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June 19, 2020
Kleinfelder Project No. 20180876.001A

Mr. Tim Thiele, PE, QSD
City Engineer | Michael Baker International
City of Del Mar
1050 Camino Del Mar
Del Mar, California 92014

SUBJECT: Preliminary Foundation Report
Camino Del Mar Bridge Replacement
Over San Dieguito River
Del Mar, California

Dear Mr. Thiele:

Kleinfelder is pleased to present this Preliminary Foundation Report (PFR) for the Camino Del Mar Bridge Replacement over San Dieguito River located in Del Mar, California. This report includes preliminary foundation recommendations for the proposed replacement bridge and includes evaluations of the associated improvements such as approach fills and retaining walls for the construction of the project. This report is presented in conjunction with the Preliminary Geotechnical Design Report (PGDR) for the project.

We appreciate this opportunity to be of service and look forward to continuing to work with you in the future. If you have any questions about this report or need additional services, please contact us at 619.831.4600.

Respectfully submitted,

KLEINFELDER

Janna Bonfiglio, PE 89334
Project Engineer

Scot Rugg, CEG 1651
Senior Engineering Geologist

Karthik Radhakrishnan, GE 3046
Senior Program Manager

DOCUMENT 5

PRELIMINARY FOUNDATION REPORT
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER
DEL MAR, CALIFORNIA
KLEINFELDER PROJECT NO. 20180876.001A

JUNE 19, 2020

A Report Prepared for:

Mr. Tim Thiele, PE
City Engineer | Michael Baker International
City of Del Mar
1050 Camino Del Mar
Del Mar, California 92014

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1 INTRODUCTION

The City of Del Mar has retained Kleinfelder to provide engineering services of the project plans, specifications and estimate (PS&E) phase of the Camino Del Mar Bridge Replacement project. The project will replace the existing bridge along Camino Del Mar and over the San Dieguito River. The purpose of this Preliminary Foundation Report (PFR) is to provide preliminary evaluation of site subsurface conditions, potential geologic and seismic hazards, and provide preliminary foundation recommendations to aid in the type selection for the proposed replacement bridge. The PFR is prepared in accordance with Caltrans' Foundation Report Guidelines (Caltrans, 2017) and is a companion report to a separately provided Preliminary Geotechnical Design Report (PGDR) for the proposed project. This PFR is not intended for final design of the project. Additional investigations and analyses will be required for final design as recommended in Section 5 of this report.

1.1 SCOPE OF WORK

The purposes of this report are to present the results of our Phase 0 geotechnical engineering investigation, evaluate the subsurface conditions at the site, determine potential geologic/seismic hazards, perform geotechnical engineering evaluations, and provide preliminary foundation recommendations for the proposed project.

The scope of services for this study included the following:

- Review of readily available geotechnical and geologic information including published geologic maps, topographic maps, aerial photography, previous and nearby geotechnical reports, and as-built and conceptual drawings;
- Obtain necessary geotechnical permits for performing explorations within the City of Del Mar right-of-way including preparation of a geotechnical investigation work plan;
- Coordination and oversight of utility clearance surveys, traffic control, and pavement coring for proposed exploration locations;
- Coordination and oversight of two exploratory borings and three Cone Penetrometer Tests (CPTs) within the existing Camino Del Mar Bridge Replacement project site;
- Performing laboratory testing on collected soil samples from the borings;
- Preparing this PFR which includes the following:
    - A description of the existing site and proposed project improvements including a site vicinity map and a site plan showing approximate locations of field explorations;

    - Discussion of pertinent geotechnical and geologic information based our review of existing geotechnical reports for the site and other available geotechnical and geologic information;
    - Discussion of field exploration methods, logs of borings and CPTs, and laboratory test procedures and results;
    - Discussion of the site and subsurface conditions observed during our field investigation;
    - Discussion of the regional geologic and seismic setting and potential geologic hazards at the site;
    - Seismic design parameters in accordance with the California Department of Transportation (Caltrans) 2019 Seismic Design Criteria including performance of a site-specific response analysis;
    - Preliminary bridge foundation recommendations including axial pile capacity;
    - Approach fill stability and settlement recommendations;
    - Preliminary recommendations for retaining walls;
    - Discussion of temporary excavations and shoring;
    - Discussion of soil corrosivity properties affecting below-grade concrete and steel;
    - Recommendations for further field investigations.
- Preparation of a Preliminary Geotechnical Design Report (PGDR) which is provided under a separate cover.

The recommendations contained within this report are subject to the limitations presented in Section 6.0 and are in conjunction with the PGDR for this project.

1.2 PROJECT DATUM

Unless otherwise noted, elevation data presented in this report are referenced to the North American Vertical Datum of 1988 (NAVD88) and the stationing is referenced from the project conceptual design drawings.

1.3 EXISTING SITE CONDITIONS AND AS-BUILT INFORMATION

The Camino Del Mar Bridge Replacement project site is located along the coast in Del Mar, California, crossing over the San Dieguito River which flows from the east and discharges into the Pacific Ocean. Based on our review of the project conceptual drawings and the topographic survey prepared by Sampo Engineering, Inc. and dated April 13, 2018, the site limits extend from approximately 400 feet north of the northern end of the bridge (approximate Station 170+00), near the access point to Del Mar North Beach, to approximately 400 feet south of the southern end of the bridge (approximate Station 156+00), just south of Sandy Lane. The existing bridge structure extends from approximate Station 166+00 at the northern end to approximate Station 160+00 at the southern end. The general site vicinity is shown on Figure 1 and the existing conditions of the site are provided on Figure 2. The coordinates of the approximate center of the bridge structure are:

Latitude:	32.9750 °N	Longitude:	117.2690°W

The project site is bounded by the on-grade portion of Camino Del Mar roadway which eventually intersects with Via De La Valle to the north and Sandy Pointe to the south. The existing San Dieguito River and the Del Mar Racetrack venue bounds the project site to the east and the Del Mar North Beach, residential housing, and the Pacific Ocean bounds the project site to the west. The extents of the recreational beach areas located below and beyond the southern and northern portions of the bridge are dependent upon the season (dry or rainy season) and the typical tidal changes of approximately 4 feet throughout the day (NOAA, 2020). Based on our review of National Oceanic and Atmospheric Administration (NOAA) tidal information, we understand that the typical current tide elevations range from approximate elevation +0 feet to +4 feet throughout the day.

At the southern area of the bridge, existing grades of the beach area below the bridge generally range from approximate elevations +5 to +9 feet with a berm having an approximate slope inclination of 1½ horizontal to 1 vertical (1½H:1V) and ranging in elevation from approximately +6 to +16 feet extending up from the beach area to the bridge abutment. The surface of this berm at the southern end of the bridge is covered with rip-rap and some vegetation for erosion control.

Within the northern area of the bridge, existing grades of the beach area generally range from approximate elevations +5 to +8 feet with the roadway elevation at approximately +18 feet extending up from the beach area to the bridge abutment. The slope inclination of this berm is up to approximately 1¼H:1V and this slope is also covered with rip-rap and some vegetation for erosion control.

Based on our site reconnaissance, our review of as-built drawings (Powell and T.Y. Lin, 2001; Caltrans, 1951), and our review of the topographic survey, current conditions at the project site consist of the approximate 596-foot-long reinforced concrete girder bridge supported by ten piers and two abutments. Per the as-built plans and bridge inspection reports, the existing bridge was built in 1932 and widened with a pedestrian walkway and curb in 1953. Additional improvements to the bridge including replacement of pavements, pedestrian walkway, and railings were performed in 2001. Our review of the as-built drawings for the existing bridge indicates that the pier and the abutment pile caps extend to approximate elevation -17 ft National Geodetic Vertical Datum of 1929 (NGVD29) and are supported on 15-inch-diameter timber piles. The existing piers are spaced at 54 feet and are each supported by a total of 41 timber piles that are configured in three longitudinal rows with 14 piles in the outer rows and 13 piles in the center row. The existing pile lengths at the piers are unknown as the as-built drawings indicate pile depths were to be determined based on load testing. However, the as-built drawings do indicate the timber piles at the piers were assumed to be 25-feet-long, or extend to approximate elevation -42 ft NGVD29, for estimating purposes. The abutments are supported on a total of 66 timber piles also extending to an unknown depth. The as-built drawings indicate that some of these timber piles may be battered at the abutments. A summary of the existing foundation data is provided in Table 1.

Table 1

Summary of As-Built Foundation Data

Location	Foundation Type	No. of Piles at Each Support	Approx. Bottom of Pile Cap Elevation[1] (NGVD29)	Approx. Tip Elevation[1] (NGVD29)
Abutments	15-inch-diameter Timber Piles	66	-17 ft	-42 ft
Piers	15-inch-diameter Timber Piles	41	-17 ft	-42 ft

Notes: [1] Elevations are approximate as bottom of pile cap elevations and pile lengths are provided on the as-built drawings but were indicated to be for cost estimating purposes only.

Outside of the existing bridge limits, asphalt concrete (AC) pavement exists along the on-grade approach embankments along Camino Del Mar. A concrete median filled with landscaping separates the northbound and southbound directions of Camino Del Mar. Concrete sidewalks line the east and west sides of the on-grade portions of Camino Del Mar to the north and south of the bridge. An existing wire fence is located along the eastern sidewalk to the north of the bridge due to the steep embankment slopes extending along the east side of the street. Furthermore, street signs for pedestrian crosswalks are also present just south and north of the existing bridge.

The as-built drawings also indicate potential abandoned timber piles from an abandoned highway bridge located to the west of the existing Camino Del Mar bridge as well as for an abandoned pipeline trestle located adjacent to the east side of the existing bridge.

Based on our site reconnaissance, utilities observed at the site include a 12-inch-diameter high pressure gas line and a 12-inch-diameter sanitary sewer line which are hung from the eastern side of the bridge and traverse the eastern side of the on-grade portion of Camino Del Mar. Additionally, a 4-inch-diameter high pressure gas line is hung from the western side of the bridge and traverses the western on-grade portion of Camino Del Mar. Communications markers and a an electrical box were also observed to the east of the Camino Del Mar roadway.

The existing conditions of the project site as well as the exploration locations are presented on Figure 2.

1.4 PROJECT DESCRIPTION

Based on our review of the draft conceptual drawings for the Bridge Type Selection Report (TSR) by Kleinfelder and discussions with the project design team, the proposed project is still in the bridge type selection phase and we understand that, after assessment of several alternatives, five bridge options are still currently being considered. These alternatives consist of various 5-span and 6-span cast-in-place box girder bridge options as well as 6-span precast concrete girder bridge options. Details of the features of each alternative including the pile type assumptions by the structural engineers and dimensions for each bridge option are provided in Table 2.

Table 2

Type Selection Bridge Options and Associated Pile Details

Alternative Number	Bridge Type Option	Abutment Support Assumptions	Pier Support Assumptions
1.1	5-Span Variable Depth Cast-In-Place Box Girder	Four 5-ft-diameter CIDH piles with 6-ft-diameter casing	Four piers each supported on two 10-ft-diameter CIDH piles with 11-ft-diameter casing
2.1	6-Span Variable Depth Cast-In-Place Box Girder	Four 5-ft-diameter CIDH with 6-ft-diameter casing	Five piers each supported on two 9-ft-diameter CIDH piles with 10-ft-diameter casing
2.2	6-Span Variable Depth Cast-In-Place Box Girder	Four 5-ft-diameter CIDH with 6-ft-diameter casing	Five piers each supported on four 6-ft-diameter CIDH piles with 7-ft-diameter casing
9.1	6-Span Prestressed Precast Concrete Girder	Four 5-ft-diameter CIDH with 6-ft-diameter casing	Five piers each supported on two 9-ft-diameter CIDH piles with 10-ft-diameter casing

Table 2 (Continued)

Type Selection Bridge Options and Associated Pile Details

Alternative Number	Bridge Type Option	Abutment Support Assumptions	Pier Support Assumptions
9.2	6-Span Prestressed Precast Concrete Girder	Four 5-ft-diameter CIDH with 6-ft-diameter casing	Five piers each supported on four 6-ft-diameter CIDH piles with 7-ft-diameter casing

Note: 1. CIDH = Cast-In-Drilled-Hole Pile
2. Casing proposed to consist of smooth wall permanent steel casing

We understand that the width of the proposed bridge structure will be approximately 68½ feet and will be constructed in a two-phased system allowing continuous traffic flow during construction. The locations of the abutments and bents for each option vary but are anticipated to consist of constructing the proposed abutments behind the existing abutments and keeping portions of the existing abutments in place as additional scour and erosion protection. The existing piles for the piers will be removed with the proposed piers and associated piles will straddle the locations of the existing piers. As the new abutments are anticipated to be constructed behind the existing abutments, the length of the proposed Camino Del Mar replacement bridge is approximately 624 feet from the beginning of bridge (BB) to the end of bridge (EB).

Based on conversations with the project team and review of the draft conceptual plans, we understand that the design storm elevation is +14.55 feet corresponding to the 50-year storm plus 2 feet of freeboard water elevation. Due to this design storm elevation, the proposed bridge is required to be raised to a higher level than the existing bridge. We understand that several grading profiles are currently being evaluated that will require new approach fills and associated retaining walls extending from the edges of the abutments along the on-grade portion of Camino Del Mar. At this stage of the project, we understand that the proposed approach retaining wall type has not yet been selected and that the final wall dimensions are still under design. Based on the conceptual plans, the proposed approach fills are anticipated to be highest at the bridge abutment and will be graded to meet existing roadway grades away from the bridge. The extents of the approach fills are approximately 300 feet to the north and south of the proposed abutments.

In order to place the proposed approach fills, the existing asphalt pavement along the on-grade portion of Camino Del Mar will be demolished. Upon completion of fill placement, the on-grade surficial pavement will be replaced with new asphalt concrete pavement and an approximate 30-foot-long concrete approach slab.

Permanent proposed slopes below the existing bridge, in front of the proposed abutments, are anticipated to be approximately 2H:1V. Temporary cut slopes and shoring may be required for the removal of portions of the existing bridge abutments and for re-direction of existing utilities at the site while cofferdams are anticipated for the proposed CIDH piles at the pier locations. Furthermore, temporary piles are anticipated to be required for temporary trestle bridges in order to construct the proposed deep foundations for the replacement bridge.

The current proposed conditions at the project site as well as the exploration locations are presented on Figure 3.

