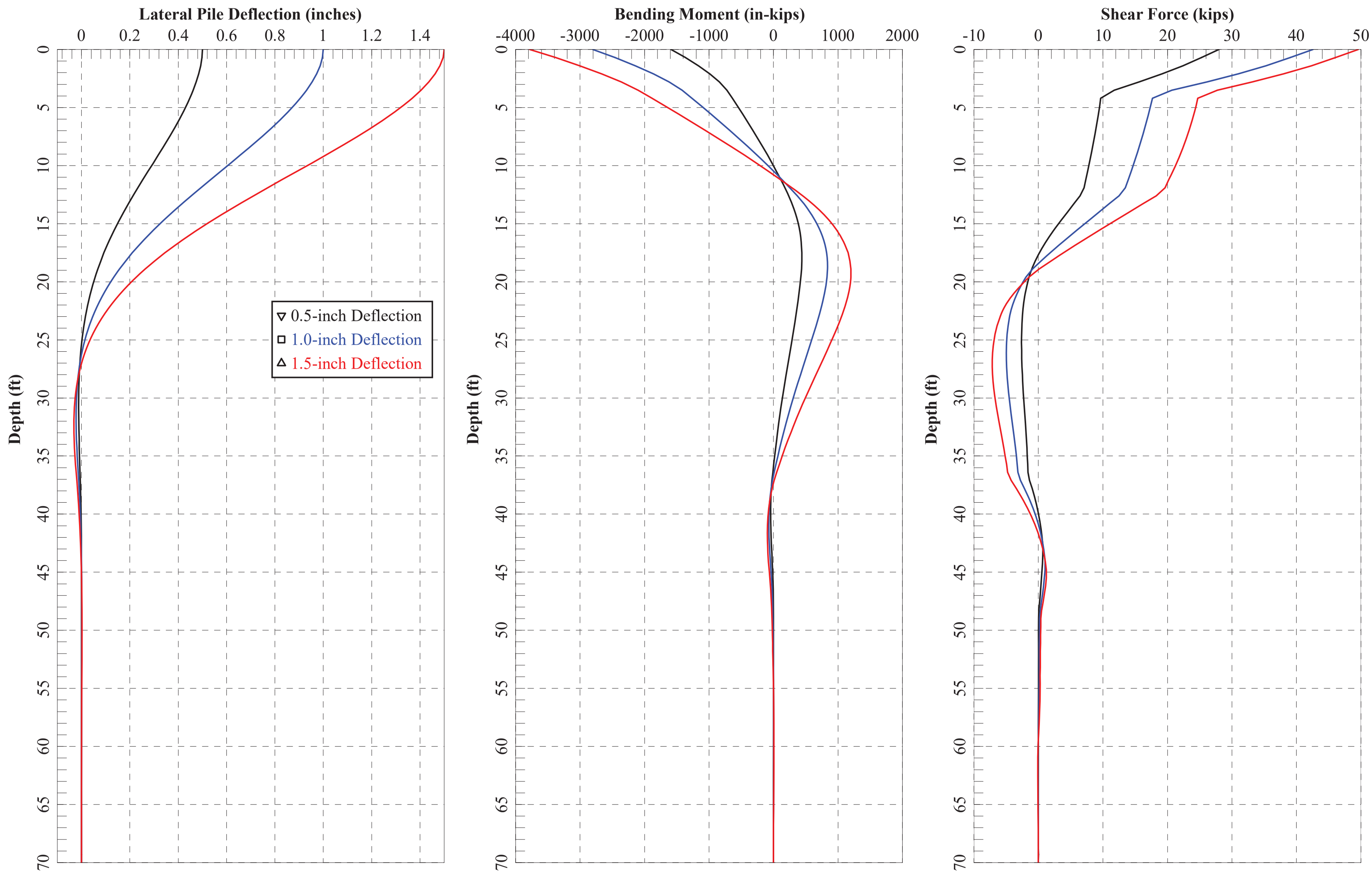
LATERAL PILE DEFLECTION, SHEAR,
AND MOMENT VERSUS DEPTH

GROUP DELTA

PROJECT NUMBER
SD760

FIGURE NUMBER
C-3.7

Midway Rising Sports Arena Complex - 24 in DDP - Fixed Head - Pm = 0.5 - Liquefied Condition