1.5 EXCEPTIONS TO POLICIES AND PROCEDURES

No exceptions to Caltrans policies or procedures were taken for the preparation of this report.

2 FIELD INVESTIGATION

Our preliminary geotechnical investigation (Phase 0 investigation) consisted of review of available geotechnical information, advancing two exploratory borings, advancing three cone penetrometer tests (CPTs), and laboratory testing. The borings and two of the CPTs were performed within accessible areas near the existing bridge abutments. A third CPT was performed on the existing bridge deck near the central portion of the of the existing Camino Del Mar bridge. Laboratory testing and review of existing geotechnical and geologic information were also performed for our geotechnical investigation and are discussed in the subsequent sections. The approximate locations of the borings and CPTs performed by Kleinfelder are shown on Figures 2 and 3.

2.1 REVIEW OF EXISTING GEOTECHNICAL INFORMATION

The following previous geotechnical reports have been reviewed as part of our scope:

- “Preliminary Geotechnical Design Report (PGDR), Camino Del Mar Bridge Replacement (Bridge No. 57C-0209), Del Mar, California,” prepared by Ninyo & Moore, dated July 31, 2018.
- “Preliminary Foundation Report (PFR), Camino Del Mar Bridge Replacement (Bridge No. 57C-0209), Del Mar, California,” prepared by Ninyo & Moore, dated July 31, 2018.
- “Progress Report of Foundation Investigation on Road XI-SD-2-SD,A, San Dieguito River Basin, Station 1216 to Station 1280,” prepared by the California Department of Public Works, Division of Highways (as available online on GeoDOG), dated May 25, 1960.
- “Supplemental Report of Foundation Investigation on Road XI-SD-2-SD,A, San Dieguito River Basin, Station 1216 to Station 1280,” prepared by the California Department of Public Works, Division of Highways (as available online on GeoDOG), dated September 12, 1960, and associated logs of the borings (LOTBs).

The Ninyo & Moore reports were prepared for the Camino Del Mar Bridge Replacement project in which the results of two borings, designated as B-7 and B-8, and two CPTs, designated as CPT-11 and CPT-12, performed in March 2013 were presented. Boring B-7 and CPT-11 were performed near the existing northern bridge abutment and Boring B-8 and CPT-12 were performed near the existing southern bridge abutment. In general, Ninyo & Moore reportedly encountered undocumented fill material overlying successive strata of alluvial deposits and the Del Mar Formation in their explorations. A summary of the Ninyo & Moore explorations is provided in Table 3 and a summary of the subsurface conditions reported from the explorations is provided in Table 4.

Table 3
Summary of Previous Geotechnical Investigation

Exploration No.	Location (Closest Approx. Station)	Drill Date	Reported Ground Surface Elev. (ft, MSL)	Drilled Depth (ft)
B-7	North Abutment (167+00)	3/11/2013	+17	81½
B-8	South Abutment (158+00)	3/15/2013	+14	95
CPT-11	North Abutment	3/13/2013	+17	155
CPT-12	South Abutment	3/13/2013	+14	196

Table 4
Summary of Subsurface Conditions from Previous Geotechnical Investigation

Location (Explorations)	Depth of Bottom of Fill/Top of Alluvial Deposits	Depth of Bottom of Alluvial Deposits/Top of Del Mar Formation	Groundwater Elevation (ft, MSL)
North Abutment (B-7 & CPT-11)	14 ft	145 ft	+3
South Abutment (B-8 & CPT-12)	12 ft	Not Encountered	+1½

The fill encountered in the Ninyo & Moore explorations reportedly consisted of brown and light gray, very loose to medium dense silty sand with trace amounts of shells, gravel, and asphalt fragments. The alluvium reportedly consists of gray and black, very loose to very dense silty sands with trace amounts of gravel interlayered with soft to very stiff lagoonal silts and clays.

As noted in Table 4, the alluvium extended to the termination depths of the boring and CPT performed near the southern end of the bridge. Ninyo & Moore reported in their report that the alluvium extended to the termination depth of all CPTs performed at the site; however, a cross-section was provided by Ninyo & Moore in their report showing a contact with the underlying Del Mar Formation at approximately 145 feet bgs in CPT-11. Based on our review of the CPT logs provided in the Ninyo & Moore report, our review of other available geologic information in the site vicinity, and the results of our field investigation as discussed in Section 3.2 of this report, we anticipate that the Del Mar Formation was encountered around this depth in CPT-11 location as shown on the Ninyo & Moore cross-section.

The progress and supplemental reports prepared by the Division of Highways in 1960 provide insight on the embankment construction proposed by the State over 50 years ago prior to construction of the Camino Del Mar bridge, indicating deeper fills may be present at the site, particularly near the abutment areas.

The Ninyo & Moore boring and CPT logs, locations of explorations, geologic cross-section, and laboratory test results are provided in Appendix E for reference along with the Division of Highway LOTBs.

2.2 CURRENT FIELD EXPLORATIONS

2.2.1 Rotary Wash Borings

Two rotary-wash boreholes, designated as R-20-001 and R-20-002, were drilled near the existing abutments of the Camino Del Mar Bridge. Borings R-20-001 and R-20-002 were completed using augering techniques in the upper soils and then rotary wash techniques below groundwater and were performed to depths of approximately 151 feet and 208 feet below ground surface (bgs), or to approximate elevations -135 feet and -192 feet, respectively. The drilling was performed by Pacific Drilling Co. between February 10th and February 21st, 2020 using a truck-mounted drill rig equipped with 8-inch outer-diameter hollow stem augers and a 4-inch-diameter tri-cone roller bit.

The boring information including boring locations and depths explored are summarized in Table 5. The geotechnical boring logs are presented in Appendix A and on the Log of Test Borings (LOTBs) in Appendix D and the locations of the borings are presented on Figures 2 and 3. The subsurface conditions encountered during drilling are summarized in Section 3.2 of this report.

Table 5

Boring Summary

Boring No.	Location (Closest Approx. Station)	Completion Date	Hammer Efficiency[1]	Approx. Ground Surface Elev. (ft, NAVD88)	Drilled Depth[2] (ft)	Groundwater Elevation (ft)
R-20-001	North Abutment (167+00)	2/21/2020	94	+16	151	+2
R-20-002	South Abutment (159+00)	2/13/2020	94	+16	208	+5

Notes: [1]Hammer efficiency as provided by Pacific Drilling for Marl Truck-Mounted Rig equipped with an Automatic Hammer.
[2]Drilled depths encountered practical refusal prior to the planned termination depth.

2.2.2 Cone Penetrometer Tests (CPTs)

Four cone penetrometer tests (CPTs), designated as CPT-20-001, CPT-20-002, CPT-20-002A, and CPT-20-003, were performed by Fugro between February 18th to February 21st, 2020. The

CPTs, which include advancement of one seismic CPT (SCPT), were advanced to depths ranging from 16 feet to 200 feet below the ground surface or bridge deck. The CPTs were advanced using a truck-mounted CPT drill rig with a 30-ton push capacity equipped with a 15cm^2 cone-shaped probe attached to cylindrical steel rods instrumented with a cylindrical-shaped friction sleeve and pore pressure transducer. During advancement of the CPTs, the cone tip penetration resistance, friction resistance along the friction sleeve, and pore water pressure were recorded. For the SCPT, shear wave velocity measurements were taken at 5-foot intervals using a cone tip equipped with geophones.

CPT-20-001 and CPT-20-003 were performed near the existing bridge abutments and CPT-20-002 and CPT-20-002A were performed through the bridge deck near the center of the existing bridge. The CPTs performed within the bridge deck required casing to be installed from the bridge deck to below the mud line of the river channel to support the CPT rods. CPT-20-002 was quickly abandoned at 16 feet below the bridge deck after beginning the CPT due to sinking of the casing and CPT rods into the soft, underlying soils in the river channel. Therefore, CPT-20-002A was advanced at the same location as a second attempt to perform the CPT on the bridge deck but refused at a depth of approximately 37 feet below the bridge deck, or at approximate elevation of -21 feet.

Prior to advancement of the CPTs, public and private utility locating was performed and the surficial pavement was cored. The first approximate five feet of the CPTs performed near the bridge deck were advanced by manual hand auger to further clear for underground utilities. Upon completion of the CPTs, the rods were extracted and the surface was patched with either AC near the abutments or concrete within the bridge deck.

The CPT information including CPT locations and depths explored are summarized in Table 6. A detailed description of the CPT methodology, logs of the CPTs, and the SCPT shear wave velocity measurements are presented in Appendix B. Subsurface conditions interpreted from the CPT data are presented in Section 3.2.

Table 6

CPT Summary

CPT No.	Location (Closest Approx. Station)	Completion Date	Approx. Ground Surface Elev. (ft, NAVD88)	Explored Depth[1] (ft)
CPT-20-001	North Abutment (167+00)	2/18/2020	+16	158
CPT-20-002A	Bridge Deck (163+00)	2/21/2020	+16	37
CPT-20-003	South Abutment (159+00)	2/19/2020	+16	200

Notes: [1]Explored depths encountered practical refusal prior to the planned termination depth.

2.3 LABORATORY TESTING

A laboratory testing program was conducted to substantiate field classifications and evaluate selected physical characteristics and engineering properties of the soils encountered. Moisture content, unit weight, Atterberg Limits, sieve analyses, R-value, direct shear, unconfined compression, unconsolidated undrained triaxial compression (TXUU), and corrosion tests were performed in general accordance with the applicable ASTM or Caltrans test methods. Results of the laboratory testing program are presented in Appendix C.

3 GEOTECHNICAL CONDITIONS

3.1 REGIONAL SETTING

In addition to our review of previous and nearby geotechnical reports and LOTBs, our geologic evaluation also consisted of reviewing available aerial photographs, topographic maps, and geologic maps along with observation of the existing site conditions during our subsurface investigation. The results of the evaluation are included in the following sections.

3.1.1 Soil Survey

Based on our review of the United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) web soil survey results (accessed May 2020), the surficial deposits at the site consist of lagoon water (LG-W) underneath the existing bridge, Tujunga Sand (TuB) to the south of the bridge, and tidal flats (Tf) to the north of the bridge.

Tujunga sand is reported to primarily consist of 'somewhat excessively drained' fine sand, gravelly sand, loamy sand, and gravelly loamy sand having a hydrologic soil group A, negligible runoff class, and high to very high infiltration capacity. Tidal flats are reported to have a negligible runoff class but are reported to be Hydrologic Soil Group D and be very poorly drained due to the depth of the water table and frequency of flooding where these are mapped.

3.1.2 Geologic Setting

The site is located within the coastal zone of the Peninsular Ranges Geomorphic Province (Norris and Webb, 1990). This province stretches from northern Los Angeles County to the tip of Baja California and is dominated by mountainous terrane composed of Cretaceous-age igneous rocks of the Southern California Batholith and various Jurassic-age metamorphic rocks. The lower-lying flanks of this basement complex are covered with a variety of younger sedimentary rocks. Within San Diego County, these sedimentary rocks consist of a westward thickening clastic wedge comprised of three sequences of deposits.

The oldest sequence consists of claystone, siltstone, sandstone, and conglomerate deposited during the late Cretaceous time as an apparent submarine fan (Abbott, 1999). These units crop out on Mt. Soledad in La Jolla, Point Loma, and Carlsbad. The second sequence of sediments was deposited during the Tertiary (Eocene and Pliocene) period within an embayment that stretched from northern San Diego County into Mexico (Kennedy, 1975). The sediments consist of a variety of claystone, siltstone, sandstone, and conglomerate. The most recent sedimentary deposits consist of early to late Pleistocene, near-shore marine, estuarine, and delta deposits, also typically identified as terrace deposits. Most of these sediments were deposited on wave cut surfaces (terraces) developed in response to sea level fluctuations during the Pleistocene. The oldest terrace deposits (Qvop), deposited during the early to middle Pleistocene, and the youngest terrace deposits (Qop), deposited during the late Pleistocene, have been mapped throughout the coastal region of San Diego County including in the vicinity of the project site.

During the late Pleistocene, the land surface throughout San Diego County was down-cut and eroded by fluvial processes in response to a world-wide, glacially-induced drop in sea level. This erosional event resulted in the dissected system of east to west flowing drainages and intervening basins that empty into the Pacific Ocean. Near the coast, these drainages were down-cut several hundreds of feet below current sea-level elevations. Near the end of the Pleistocene epoch and continuing up to the present, sea level gradually rose as the continental glaciers receded. This event forced in-filling of the eroded drainages with alluvial sediments which range in age from the latest Pleistocene to recent times. The project site is located within one of these drainages associated with the San Dieguito River. The surrounding highlands to the north and south are comprised of Pleistocene-age old paralic deposits (Qop_6) deposited over Eocene-age sedimentary rocks consisting of the Del Mar Formation (Td) and the Torrey Sandstone (Tt). These deposits are shown on the Regional Geologic Map presented as Figure 4.

3.1.3 Tectonic Setting

California is one of the most tectonically active areas of the United States. The high seismicity of California is attributed to the fact that the state straddles the boundary of two global tectonic plates known as the North American Plate (on the east) and the Pacific Plate (on the west). The main plate boundary fault is defined by the San Andreas fault which crosses through some of the most densely developed areas of both Southern and Northern California. This fault stretches northwest from the Gulf of California in Mexico, through the desert region of the Imperial Valley, crossing the San Bernardino region, and traversing up into northern California, where it eventually trends offshore near San Francisco (Jennings, 1994; Jennings and Bryant, 2010). Within Southern California, the plate boundary is actually a complex system of numerous faults known as the San Andreas Fault System (SAFS) that spans a 150-mile-wide zone from the main San Andreas fault in the Imperial Valley, westward to offshore of San Diego (Powell et al., 1993; Wallace, 1990).

The major faults east of the site (from east to west) include the San Andreas, San Jacinto, and Elsinore faults. Major faults west of the site are all offshore and include the Rose Canyon-Newport-Inglewood, Palos Verdes-Coronado Bank, San Diego Trough, and San Clemente faults

(Kennedy and Welday, 1980). The most dominant zone of active faulting within the San Diego region is the Rose Canyon Fault Zone (RCFZ).

Approximately 49 mm/yr of overall lateral displacement has been measured geodetically as fault slip across these plate boundaries. The Elsinore, San Jacinto, and San Andreas faults combined account for up to approximately 41 mm/yr of the total plate displacement (84 percent), meaning that the remaining 8 mm/yr (16 percent) is accommodated across the offshore faults to the west of the site (Bennett et al., 1996). Studies within the Rose Canyon, east of Mount Soledad, have revealed fault strands that have displaced Holocene soil horizons with slip rates from 1 to 2.4 mm/yr (Rockwell, 2010).

The RCFZ may be part of a more extensive fault zone that includes the Offshore Zone of Deformation and the Newport-Inglewood fault to the north (Grant and Shearer, 2004; Sahakein, et al., 2017), and several possible extensions southward, both onshore and offshore (Treiman, 1993). The RCFZ is composed of predominantly right-lateral strike-slip faults that extend north to northwest through the San Diego metropolitan area towards La Jolla, however, various fault strands display normal, oblique, or reverse components of displacement as well. The fault zone extends offshore at La Jolla and continues north-northwest subparallel to the coastline. To the south in the San Diego downtown area the fault zone appears to splay out into a group of generally right-normal oblique faults extending into San Diego Bay (Treiman, 1993).

The closest fault to the site is the off-shore portion of the Rose Canyon-Newport-Inglewood connected fault located approximately 2.2 miles west of the site. The locations of this and other nearby faults with respect to the site is shown on the regional fault and seismicity map shown on Figure 5.

3.2 SURFACE AND SUBSURFACE CONDITIONS

Geologic units observed in the borings consist of successive strata of recent alluvial deposits, young alluvial deposits, young estuarine deposits, old alluvial deposits, and the Del Mar Formation. The alluvial deposits underly surficial pavement and artificial fill material and overly the Del Mar Formation. The areal extent of these geologic units is depicted on the regional geologic map in Figure 4. Artificial fill soils overlie the alluvial deposits and existing AC pavement caps the fill soils at the surface at the approach embankments on both the north and south sides of the bridge. Detailed descriptions of these units are provided on the boring logs in Appendix A and generalized descriptions are provided in the subsequent sections below. Additionally, the subsurface geologic conditions are also depicted on the geologic cross-section in Figure 6.

3.2.1 Surficial Pavement

Asphalt concrete (AC) was encountered at the surface of all the boring and CPTs performed for our study. The surficial AC was measured to be approximately 5 to 6 inches thick in the borings and CPTs performed near the abutments. At the CPT-20-002/2A location, a 5-inch-thick surficial AC layer was underlain by approximately 12 inches of reinforced concrete associated with the bridge deck.

3.2.2 Artificial Fill (af)

Artificial fill soils were encountered underlying the surficial pavement in the borings and CPTs performed near the abutments. The fill material generally consists of yellowish red, dark yellowish brown, strong brown, and light brownish gray poorly graded sand with variable amounts of silt and trace amounts of gravel. The fill layer extends to depths of approximately 9 feet bgs in boring R-20-001 located near the existing northern abutment, or to approximate elevation +7 feet, and to 8½ feet bgs in boring R-20-002 located near the existing southern abutment, or to approximate elevation +7½ feet. Based on our review of previous plans, these fills were likely placed for the existing bridge embankments and it is possible that deeper fills may be present beyond our exploration locations. Field SPT penetration blow counts (field N-values corrected only for sampler type) of the fill material ranged from 10 to 26 blows per foot (bpf) corresponding to medium dense material.

No earthwork reports were available for our review documenting placement and/or compaction of the encountered fill. Therefore, the existing fill at the site is considered undocumented.

3.2.3 Recent Alluvial Deposits (Qa)

Recent alluvial deposits were encountered underlying the fill materials in the borings and CPTs performed near the existing abutments and were encountered at the ground surface below the bridge deck in CPT-20-002/2A. The recent alluvial materials generally consist of brown, gray, and dark gray silty sand and poorly graded sand with various amount of silt and gravel. An interbedded lean to fat clay layer was encountered within the recent alluvium in boring R-20-001 and CPT-20-001 at the northern end of the existing bridge. This clayey layer likely pinches out towards the south as evidenced by the subsurface conditions encountered in the CPT-20-002/2A and CPT-20-003 and boring R-20-002. This geologic unit is a loose modern alluvial deposit of the San Dieguito River. Field SPT N-values ranging from 2 to 34 bpf for coarse-grained layers and 4 to 8 bpf for fine-grained layers, corresponding to very loose to dense and soft to medium stiff materials. Furthermore, CPT tip resistances in this unit generally ranged from approximately 5 to greater than 200 tsf and field pocket penetrometer values of 0 tsf were observed in the fine-grained samples of this unit. It should be noted that the presence of gravel may result in unreasonably high SPT N-values or tip resistances.

The thickness of the recent alluvial deposits varies at the site with thicker recent alluvium at the northern end of the existing bridge. The recent alluvium extends to a depth of approximately 48 feet bgs in the explorations performed at the northern end of the bridge, or to approximately elevation -32 feet. At the southern end of the existing bridge, the recent alluvium extended to a depth of approximately 30 feet bgs, or to approximate elevation -14 feet. CPT-20-002/2A, performed within the center of the bridge, terminated in the recent alluvial deposits.

3.2.4 Young Alluvial Deposits (Qya)

Middle Holocene-age young alluvial deposits were encountered underlying the recent alluvial deposits in the borings and CPTs performed near the existing abutments. This unit generally consists of dark gray silty sand and poorly graded sand with various amount of silt and trace amounts of gravel and shells and thin interbedded clayey layers. This geologic unit was encountered to be loose to very dense as evidenced by field SPT N-values ranging from 8 to greater than 50 bpf with an average field SPT N-value of 33 bpf, and CPT tip resistances generally ranging from approximately 20 to greater than 300 tsf.

The thickness of the young alluvial deposits was encountered to be approximately 37 feet thick in explorations performed near the northern abutment and approximately 48 feet thick in the explorations performed at the southern abutment. The young alluvium extended to depths of approximately 78 to 85 feet bgs, or to approximate elevations of -62 feet and -69 feet, at the southern and northern abutments, respectively.

3.2.5 Young Estuarine Deposits (Qyes)

Below the young alluvial deposits, a relatively thin layer of Middle to Early Holocene-age young estuarine deposits was encountered in the borings and CPTs performed near the abutments. This geologic unit generally consists of an approximate 6 to 8-feet-thick black and dark gray, low to medium plasticity, lean clay with trace amounts of sand, mica, and shells. The fine-grained conditions encountered in this unit represent a pause in sea-level rise which occurred at the end of the Pleistocene indicating a transition from the young alluvium overlying above and old alluvium below.

The young estuarine deposits extended to a depth of approximately 95 feet bgs in the explorations performed at the northern end of the bridge, or to approximately elevation -79 feet. At the southern end of the existing bridge, the recent alluvium extended to a depth of approximately 94 feet bgs, or to approximate elevation -78 feet. Field pocket penetrometer values of 0.5 tsf were observed in this unit and field SPT N-values in this unit ranged from approximately 8 to 21 bpf, with an average of approximately 12 bpf, indicating stiff to very stiff fine-grained materials. Furthermore, CPT tip resistances generally ranged from approximately 9 to 36 tsf in the fine-grained portions this unit.

3.2.6 Old Alluvial Deposits (Qoa)

Pleistocene-age old alluvial deposits were encountered underlying the young estuarine deposits in the borings and CPTs performed near the existing abutments. The old alluvial materials generally consist of very dark gray silty sand and poorly graded sand with silt. Boring R-20-002 and CPT-20-003 refused and terminated in the old alluvium unit at depths of approximately 200 and 208 feet bgs, or at approximate elevation of -184 feet and -192 feet, as gravel content increased in the old alluvium. In boring R-20-001 and CPT-20-001 performed near the northern abutment, the old alluvium extended to a depth of approximately 146 feet bgs, or to approximate elevation -130 feet. The old alluvial deposits were encountered to be medium dense to very dense as evidenced by field SPT N-values ranging from 10 to greater than 50 bpf, with an average of approximately 30 bpf, and CPT tip resistances generally ranging from approximately 30 tsf to greater than 300 tsf.

3.2.7 Del Mar Formation

The Del Mar Formation is an Eocene-age geologic unit deposited in an ancient lagoonal environment. It was encountered below the old alluvial deposits at the northern end of the bridge in boring R-20-001 and CPT-20-001 at an approximate depth of 146 feet, or approximate elevation -130 feet. This geologic contact is generally consistent with the contact of the Del Mar Formation reported in Ninyo & Moore CPT-11 which was also performed near the northern end of the bridge. The Del Mar Formation was penetrated approximately five feet and was observed in one sample to consist of dark reddish brown with grayish green claystone. This unit is known to also have interbedded sandstone layers. Field SPT N-values of the Del Mar Formation were greater than 50 bpf and CPT tip resistances generally ranged from 100 to 300 tsf corresponding to very dense material.

3.3 UNSUITABLE MATERIALS

3.3.1 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or pavements supported on grade.

Visual classification of the soils near anticipated subgrade elevations indicates that these soils primarily consist of non-plastic poorly-graded sand with small amounts of silt. Based on the results of our field investigation and review of existing information, it is our opinion that the site soils near the ground surface generally have a very low to low expansion potential. Isolated zones of more expansive soil may also be encountered near the surface but are not anticipated.

3.4 SURFACE WATER AND GROUNDWATER

Groundwater was encountered in all the borings and CPTs performed for our field investigation. Encountered groundwater depth ranged from approximately 11 to 14 feet bgs, or at approximate elevations +5 to +2 feet, during drilling. Upon completion of drilling, the groundwater levels were measured to be approximately 17 feet bgs, or approximate elevation -1 foot. It should be noted that the borings were converted into rotary wash upon encountering groundwater. Circulation of water and drilling mud in the boreholes are required as part of the rotary wash drilling. Therefore, water level measurements after completion of the borings may have been influenced by introduction of water and drilling fluids in the boreholes. Also, some rains occurred prior to and during our field investigation and a rise and fall in the water surface level within the San Dieguito River channel was observed.

The Ninyo & Moore borings for this site reported groundwater at depths of approximately 12½ and 14 feet bgs, or at approximate elevations +1½ feet and +3 feet.

Due to the proximity of the site to the coast, groundwater levels are expected to fluctuate due to tidal and seasonal influences. Based on our available information review, we understand that historic minimum and maximum tidal elevations ranges from approximate elevation -3 feet to +7½ feet (NOAA, 2020). The design storm elevation for the project is determined to be +14.55 feet based on the project conceptual plans.

The flood hazard potential for the site was evaluated based on the Federal Emergency and Management Administration (FEMA) Flood Insurance Rate Maps (FIRM). These maps identify those areas that may be subject to special flood events. According to FEMA FIRM 06073C1307H dated December 20, 2019, the site is located within a regulatory floodway flood hazard area with a base flood elevation of 12 feet NAVD 88. Therefore, the hazard at the site with respect to flooding is considered high and flood loads should be considered in the design in accordance with the AASHTO Bridge Design Specifications. We understand that a design flood elevation of +14.55 feet is currently being used for design which corresponds to a 50-year event plus two feet of freeboard.

3.5 SCOUR POTENTIAL

Scour is the loss of ground by erosion in flowing water environments caused by changes in flow volume, flow velocity or flow direction. Scour can occur over the width of the stream or river bed and can be concentrated at locations in which hard protrusions occur in a river bed, such as at bridge piers. The San Dieguito River channel may scour during high flow events and could be impacted by the construction of the new bridge piers and removal of existing piers.

We understand that rip-rap slope protection will be placed at the abutment slopes to protect the abutment from surficial erosion and scour, as is the existing condition at the site. For the bridge piers, we understand that general and local scour have been estimated along the San Dieguito River channel by the project hydraulic engineer. Based on conversations with the hydraulic engineer, we understand that at the deepest part of the river channel (approximately 150 feet south of the northern abutment), general scour elevation at peak flow assuming a 200-year flood event reaches elevation -14.5 feet NAVD88, which is up to approximately 8½ feet below initial bed conditions at this location within the river channel. Scour towards the southern abutment, where the initial bed elevation is higher, is estimated to reach approximate elevation -10 feet NAVD88 during peak flow for the 200-year flood event, which is approximately 16 to 18 feet below initial bed conditions.

For local scour, the 200-year flood event was used to estimate scour depths. Dependent on the bridge option and pier analyzed, local pier scour ranged from 15.7 feet to 21.1 feet in depth. Local pier scour should be added to general scour depth to obtain total scour for design. These scour estimates should be updated once the final bridge option is selected.

3.6 CORROSION POTENTIAL

Preliminary laboratory corrosive soil screening of the on-site soils was performed on samples collected from borings R-20-001 and R-20-002 to evaluate the potential corrosion on concrete and ferrous metals. The results of the testing are presented in Table 7 and included in Appendix C. Furthermore, one laboratory corrosion test was performed on a near-surface sample from Ninyo & Moore boring B-8 performed at the southern end of the bridge. The results from this test are also provided in Table 7 as well as in Appendix E.

Table 7
Preliminary Corrosion Test Results

Boring	Depth (feet)	Minimum Resistivity (ohm-cm)	pH	Water Soluble Sulfates (ppm)	Water Soluble Chlorides (ppm)
R-20-001	0.5 - 5.5	12,000	9.0	42	21
R-20-001	51-51.5	190	9.0	600	2,460
R-20-002	0.5 - 4	13,000	8.7	45	21
R-20-002	126-126.5	85	8.0	870	7,480
B-8 (N&M)	5-6.5	10,000	8.4	40	50

Caltrans Corrosion Guidelines (Caltrans, 2018) considers the subsurface conditions at a site to be aggressive to below-grade concrete if one or more of the following conditions exist for the representative soil samples taken at the site: chloride concentration is 500 parts per million (ppm) or greater, sulfate concentration is 1,500 ppm or greater, or the pH is 5.5 or less. Since resistivity serves as an indicator parameter for the possible presence of soluble salts, it is not included as a parameter to define a corrosive area for structures based on Caltrans Guidelines.

Based on the Caltrans criteria, the near-surface artificial fill soils are considered to be not aggressive to below-grade metals or concrete. However, the natural soils at depth below the groundwater table are considered to be aggressive to below-grade concrete due to the high soluble chloride concentration laboratory test results. Based on these test results and the proximity of the project site to salt water, buried metal and concrete elements should be designed for corrosive conditions in accordance with applicable sections of the AASHTO Bridge Design Specifications with California Amendments and Caltrans Memos to Designers and Standard Specifications.