PROJECT NAME

SPORTS ARENA COMPLEX
MIDWAY RISING

FIGURE NAME

LATERAL PILE DEFLECTION, SHEAR,
AND MOMENT VERSUS DEPTH

GROUP DELTA

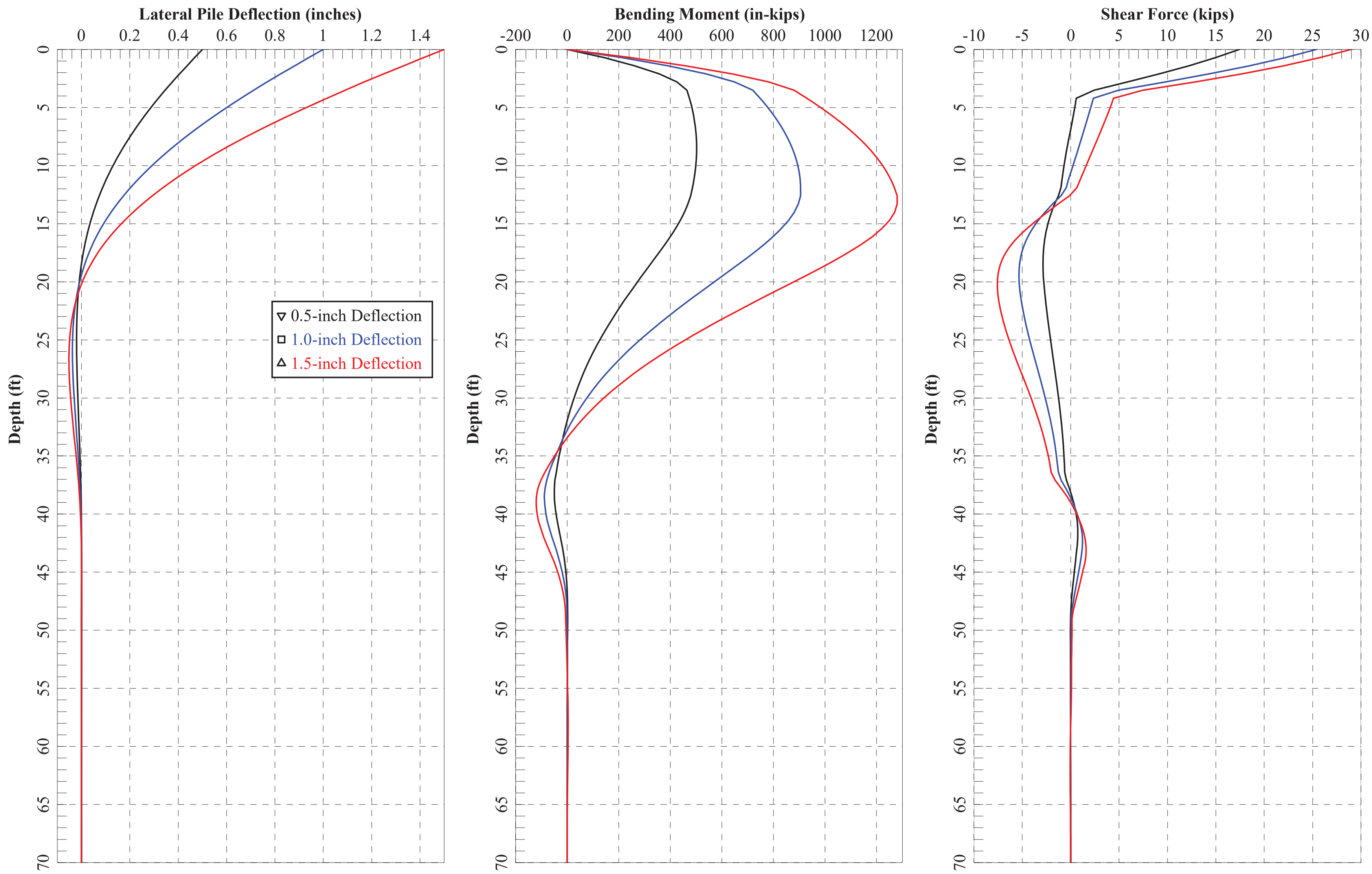
PROJECT NUMBER

SD760

FIGURE NUMBER

C-3.8

Midway Rising Sports Arena Complex - 24 in DDP - Free Head - Pm = 0.5 - Liquefied Condition



PROJECT NAME

SPORTS ARENA COMPLEX
MIDWAY RISING

FIGURE NAME

LATERAL PILE DEFLECTION, SHEAR,
AND MOMENT VERSUS DEPTH

GROUP DELTA

PROJECT NUMBER
SD760

FIGURE NUMBER
C-3.9