Preliminary corrosion tests are only an indicator of potential soil aggressivity for the sample tested. We recommend that additional corrosion tests be performed at variable depths and on soil samples taken at additional investigative locations. Furthermore, due to the proximity of the site to the Pacific Ocean and the high groundwater table encountered at the site, we recommend corrosion of below-grade elements should consider corrosive groundwater conditions as well. Corrosion test results should be reviewed and evaluated by the project designers considering the proposed improvements and project lifespan requirements. Kleinfelder does not practice corrosion engineering and the purpose of our tests is only to provide a preliminary screening. A qualified corrosion engineer should be contacted for detailed evaluation of corrosion potential with respect to construction materials at this site and the proposed design.

4 PRELIMINARY SEISMIC INFORMATION AND RECOMMENDATIONS

Kleinfelder has reviewed the site with respect to potential seismic hazards. This evaluation is based on review of available geologic maps, aerial photographs, topographic maps, hazard maps, our geologic site reconnaissance, boring, CPT, and laboratory data, and engineering analyses. Potential seismic hazards considered in our study include surface fault rupture, seismic shaking, liquefaction and seismically induced settlement, lateral spreading, seismic slope instability, and tsunamis. The following sections discuss these hazards and their potential at this site in more detail.

4.1 POTENTIAL SEISMIC HAZARDS

4.1.1 Surface Rupture

As previously discussed in Section 3.1, the subject site is not underlain by any known active or potentially active faults. The closest active fault is the Rose Canyon-Newport Inglewood off-shore fault which is located approximately 2.2 miles offshore to the west of the site. The results of our site reconnaissance and review of historical aerial photography did not reveal indications of faults crossing the project site. Based on this data, it is our opinion that the potential for ground rupture due to faulting at the site is negligible.

4.1.2 Liquefaction and Seismic Settlement

The term liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confinement, depth to groundwater, and the intensity and duration of the seismic ground shaking. Liquefaction is most prevalent in loose to medium dense sandy and gravely soils below the groundwater table but can also occur in non-plastic to low plasticity fine-grained soil.

Based on the guidelines provided for liquefaction evaluation in the Caltrans Geotechnical Manual (Caltrans, 2020), evaluations of potential liquefaction susceptibility based on groundwater level, deposit age, and soil composition were made according to the criteria of Youd et al. (2001), Boulanger and Idriss (2006), and Caltrans' Geotechnical Manual. For CPT analyses, we used the recommendations of Youd et al. (2001) to consider layers with soil behavior type index, I_c<2.6 as potentially liquefiable. It should be noted that based on these criteria, the old alluvial deposits were considered to have a low liquefaction susceptibility based on the age of the geologic deposits.

For layers that met the compositional criteria, liquefaction triggering (factor of safety) analyses were performed using methodologies proposed by Youd et al. (2001) (NCEER, 2001). The analyses utilized both SPT data from our boreholes and tip resistance from our CPTs. In order to perform liquefaction analysis, estimated earthquake magnitude (M_w) and peak ground acceleration (PGA) are needed. Liquefaction analyses were evaluated for a magnitude of 6.63 and a PGA of 0.41g based on Caltrans ARS Online V3.0.1. A groundwater depth of 10 feet was used in our analysis for the explorations performed near the abutments due to potential fluctuations of groundwater level due to tidal influence.

Based on the Liquefaction Evaluation Guidelines in Caltrans' Geotechnical Manual, liquefaction triggering potential was only evaluated for the upper 70 feet and liquefaction-induced volumetric settlements are only reported for induced settlements in the upper 50 feet. It should be noted that there is a potential for liquefaction to occur at deeper depths based on our analyses, however, due to the depths of these deposits and associated overburden stresses, liquefaction at these depths are likely to not result in volumetric surface settlements.

Liquefaction-induced volumetric settlements were estimated using the methods of Tokimatsu and Seed (1987) and Zhang et al. (2002). Based on the methods used, the seismic loading, and the site conditions, the calculated post-liquefaction vertical volumetric settlements within the upper 50 feet of the soil profile generally ranged from 3 to 7 inches.

Another type of seismically-induced ground failure that can occur as a result of seismic shaking is dynamic compaction, or seismic settlement. This phenomenon typically occurs in unsaturated, loose to medium dense granular material or poorly-compacted fill soils. The granular fill soils encountered above the groundwater table at the site were generally found to be in a medium dense condition. We evaluated seismic settlement potential of the existing artificial fill soils using the method of Tokimatsu and Seed (1987). Based on the results of the borings and CPTs and the seismic loading, we calculated seismic compression settlement to be less than approximately 1/3-inch.

The liquefaction and seismic settlement calculations for the borings and CPTs from our field investigation are provided in Appendix G.

4.1.3 Lateral Spreading at Piers

Because the limited height of slopes within the active river channel and configuration of the proposed bridge piers, the static stability of the river channel slopes is not considered consequential to the bridge and therefore was not analyzed.

The active river channel geometry and underlying liquefiable soils present conditions where there is a potential for lateral spreading. Empirical lateral spreading analyses were performed using the results of the borings and CPTs and the methodology by Youd et al. (2002) in accordance with Caltrans Lateral Spreading Guidelines (Caltrans, 2020) and are presented in Appendix G. The methodology is performed for “free field” conditions in which the resistance from the piers are conservatively not modeled which is considered adequate for the type selection stage of design. For the site’s PGA of 0.41g and mean magnitude of 6.63 per Caltrans ARS Online V3.0.1, the analyses indicate median free field lateral spreading displacements of approximately 4½ to 6 feet for sloping conditions within the river channel. These displacements would occur toward the channel, basically along the longitudinal axis of the bridge. It should be noted that the borings and CPTs performed near the abutments were also used to analyze lateral spreading as geotechnical information is limited within the existing river channel.

Lateral spreading toward the channel would be resisted by the proposed foundations and the resulting lateral spreading displacements are expected to be less than a free field condition. Once type selection is finalized, the bridge designer should analyze and develop the restraining forces of the piles at the piers in accordance with Caltrans’ Memo to Designers 20-15 (Caltrans, 2017) so that updated lateral displacements considering the proposed foundations can be estimated. Furthermore, the bridge designer should account for loading on the foundations from lateral spread displacements in accordance with Caltrans’ Guidelines on Foundation and Deformation Loading due to Liquefaction Induced Lateral Spreading (Caltrans, 2013).

4.1.4 Tsunami Hazard

A tsunami is a giant sea wave usually generated by catastrophic displacement on a submarine fault. Tsunamis can travel at speeds of hundreds of miles per hour over distances of thousands of miles. In the open ocean, tsunamis have large wavelengths and are difficult to detect. As the sea wave approaches shore, the wave decreases in wavelength and increases in amplitude (height). Large tsunamis can travel well beyond the normal wave break of the shoreline and can cause damage to near-shore structures. Based on the “Tsunami Inundation Map for Emergency Planning, State of California, County of San Diego, Del Mar Quadrangle,” prepared by the

California Emergency Management Agency, dated June 1, 2009, the project site is located within a mapped tsunami inundation area. Therefore, we anticipate the potential for damage due to a tsunami is considered high for the site.

Furthermore, since the site is located within a half-mile of the Pacific Ocean and is situated below an elevation of +40 feet MSL, tsunami hazard should therefore be considered in the design phase of the project, including potential hydrostatic loads on bridges and retaining walls, in accordance with Caltrans' Memo to Designers 20-13 (Caltrans, 2010). Based on an information request submitted to Caltrans by the design team, we understand that the maximum design wave elevation is +10.7 feet NAVD88 with a maximum design flow velocity of 9.8 ft/s (3 m/s). We understand that these values consider sea level rise to year 2100 which is applicable for tsunami hazard. Although the roadway elevation is above this, this design tsunami wave should be considered in the design of the project structures in accordance with Caltrans standards where applicable.

4.1.5 Seismic Slope Instability

We evaluated the slope stability for yield acceleration and post-liquefaction conditions along the abutment slopes at the project site. Details regarding the slope stability methodology and results are provided in Section 5.3 of this report along with estimated seismic slope displacements.

4.2 SEISMIC SHAKING AND PRELIMINARY SEISMIC DESIGN PARAMETERS

As discussed in Section 3.1.3, the project site is located in a seismically active region. The most significant seismic event likely to affect the project site would be an earthquake resulting from rupture along the offshore Rose Canyon fault, which is located approximately 2.2 miles west of the site.

Based on the results of our field investigation in which we performed a SCPT at the southern portion of the site, the average shear wave velocity in the upper 100 feet (30 meters) of the soil profile, deemed the V_{S30} value, is estimated to be approximately 710 ft/s. This V_{S30} value corresponds to a Soil Profile Type D based on Caltrans Seismic Design Criteria (SDC) V2.0 (Caltrans, 2019). Soil Profile Type D is defined as a stiff soil site with average shear wave velocities within the upper 100 feet of the soil profile between 600 and 1,200 ft/s, an average field standard penetration resistance between 15 and 50 bpf, or an average undrained shear strength between 1,000 and 2,000 psf.

However, as discussed in Section 4.1.2 of this report, there is a high liquefaction hazard at the site and; therefore, Caltrans SDC requires the site be classified as Soil Type F. As required by SDC, a site response analysis must be performed for Soil Type F sites. Thus, we have performed a site response analysis based on the field investigations performed at the project site and the requirements set forth in Caltrans SDC and the results are provided in Appendix F.

5 PRELIMINARY EVALUATIONS AND RECOMMENDATIONS

Geotechnical engineering discussions, conclusions, and preliminary recommendations for the type selection phase of the Camino Del Mar Bridge Replacement project are presented in the subsequent sections. These recommendations are consistent with the guidelines presented in Caltrans' Foundation Report Guidelines (Caltrans, 2017) and cover the preliminary geotechnical recommendations pertinent to the project for the bridge structure, including foundation recommendations and recommendations for the bridge approaches. Preliminary recommendations for geotechnical aspects of the project outside of the immediate bridge footprint are provided in the PGDR.

5.1 MATERIAL STRENGTH PARAMETERS

Material strength parameters were developed for engineering analyses including foundation design, retaining wall design, and global slope stability. Generalized soil engineering parameters and subsurface geometry used in the analyses were developed based on the results of our field and laboratory investigation, review of existing available information, and previous experience in the site vicinity. These generalized parameters were used for characterizing the subsurface at both abutments and pier locations. Ultimate and peak soil strengths were modeled using Mohr Coulomb failure criteria.

For the slope stability analyses, static, post-seismic, and rapid drawdown conditions used ultimate strengths in the analyses. For yield acceleration analyses, peak strengths were utilized for non-liquefiable layers. Liquefiable layers were modeled for yield acceleration and post-seismic conditions using the liquefied undrained residual shear strength in accordance with Caltrans' Memo to Designers 20-15 (Caltrans, 2017). Slope stability strength parameters are presented in Tables 8 and 9 for each case analyzed. Strength parameters used in the pile capacity analyses are provided in the calculations in Appendix G. The extents of each modeled layer are based on the geologic cross section provided in Figure 6 and as shown in the slope stability analysis results in Appendix G.

Table 8

Strength Parameters for Static and Rapid Drawdown Analyses

Soil Description	Model	γ_{total} (pcf)	Static		Rapid Drawdown	
			c' / S_u (psf)	Φ' (deg)	c (psf)	Φ (deg)
Artificial Fill (af) (above water)	Mohr-Coulomb	120	50	33	50	33
Recent Alluvial Deposits (Qa) (Sand)	Mohr-Coulomb	120	50	28	50	28
Recent Alluvial Deposits (Qa) (Clay)	Undrained (Phi=0)	110	400	0	0	18
Young Alluvial Deposits (Qya) (Sand)	Mohr-Coulomb	125	50	32	50	32
Young Estuarine Deposits (Qyes) (Clay)	Undrained (Phi=0)	115	750	0	0	22
Old Alluvial Deposits (Qoa) (Sand)	Mohr-Coulomb	125	50	34	50	34
Old Alluvial Deposits (Qoa) (Dense Sand)	Mohr-Coulomb	130	50	36	50	36
Old Alluvial Deposits (Qoa) (Gravelly Sand)	Mohr-Coulomb	130	0	36	0	36
Del Mar Formation	Mohr-Coulomb	135	4,000	0	0	36

Table 9

Strength Parameters for Yield Acceleration and Post-Liquefaction Analyses

Soil Description	Model	γ_{total} (pcf)	Yield Acceleration Peak Strength		Post-Liquefaction Ultimate Strength		Residual Strength
			c' / S_u (psf)	Φ' (deg)	c' / S_u (psf)	Φ' (deg)	S_r (psf)
Artificial Fill (af) (above water)	Mohr-Coulomb	120	50	34	50	33	-
Recent Alluvial Deposits (Qa) (Sand)	Undrained (Phi = 0)	120	-	-	-	-	450
Recent Alluvial Deposits (Qa) (Clay)	Undrained (Phi=0)	110	400	0	400	0	-
Young Alluvial Deposits (Qya) (Sand)	Undrained (Phi = 0)	125	-	-	-	-	700
Young Estuarine Deposits (Qyes) (Clay)	Undrained (Phi=0)	115	750	0	750	0	-
Old Alluvial Deposits (Qoa) (Sand)	Mohr-Coulomb	125	50	35	50	34	-
Old Alluvial Deposits (Qoa) (Dense Sand)	Mohr-Coulomb	130	50	37	50	36	-
Old Alluvial Deposits (Qoa) (Gravelly Sand)	Mohr-Coulomb	130	0	37	0	36	-
Del Mar Formation	Mohr-Coulomb	135	4,000	0	4000	0	-

5.2 BRIDGE FOUNDATIONS

For preliminary estimating purposes, Cast-In-Drilled-Hole (CIDH) type piles are recommended for the support of piers and abutments for the proposed replacement bridge. Based on the loose soil conditions and shallow groundwater at the site, it is recommended that permanent steel casing having a diameter of one foot larger than the diameter of the CIDH shaft and extending to the top of the Old Alluvium layer, or to approximate elevation -80 feet, be used for construction of CIDH piles. Steel casings are recommended in consideration of shaft constructability, liquefiable deposits, and liquefaction induced downdrag. The final casing tip elevation should be determined once further explorations have been made within the river channel.

As an alternative to CIDH piles, driven steel or concrete piles, including driven Cast-In-Steel-Shell (CISS) concrete piles, driven steel H-piles, driven steel pipe piles, and driven precast concrete piles, were also considered for support of the proposed bridge replacement. However, the ability to support large loads, obstructions during pile driving, localized settlements during to driving vibrations, and noise control were all considered as limitations for driven steel or concrete piles. For instance, based on our refusal of CPT-20-002A on gravels/cobbles within the river channel and the depositional environment of the San Dieguito River, obstructions from potential cobbles may interfere with the ability to drive piles. Additionally, the density of the Old Alluvial deposits and Del Mar Formation and the presence of the current timber piles from the existing bridge may also interfere with the drivability of driven piles. Furthermore, due to the proximity of the site to residential and recreational areas, noise from pile driving will likely be a concern during construction.

For type selection purposes, a summary of the preliminary axial capacity analyses for the abutments and various pier options are provided in the subsequent section below and the results of our pile capacity analyses are provided in Appendix G.

5.2.1 Axial Resistance

The compressive axial capacity analysis for CIDH shafts was completed in accordance with the AASHTO LRFD Bridge Design Specifications with California Amendments (8th Edition) using the computer program SHAFT by Ensoft. A LRFD factor of 0.7 was applied to nominal compressive shaft side resistance for calculating the factored shaft side resistance for the strength limit state. A LRFD factor of 1.0 was used for the extreme event loads. Due to the wet pile installation conditions and anticipated depths of the pile tips, tip resistance is ignored for the preliminary analyses.

Use of partial tip resistance based on load-settlement calculations can be considered during the detailed design phase if additional resistance is required. If substantial tip resistance is necessary for the proposed piles, post-grouting of the pile tips and load testing should be considered. Post-grouted piles are recommended to be load tested using the Osterberg Load Cell™ or Statnamic™ techniques to verify the end bearing resistance. The possible need for end bearing and the economic and construction feasibility of tip post-grouting and load testing should be discussed by the project design team and the client once the final bridge type is selected. If tip resistance is necessary, Kleinfelder can provide these values in the final foundation report.

Based on the request of the bridge designer, we have performed compressive axial pile capacity analyses for 5-ft-diameter CIDH shafts with 6-ft-diameter permanent steel casings for support of the proposed abutments. For the piers, we performed compressive pile capacity analyses for 6-ft-diameter, 9-ft-diameter, and 10-ft-diameter CIDH shafts each having permanent steel casings one foot larger than the shaft. Based on discussions with the project team, we understand that adequate space to provide pile spacings of at least three diameters on-center may be provided for these pile options with the exception of the 6-ft-diameter CIDH shafts for support of the piers. For this option, we have considered group effects using a pile spacing of 2.8 diameters on-center.

Furthermore, due to the use of smooth-walled permanent steel casing which is recommended to be installed by an oscillator/rotator, the geotechnical side resistance of the cased portion of the CIDH pile is ignored in accordance with Caltrans' Guidelines for Foundation Reports for Bridges (Caltrans, 2017). As the permanent casing is proposed to extend below the potentially liquefiable layers at the site and considering the annular space between the casing and the pile, downdrag loads due to liquefaction are not expected to be transferred to the pile. Therefore, liquefaction induced downdrag were also ignored. Scour was considered in the analysis; however, the depth of the permanent steel casing extends below the estimated total scour elevations.

Preliminary CIDH axial pile capacity charts as a function of depth for the abutment piles and various pier pile alternatives are provided in Appendix G. Appendix G also includes tables of the soil engineering properties used in the capacity analyses as well as the SHAFT output files.

Preliminary compression loads provided by the bridge engineer for the Type Selection Phase generally range from 2,000 kips to 3,800 kips per pile at the piers and 1,000 kips to 1,100 kips per pile at the abutments, dependent on the bridge option. Based on these preliminary compressive loads, the preliminary pile tip elevations for type selection purposes of the various bridge options are anticipated to range from -167 ft to -172 ft at the piers and -129 ft to -132 ft at the abutments. Once final loads are determined for the selected bridge type, final pile tip elevations will be established.

5.3 ABUTMENTS

Based on conversations with the design team, we understand that the abutments at the north and south abutment will be non-integral seat-type abutments and will extend to approximate elevations +7 feet and +5½ feet, respectively. Preliminary recommendations for the abutment walls are provided in the subsequent sections. These recommendations should be updated once the final abutment configurations have been established.

5.3.1 Lateral Earth Pressures

We recommended that the abutments be backfilled with Caltrans structural backfill and be designed with the following soil parameters per Caltrans Standard Plans (Caltrans, 2018) and Caltrans Memo to Designers 5-5 (Caltrans, 2014):

- Total unit weight: 120 pcf
- Internal friction angle: 34°
- Cohesion: 0 psf
- Horizontal seismic coefficient: 0.2g
- Vertical seismic coefficient: 0.0g

The abutments backfilled with Caltrans structural backfill should be designed to resist an active equivalent fluid earth pressure of 35 pcf for drained conditions and 80 pcf for undrained conditions. The active equivalent earth pressure assumes the wall is free to rotate at least 0.002 times the height of the wall to mobilize the active condition. If the abutments are restrained against movement at the top, they should be designed using the at-rest equivalent fluid earth pressure of 55 pcf for drained conditions and 90 pcf for undrained conditions. These pressures include hydrostatic pressure for undrained conditions and assume level backfill conditions.

Resistance to horizontal loadings can be developed by passive earth pressure using a factored equivalent fluid weight of 450 pcf for drained conditions and 250 pcf for undrained conditions (includes hydrostatic pressure). These passive pressures include a resistance factor of 0.5 in accordance with AASHTO LRFD BDS. The passive pressure should be ignored within the scour/erosion depth of the abutment if applicable.

We preliminarily recommend a design groundwater elevation of +6 feet for design of the retaining walls assuming adequate drainage is provided above this elevation. If the bottoms of the abutment walls are designed to bear at an elevation lower than +6 feet, or if adequate drainage is not provided above this elevation, then the abutment walls should also be designed for hydrostatic pressures. Surcharge pressures (dead or live) should be added to the lateral pressures where such loads (e.g., traffic) may occur adjacent to the wall and should be estimated by multiplying the surcharge load by the active earth pressure coefficient (K_a) of 0.29 or the at-rest earth pressure coefficient (K_o) of 0.45, whichever is applicable.

Seismic wall pressures were estimated using the Monokobe-Okabe method (AASHTO, 2017) assuming level backfill conditions. Based on the design peak horizontal ground acceleration of 0.2g per Caltrans standards, the resultant seismic force (in pounds) for each linear foot of wall in terms of equivalent fluid weight can be estimated as $17.5H^2$ pcf within Caltrans structural fill soils, where H is the height of the wall (in feet) above its base. The resultant seismic force acts at 0.5H above the wall base in accordance with Caltrans MTD 5-5. This dynamic incremental earth pressure should be added to the static active earth pressure.

5.3.2 Slope Stability and Seismic Slope Displacements at Abutments

Limit equilibrium slope stability analyses were performed at the abutments using the computer program Slope/W by Geo-Slope International. Spencer's method of slices was selected for the slope stability analyses as this method satisfies both moment and force equilibrium. The analysis employed circular critical slip surface search routines. Analyses were performed for short-term and long-term static conditions, post-seismic conditions, and rapid drawdown conditions. The required minimum factors of safety for each condition are provided in Table 10. Analyses were also performed for yield acceleration (seismic) conditions in which seismic coefficient values corresponding to a factor of safety of 1.0 were determined in order to estimate seismic slope displacements at the abutments.

Table 10

Required Minimum Factors of Safety for Global Stability Analyses

Analysis Type	Minimum Factor of Safety
Static Short-Term	1.3
Static Long-Term	1.5

Table 10 (Continued)
Required Minimum Factors of Safety for Global Stability Analyses

Analysis Type	Minimum Factor of Safety
Post-Liquefaction	1.1
Rapid Drawdown	1.1

The slopes at the abutments were modeled without the abutment wall to analyze short-term construction conditions. Long-term static conditions were modeled with the abutment wall. The effect of restraining forces provided by the proposed pile foundations are not included in the current analyses and, therefore, they are considered “free-field” analyses.

For all cases, a 250 psf surcharge load was modeled to account for traffic loading. Liquefied strengths were used in the yield acceleration and post-liquefaction models in accordance with Caltrans guidelines. No seismic loading (i.e. no horizontal seismic coefficient, k_h) was applied in the post-liquefaction analyses assuming liquefaction occurs subsequent to seismic ground shaking. For the yield acceleration analyses, seismic loading was applied so that the resulting factor of safety is equivalent to 1.0. This seismic loading is designated as the yield coefficient, k_y, and was used along with the dynamic characteristics of the slope/slide mass and design spectral accelerations to estimate the residual seismic slope displacements.

For the rapid drawdown condition, the site’s design storm water level of approximate elevation +14.55 feet was used as the pre-drawdown groundwater condition. Rapid drawdown was performed to model drawdown of the surficial water within the river channel to the ground surface. Using these conditions for our analyses, Table 11 summarizes the resulting critical slope stability safety factors for the static analyses performed and Table 12 summarizes the yield coefficient results.

Table 11
Preliminary Results of Static Slope Stability Analyses

Case		Critical Factor of Safety
Static Short-Term	North Abutment	1.616
	South Abutment	1.515
Static Long-Term	North Abutment	1.861

Table 11 (Continued)

Preliminary Results of Static Slope Stability Analyses

Case		Critical Factor of Safety
	South Abutment	3.014
Post-Liquefaction	North Abutment	1.202
	South Abutment	1.363
Rapid Drawdown	North Abutment	1.480
	South Abutment	2.195

Table 12

Preliminary Results of Yield Acceleration Slope Stability Analyses

Case		Yield Coefficient
Yield-Acceleration	North Abutment	0.049g
	South Abutment	0.072g

Based on the critical safety factors as provided in Table 11, the proposed abutment slopes appear to have adequate static short-term and long-term stability and adequate stability against rapid drawdown using the design storm water elevation. Furthermore, using the residual strength parameters, the proposed abutments also appear to have adequate post-liquefaction stability against flow failure.

For the yield acceleration condition, seismic slope displacements were estimated at the abutments using the yield coefficient values shown in Table 12, the site's seismic design response spectrum as provided in Appendix F, and the method of Bray and Travasarou (2007) in accordance with Caltrans' Memo to Designers 20-15 (Caltrans, 2017). Based on the Bray and Travasarou (2007) method and the site's seismic parameters, the median seismic slope displacements were estimated to be approximately 2 feet at the southern abutment and 3.2 feet at the northern abutment during design level ground shaking. The spectral accelerations for the design level ground shaking used in the seismic slope displacement estimates were based on the preliminary acceleration response spectrum as discussed in Section 4.2 of this report. It should be noted that there is inherent uncertainty in seismic slope displacement estimates. The likely range of displacements is approximately one-half to two times the median value.

Displacements of this magnitude may or may not require mitigation. Slope displacements are expected be resisted by the proposed foundations at the abutments and the resulting displacements would be less than the estimates for a free-field condition. If these slope displacements are too large to currently design to, and once type selection is finalized, the bridge designer should analyze and develop the restraining forces of the piles at the abutments in accordance with Caltrans' Memo to Designers 20-15 (Caltrans, 2017) so that updated lateral displacements considering the proposed foundations can be estimated.

The various slope stability models, resulting failure surfaces and critical safety factors, and the seismic slope displacement calculations are provided in Appendix G.

5.4 BRIDGE APPROACHES

5.4.1 Approach Fill Settlement

Based on preliminary civil plans and discussions with the project team, we understand that several approach fill profiles are currently under consideration at this phase of the project. Based on the conceptual grading profiles for the bridge approaches and the subsurface conditions at the site, we anticipate minimal static settlement due to the new approach fills. Furthermore, due to the granular soils within the zone of influence below the approach embankments, a majority of the static settlements are anticipated to occur during fill placement and compaction and very little static settlement is expected after construction activities. Once the final grading profiles have been established, bridge approach settlements due to placement of new approach fills should be evaluated.

Liquefaction-induced settlements of between 3 and 7 inches along the approaches may occur during a seismic event as discussed in Section 4.1.2 of this report. This should be considered a maintenance issue and should be included as an item for post-earthquake inspection and repair.

5.4.2 Approach Retaining Walls

At this phase of the project, we understand that the retaining wall type and dimensions have not yet been determined. However, based on conversations with the design team, we understand that consideration is being given for a Caltrans Standard reinforced concrete cantilever type wall along the bridge approaches. We have provided preliminary design recommendations for this Caltrans Standard type wall anticipated along the bridge approaches. These recommendations should be updated once the final wall type and dimensions have been established.

5.4.2.1 Foundations

Retaining walls may be supported on shallow continuous foundations supported on improved ground to mitigate liquefaction and lateral spreading or on deep foundations designed to resist lateral spreading forces.

Retaining walls with shallow continuous foundations designed in accordance with the Caltrans Standard Plans (Caltrans, 2018) were evaluated for bearing resistances. Bearing resistances are expected to meet the load demands provided wall foundations are supported on at least 3 feet of compacted Caltrans Standard fill as the friction angle for Caltrans Standard fill and the existing fill soils are estimated to be greater than 30 degrees and the PGA at the site is less than 0.6g.

However, due to the underlying liquefiable deposits and estimated liquefaction induced settlements of 3 to 7 inches in addition to about 21 to 30 inches of settlement associated with the seismic displacement of slopes (about two-thirds of estimated seismic slope displacements), ground improvement or deep foundations are recommended for the support of retaining walls to be able to resist the liquefaction induced displacements.

Ground improvement methods to increase the strength of the soil susceptible to liquefaction and lateral spreading may include deep soil mixing, compaction grouting, jet grouting, vibro-compaction, or rigid inclusions. We anticipate the depth of ground improvement should likely be deeper than a minimum of 30 feet to mitigate for lateral spreading potential.

Should shallow foundations with ground improvement be used for support of approach retaining walls, foundations should have a minimum width in accordance with Caltrans Standard Plans and based on the structural stability analyses performed by the wall designer. Retaining wall foundations should be embedded at least 1½ feet below the lowest adjacent grade or to the depth necessary to provide adequate factors of safety against sliding and overturning as determined by the retaining wall designer, whichever is greater. Shallow foundations for retaining walls embedded near slopes may also experience a reduction in bearing capacity and should be embedded deep enough to provide an adequate daylight distance to nearby slopes.

Alternatively, deep foundations such as CIDH piles can be considered for the support of retaining walls. The piles should be sized to provide adequate restraining forces against lateral spreading displacements. If deep foundations are selected to mitigate the liquefaction and lateral spreading hazard for the approach retaining walls, the structural designer should analyze and develop the restraining forces of the piles supporting the retaining walls in accordance with Caltrans' Memo to Designers 20-15 (Caltrans, 2017) so that updated lateral displacements and associated vertical settlements considering the proposed foundations can be estimated. Based on the anticipated approach fill and retaining wall heights, embedment depth of the piles are expected to be in the range of 30 to 40 feet to mitigate lateral spreading hazard for the retaining walls.

Once the final wall type and dimensions have been determined, foundation options should be re-evaluated and designed accordingly per Caltrans standards considering the seismic hazards at the site.

5.4.2.2 Retaining Wall Global Stability

Based on discussions with the project team and the conceptual plans, we understand that various grade profiles and wall dimensions are currently under consideration and that final grades for the embankment slopes extending from the roadway to the beach areas along the embankments have not yet been determined. We anticipate the global stability of the proposed Caltrans Standard walls will be considered stable with the use of ground improvement and adequate foundation embedment, however, analyses should be performed to confirm the global stability of the wall system once the final civil profile, slope grades, and wall type and dimensions have been determined.

We have performed limit equilibrium slope stability analyses for the bridge abutment walls/slopes as provided in Section 5.3 of this report. These slope stability analyses the critical stability sections of the wall/bridge system.

5.5 TEMPORARY EXCAVATIONS AND SHORING

5.5.1 Temporary Piles

We understand that temporary trestle bridges are proposed for installation of the CIDH piles for the replacement bridge. The trestle bridges will need to be supported on temporary piles capable of supporting construction loads from the pile drill rig, oscillator, crane, etc. We understand that the trestle bridge piles have not yet been selected but are being considered for cost estimates during the type selection phase.

Possible pile types suitable for supporting the temporary trestle bridges include driven steel H-piles or pipe piles as well as driven CISS concrete piles. Driven piles have the advantage of fast installation times, are generally cost effective, can be installed to relatively deep depths, and generally generate minimal spoils. However, noise control is a concern for driven piles due to the proximity of the site to residential areas. Localized settlements of the loose, granular soils supporting nearby structures during pile driving is also a concern while keeping existing structures in service dependent on the location of the temporary piles relative to the existing bridge. Furthermore, based on our refusal of CPT-20-002A on gravels/cobbles within the river channel and the depositional environment of the San Dieguito River, obstructions from potential cobbles may interfere with the ability to drive piles for the temporary trestle bridges.

As noise control is necessary during installation of temporary piles and driving ability due to potential settlement issues and obstructions are concerns for the temporary trestle bridge piles, helical or screw piles may also be considered. These piles consist of steel and are augered/screwed into the ground. They are ideal for temporary structures as they are easily removed by reversing the installed rotation process. However, helical/screw piles typically have shallower installation lengths and lower capacities which may result in a higher number of required temporary piles.

Temporary piles for trestle bridges are typically designed by the contractor. Kleinfelder should review any temporary pile designs to confirm adequate safety factors are achieved for the selected temporary piles based on the construction loads.

5.5.2 Temporary Shoring

Based on conversations with the design team, we understand that portions of the existing abutments will remain in place to help with temporary excavation support, in addition to scour protection. Additional temporary shoring support may be required for construction of the abutments or possibly for installation of the embankment retaining walls or relocation of existing utilities.

Temporary shoring design should be performed by a Registered Professional Engineer. Temporary shoring may consist of sheet piles, shore boxes, soldier piles and lagging, secant piles, etc. If excavations extend below the groundwater table, the shoring design should consider possible dewatering methods or, if dewatering is not possible, the shoring type should be selected and designed to resist hydrostatic pressures.

Where shoring or excavation is performed next to sensitive structures or utilities, instrumentation should be installed to monitor displacements. The final shoring designs should be reviewed by Kleinfelder.

5.6 ADDITIONAL GEOTECHNICAL INVESTIGATIONS

The recommendations provided in this preliminary geotechnical design report are based on the currently available preliminary plans, our available information review, geotechnical field investigation, and our understanding of the proposed project. We recommend that an additional geotechnical investigation be completed at the site once the final bridge type has been selected and the alignment design plans, profiles and cross-sections are developed. Depending on the location and height of fills, retaining walls, bridge piers and abutments, and any other improvements, additional explorations may be required. Based on any additional explorations, our preliminary observations and recommendations should be updated, and final geotechnical recommendations should be prepared for the project. We recommend the additional geotechnical investigation should include the following:

- Additional exploratory borings and/or CPTs located at each proposed pier location. The additional explorations should be advanced deep enough to appropriately evaluate the subsurface conditions for purposes of foundation design based on the preliminary foundation recommendations provided in the PFR. It should be noted that a CPT was attempted at the central portion of the existing bridge and early refusal on gravel and cobbles was encountered. This, along with environmental and permitting restrictions, should be considered during the planning of future explorations within the river channel.
- Additional laboratory testing of collected soil samples to provide final geotechnical design parameters for proper foundation, abutment wall, approach retaining wall, and pavement design.

Final geotechnical design analyses should be completed for the Camino Del Mar Bridge Replacement project in order to provide recommendations for the finalized bridge type and configuration. The analyses should be conducted to confirm our preliminary recommendations and provide updated recommendations in a final foundation report.

6 LIMITATIONS

This report has been prepared for the exclusive use of the City of Del Mar and their consultants for specific application to the design and construction of the Camino Del Mar Bridge Replacement project. The findings, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No warranty, express, or implied is made.

The scope of services was limited to the field exploration program described in this report. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining the level of service necessary to provide information for their project at an acceptable level of risk. The client and key members of the design team should discuss the issues addressed in this report with Kleinfelder so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk, and expectations for future performance and maintenance.

Conclusions and recommendations contained in this report are based on our field observations and subsurface explorations, laboratory tests, engineering analyses, and our understanding of the proposed construction. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, then the client is responsible for ensuring that Kleinfelder is notified immediately so that we may re-evaluate the conclusions and recommendations of this report. If the scope of the proposed construction, or locations of the improvements, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid until the changes are reviewed and the conclusions of this report are modified or approved in writing by Kleinfelder.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder should be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including but not limited to site preparation, preparation of foundations, and placement of engineered fill. These services provide Kleinfelder the opportunity to observe the actual soil and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, then Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinions, recommendations, or conclusions contained in the report. Due to the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder can be contacted to confirm those conditions. We recommend contingency funds be reserved for potential problems during earthwork and foundation construction.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year from the date of the report. Land use, site conditions (both on and off site), or other factors may change over time and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

Our geotechnical scope of services for this subsurface exploration and preliminary geotechnical design report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances at this site. Kleinfelder will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site or from the discovery of such hazardous materials. Additional important information about this report is presented in the attached Geotechnical Business Council insert in Appendix H.

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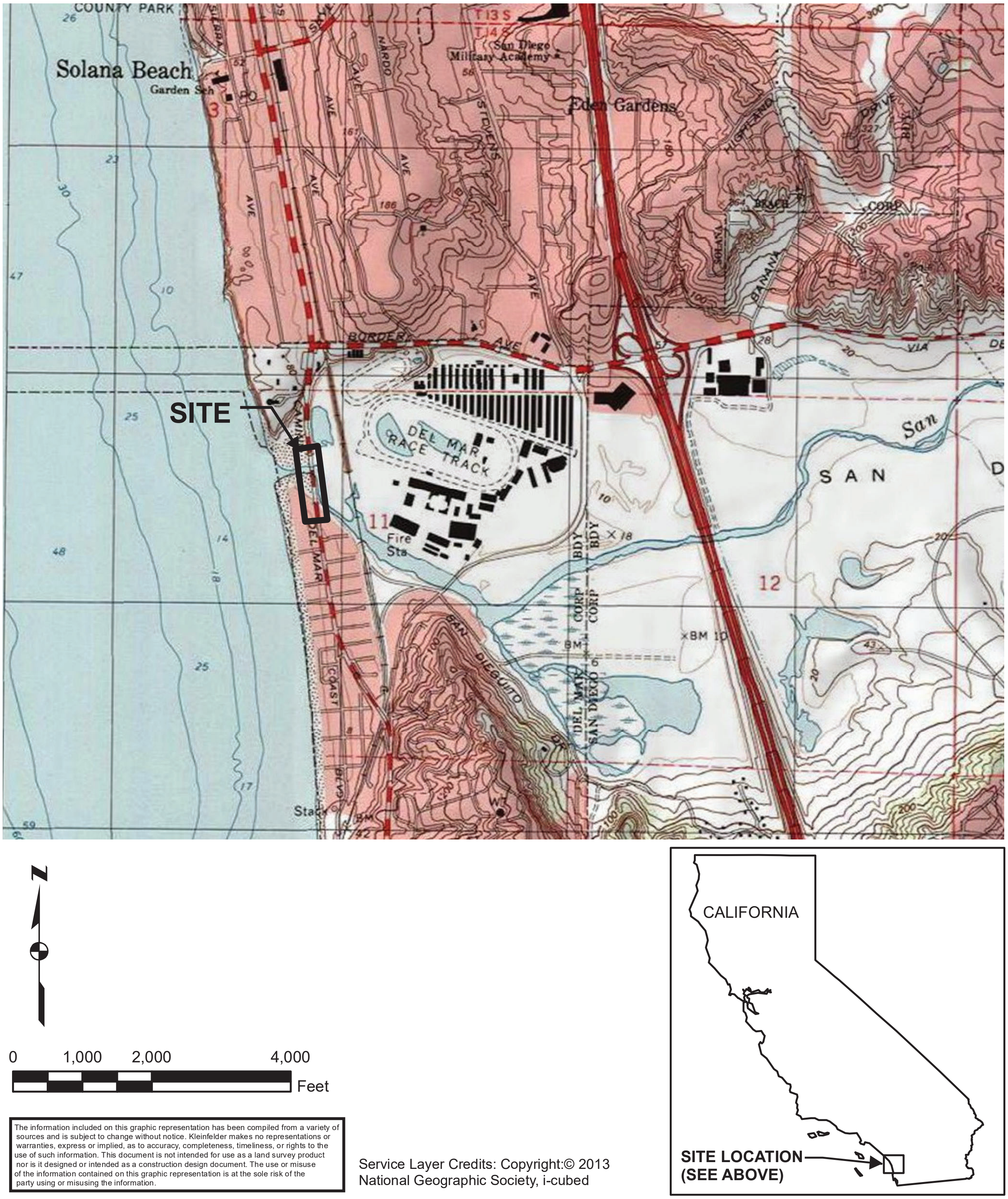
FIGURES



Service Layer Credits: Copyright:© 2013
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\\sandiego\swe-data\G\Bridge Division\Job Files\03 FY2018\0876 - Camino Del Mar Bridge Replacement\Phase B- PAED\Geotech\Deliverables\Figures\Figure 1_Vicinity Map.mxd

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PROJECT NO.	20180876
DRAWN:	12/26/2019
DRAWN BY:	JB
CHECKED BY:	KR
FILE NAME:	Figure 1_Vicinity Map.mxd

SITE VICINITY MAP

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

1

PLOTTED: 4/7/2020 12:15 PM BY: janna bonfiglio

LAYOUT: Layout1 (2)

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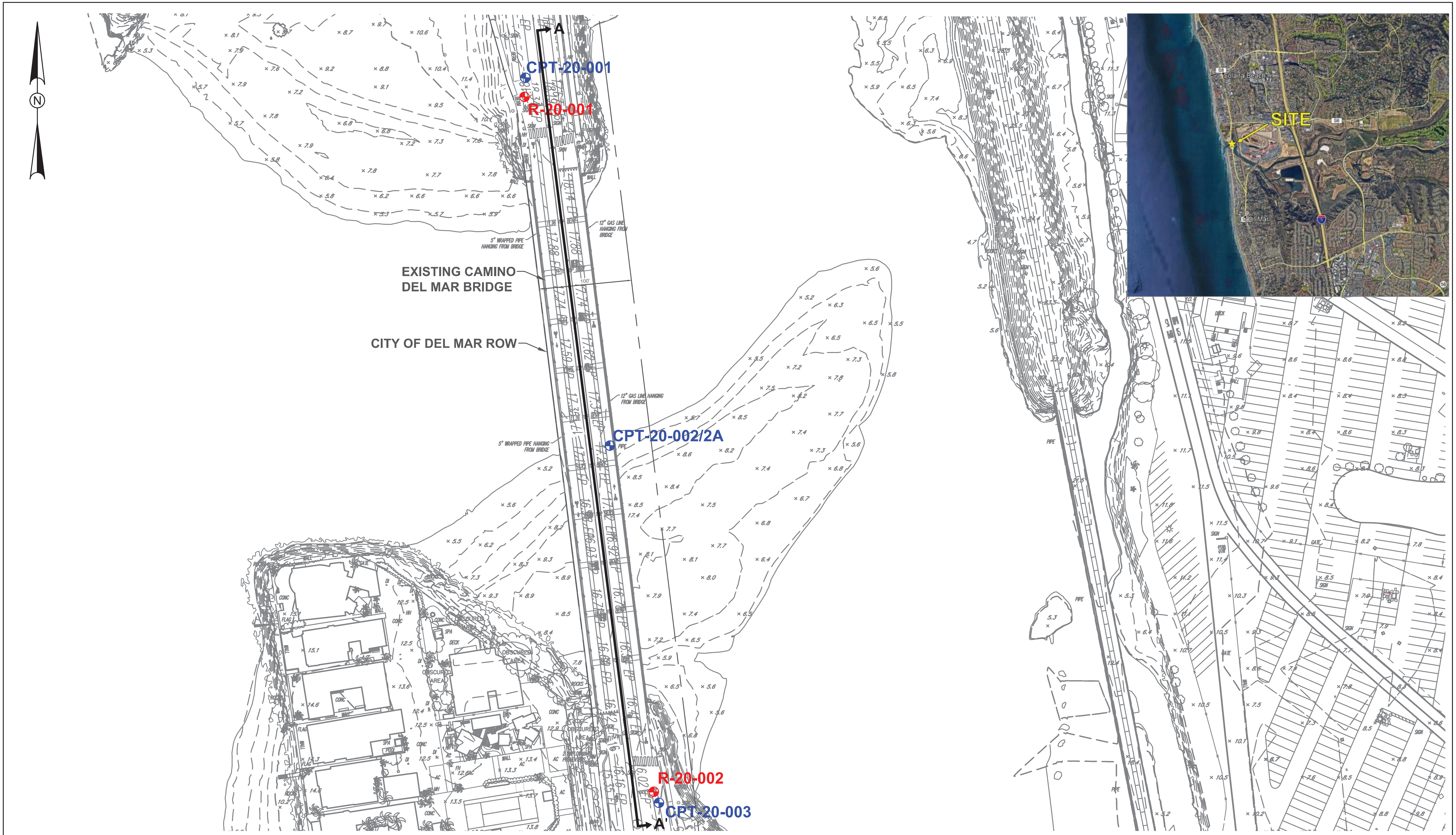

NOTES:

1. EXISTING CONDITIONS REFERENCED FROM THE TOPOGRAPHIC SURVEY PREPARED BY SAMPO ENGINEERING, INC. DATED APRIL 13, 2018.

SCALE: 1" = 100'

100 50 0 100 200

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DATE:	04/2019
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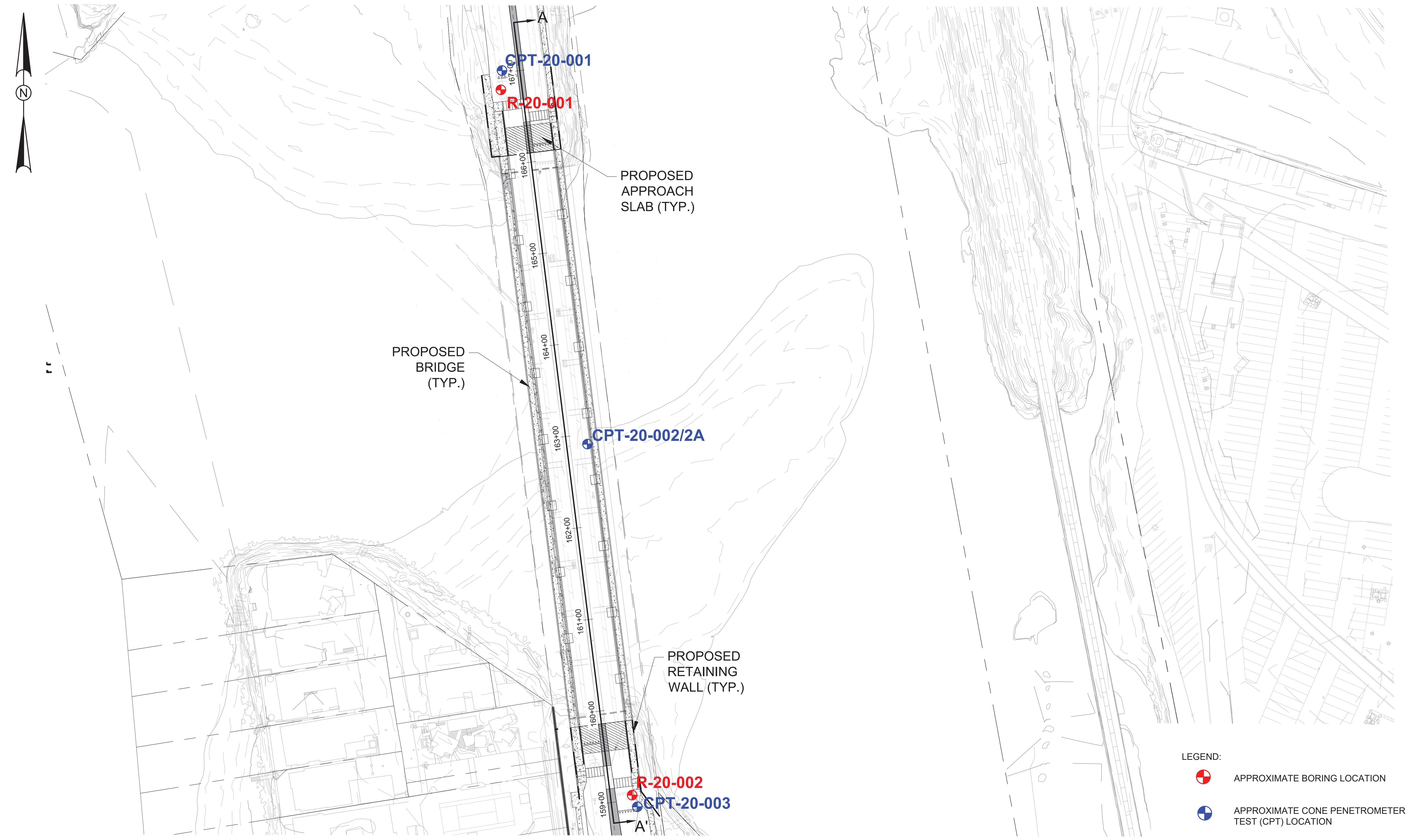
SITE AND PHASE 0 EXPLORATION LOCATION PLAN

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

2



NOTES:

1. EXISTING CONDITIONS REFERENCED FROM THE TOPOGRAPHIC SURVEY PREPARED BY SAMPO ENGINEERING, INC. DATED APRIL 13, 2018.
2. PROPOSED CONDITIONS REFERENCED FROM THE SITE PLAN PREPARED BY KLEINFELDER, INC. DATED MAY 19, 2020.

SCALE: 1" = 100'

100 50 0 100 200

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PROJECT NO. 20180876
DRAWN BY JLB
CHECKED BY KR
DATE: 05/2020
REVISED: -

PROPOSED CONDITIONS AND PHASE 0
EXPLORATION LOCATION PLAN

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
3

PAGE: 1 of 1

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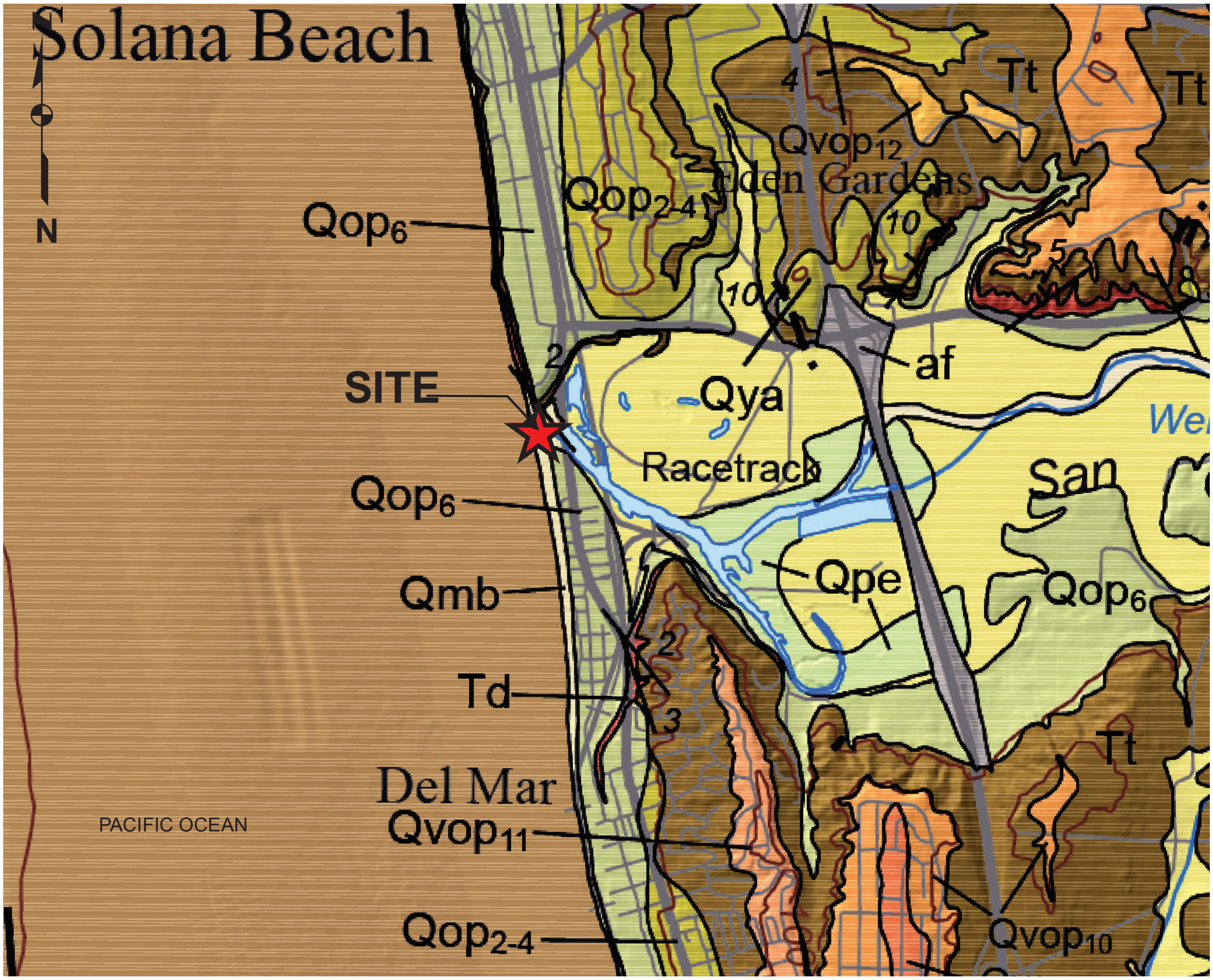

LEGEND

- ▨ af - ARTIFICIAL FILL
- ▨ Qmb - MARINE BEACH DEPOSITS
- ▨ Qya - YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS
- ▨ Qop$_6$ - OLD PARALIC DEPOSITS, UNDIVIDED, UNIT 6
- ▨ Qop$_{2\text{-}4}$ - OLD PARALIC DEPOSITS, UNDIVIDED, UNITS 2-4
- ▨ Qop$_{11}$ - OLD PARALIC DEPOSITS, UNDIVIDED, UNIT 11
- ▨ Qop$_{10}$ - OLD PARALIC DEPOSITS, UNDIVIDED, UNIT 10
- ▨ Qop$_{10a}$ - OLD PARALIC DEPOSITS, UNDIVIDED, UNIT 10a
- ▨ Qop$_{12}$ - OLD PARALIC DEPOSITS, UNDIVIDED, UNIT 12
- ▨ Td - DELMAR FORMATION
- ▨ Tt - TORREY SANDSTONE

0 1,000 2,000 4,000 6,000 Feet

SOURCE:
GEOLOGIC MAP OF THE SAN DIEGO 30' X 60' QUADRANGLE, CALIFORNIA, BY KENNEDY AND TAN, 2008.

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DRAWN BY:	JP
CHECKED BY:	JB
FILE NAME:	Geo Map.mxd

REGIONAL GEOLOGIC MAP

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
4

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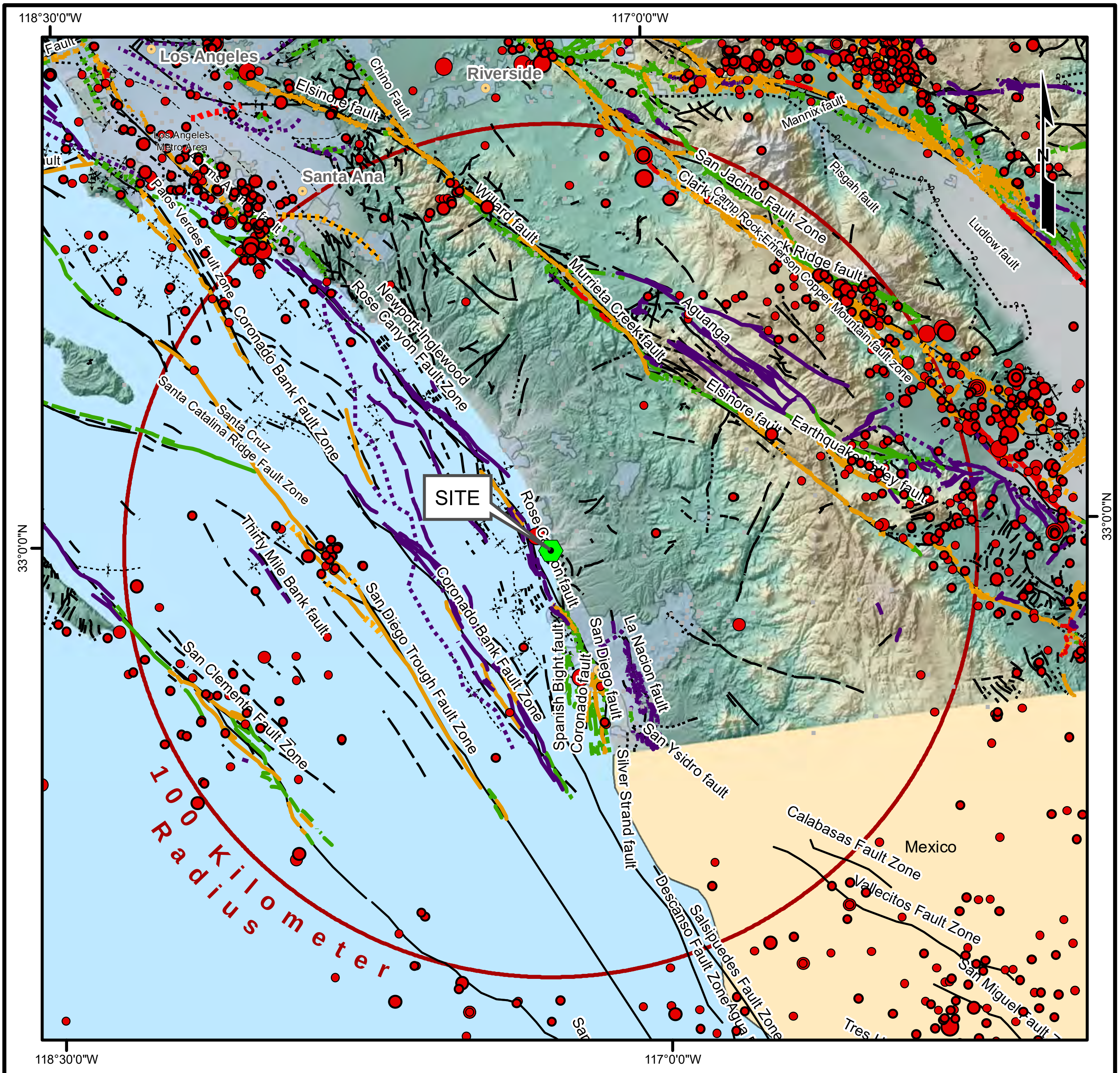

Faulting Legend

Quaternary Faults (Bryant, 2005; USGS, 2018)

Historic displacement (< 150 years)
- 🟥 Mapped Fault Location
- 🟥 Dashed were Approximated
- 🟥 Concealed

Holocene displacement (< 15,000 years)
- 🟧 Mapped Fault Location
- 🟧 Dashed were Approximated
- 🟧 Concealed

Late Quaternary displacement (< 750,000 years)
- 🟩 Mapped Fault Location
- 🟩 Dashed were Approximated
- 🟩 Concealed

Quaternary & unspecified displacement (< 1,600,000 years)
- 🟪 Mapped Fault Location
- 🟪 Dashed were Approximated
- 🟪 Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- — - fault, approx. located
- —?- fault, approx. located, queried
- — fault, certain
- ······ fault, concealed
- ?······ fault, concealed, queried
- —?- - fault, inferred, queried

ANSS Earthquakes

Magnitude
- ● 4.0 - 4.9
- ● 5.0 - 5.9
- ● 6.0 - 6.9
- ● 7.0 - 7.9
- ● 8.0 - 8.9

35 17.5 0 35
Kilometers

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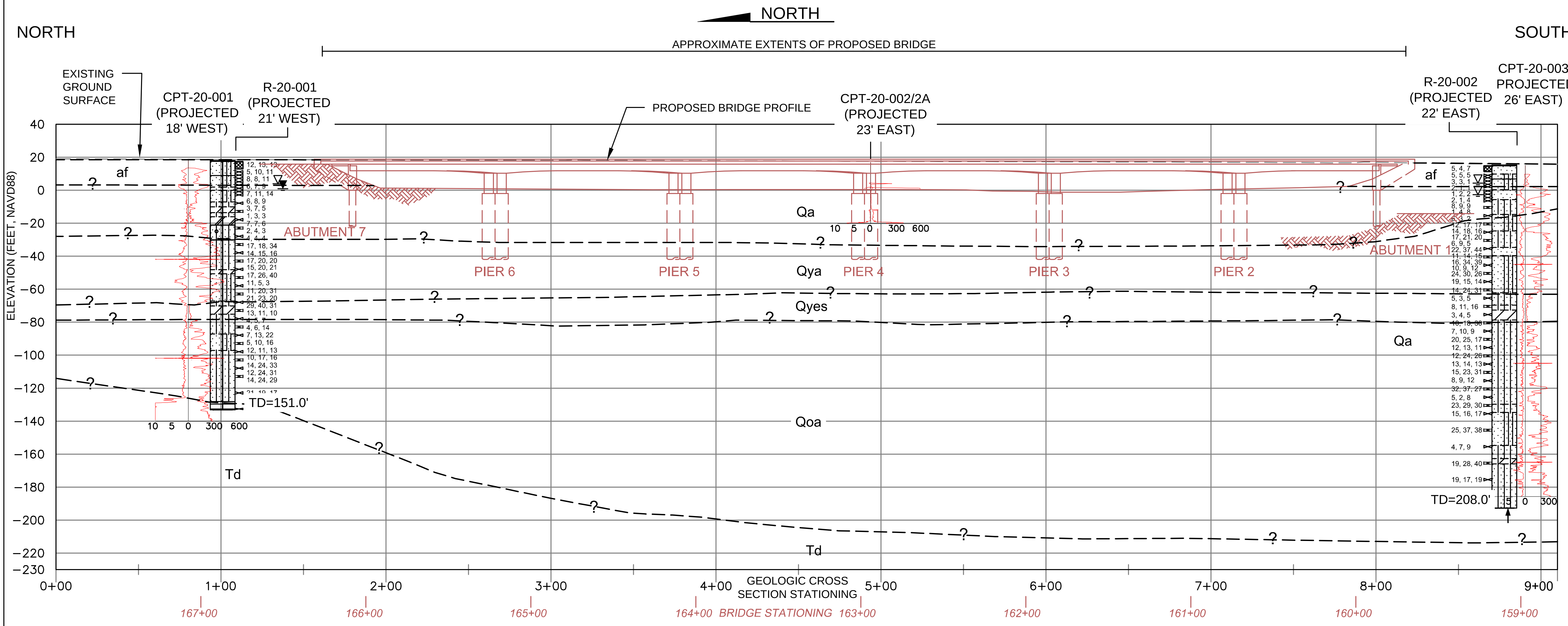
DRAWN: 12/26/2019
DRAWN BY: JB
CHECKED BY: KR
FILE NAME: EQ Map.MXD

REGIONAL FAULT MAP AND EARTHQUAKE EPICENTERS (1800 - JULY 2019)

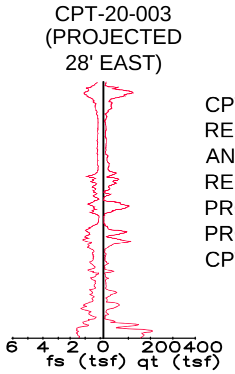
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
5

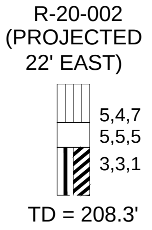


LEGEND

af	ARTIFICIAL FILL
Qa	RECENT ALLUVIAL DEPOSITS
Qya	YOUNG ALLUVIAL DEPOSITS
Qyes	YOUNG ESTAURINE DEPOSITS
Qoa	OLD ALLUVIAL DEPOSITS
Td	DEL MAR FORMATION
— — —?— — —	APPROXIMATE GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN



CPT LOCATION WITH TIP RESISTANCE (RIGHT SIDE) AND FRICTION RESISTANCE (LEFT SIDE) PROFILES, SHOWING PROJECTION AND TOTAL CPT DEPTH



BORING LOCATION WITH GRAPHIC LOG AND N-VALUES, SHOWING PROJECTION AND TOTAL BORING DEPTH (N = Blows/Foot for both Cal and SPT samples - see Boring logs in report for sample types and description of graphic symbols)

REFER TO FIGURES A-1 AND A-2 IN APPENDIX A FOR FURTHER EXPLANATION OF BORING LOGS

∇	GROUNDWATER LEVEL DURING DRILLING
▼	GROUNDWATER LEVEL AFTER DRILLING

NOTE:
1. EXISTING CONDITIONS REFERENCED FROM THE TOPOGRAPHIC SURVEY PREPARED BY SAMPO ENGINEERING, INC. DATED APRIL 13, 2018.

60 30 0 60
SCALE (FEET)

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CHECKED BY SR
DATE: 06/2020
REVISED: -

GEOLOGIC CROSS-SECTION A-A'

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
6

PAGE: 1 of 1

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LAYOUT: Layout1 (2)
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APPENDIX A

BOREHOLE LOGS

APPENDIX A
BOREHOLE LOGS

The geotechnical borehole explorations for the project consisted of drilling and logging two borings, designated as R-20-001 and R-20-002, advanced by Pacific Drilling of San Diego, California. The borings were drilled using a truck-mounted drill rig between February 10th and 21st, 2020. The borings were advanced to depths of approximately 151 and 208 feet below ground surface, respectively, using 8-inch outer-diameter hollow-stem augers and a 4-inch-diameter tri-cone roller bit with the rotary wash method. The approximate locations of the boreholes are presented in Figures 2 and 3.

A Unified Soil Classification System (USCS) chart, graphics key, and borehole log legend are presented in Appendix A in addition to the borehole logs. The borehole logs describe the earth materials encountered, samples obtained, and show results of field and select laboratory tests. The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

The boreholes were logged by our field engineer who collected bulk and intact samples of encountered materials for further evaluation and laboratory testing. In-place soil samples were obtained at the test boring locations using a Standard Penetration Test (SPT) or California-type Samplers driven a total of 18 inches (or until practical refusal) into the undisturbed soil at the bottom of the borehole. The soil sampled by the SPT (2-inch outer diameter) or California-type sampler (3-inch outer diameter) was returned to our laboratory for testing. The samplers and associated rods were driven using a 140-pound automatic hammer falling a distance of 30 inches. The number of hammer blows to drive the samplers every 6 inches is recorded on the boring logs. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count (or N-value). The blow count values are the field values and have not been corrected for effects such as overburden pressure, sampler size, sample depth, hammer efficiency, etc. on the boring logs.

Prior to drilling of the borings, a utility mark-out was performed by Southwest Geophysics using various geophysical survey equipment. Additionally, prior to the start of drilling, the surficial pavement was cored by Cut N Core and the first 5 to 6 feet of each borehole was advanced via a manual hand auger to further clear for utilities. Upon completion, the boreholes were backfilled with bentonite and patched at the surface with asphalt concrete. Soil cuttings were stored in 55-gallon steel drums and were disposed of offsite.

GROUP SYMBOLS AND NAMES

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

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

FIELD AND LABORATORY TESTS

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

SAMPLER GRAPHIC SYMBOLS

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

DRILLING METHOD SYMBOLS

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

WATER LEVEL SYMBOLS

- ▽ First Water Level Reading (during drilling)
- ▼ Static Water Level Reading (short-term)
- ▼ Static Water Level Reading (long-term)

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REPORT TITLE
BORING RECORD LEGEND

DIST.	COUNTY	ROUTE	POSTMILE	EA
11	**San Diego**	**NA**	**NA**	**NA**

PROJECT OR BRIDGE NAME
Camino Del Mar Bridge Replacement

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
NA	**ST**	**2-26-20**	**1 of 3**

PROJECT NUMBER: 20180876.001A
gINT FILE: Klf_gint_master_2018
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2018.GLB [CLIENT_CALTRANS BR KEY P1_SOIL]

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

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

CEMENTATION

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

NOTE: This legend sheet provides descriptors and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), Section 2, for tables of additional soil description components and discussion of soil description and identification.

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REPORT TITLE
BORING RECORD LEGEND

DIST.	COUNTY	ROUTE	POSTMILE	EA
11	**San Diego**	**NA**	**NA**	**NA**

PROJECT OR BRIDGE NAME
Camino Del Mar Bridge Replacement

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
NA	**ST**	**2-26-20**	**2 of 3**

ROCK GRAPHIC SYMBOLS

▩	IGNEOUS ROCK
▤	SEDIMENTARY ROCK
▧	METAMORPHIC ROCK

BEDDING SPACING

Descriptor	Thickness or Spacing
Massive	> 10 ft
Very thickly bedded	3 to 10 ft
Thickly bedded	1 to 3 ft
Moderately bedded	3-5/8 inches to 1 ft
Thinly bedded	1-1/4 to 3-5/8 inches
Very thinly bedded	3/8 inch to 1-1/4 inches
Laminated	< 3/8 inch

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Solutioning		
Descriptor	Body of Rock	Fracture Surfaces		Texture	Solutioning	General Characteristics
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	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 may be noted	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 (refer to grain boundary conditions)	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Altered by chemical disintegration such as via hydration or 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".

Note: Combination descriptors (such as "slightly weathered to fresh") are used where equal distribution of both weathering characteristics is present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, combination descriptors should not be used where significant identifiable zones can be delineated. Only two adjacent descriptors shall be combined. "Very intensely weathered" is the combination descriptor for "decomposed to intensely weathered".

RELATIVE STRENGTH OF INTACT ROCK

Descriptor	Uniaxial Compressive Strength (psi)
Extremely Strong	> 30,000
Very Strong	14,500 - 30,000
Strong	7,000 - 14,500
Medium Strong	3,500 - 7,000
Weak	700 - 3,500
Very Weak	150 - 700
Extremely Weak	< 150

ROCK HARDNESS

Descriptor	Criteria
Extremely Hard	Specimen cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Specimen cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Specimen can be scratched with pocket knife or sharp pick with heavy pressure; heavy hammer blows required to break specimen
Moderately Hard	Specimen can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately Soft	Specimen can be grooved 1/6 in. with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Specimen can be grooved or gouged with pocket knife or sharp pick with light pressure, breaks with light to moderate hand pressure
Very Soft	Specimen can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light hand pressure

CORE RECOVERY CALCULATION (%)

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

RQD CALCULATION (%)

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

FRACTURE DENSITY

Descriptor	Criteria
Unfractured	No fractures
Very Slightly Fractured	Lengths greater 3 ft
Slightly Fractured	Lengths from 1 to 3 ft, few lengths outside that range
Moderately Fractured	Lengths mostly in range of 4 in. to 1 ft, with most lengths about 8 in.
Intensely Fractured	Lengths average from 1 in. to 4 in. with scattered fragmented intervals with lengths less than 4 in.
Very Intensely Fractured	Mostly chips and fragments with few scattered short core lengths

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REPORT TITLE BORING RECORD LEGEND				
DIST. 11	COUNTY San Diego	ROUTE NA	POSTMILE NA	EA NA
PROJECT OR BRIDGE NAME Camino Del Mar Bridge Replacement				
BRIDGE NUMBER NA	PREPARED BY ST	DATE 2-26-20	SHEET 3 of 3	

LOGGED BY **S.Tena**	BEGIN DATE **2-18-20**	COMPLETION DATE **2-21-20**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **32.97607° / -117.26928° WGS84**
DRILLING CONTRACTOR **Pacific Drilling**			BOREHOLE LOCATION (Offset, Station, Line) **Sta N/A**
DRILLING METHOD **Mud Rotary**			DRILL RIG **Marl 10**
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4"), CAL (2.5")**			SPT HAMMER TYPE **Auto; 140 lbs / 30-inch drop**
BOREHOLE BACKFILL AND COMPLETION **Bentonite and grout**			GROUNDWATER READINGS DURING DRILLING **14.0 ft** AFTER DRILLING (DATE) **17.0 ft on 2-21-20**

HOLE ID	**R-20-001**
SURFACE ELEVATION	**~16.00 ft NAVD88**
BOREHOLE DIAMETER	**8 in / 4 in**
HAMMER EFFICIENCY, ERi	**94%**
TOTAL DEPTH OF BORING	**151.0 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	0	ASPHALT CONCRETE; (5").									
15.0		POORLY GRADED SAND with SILT (SP-SM); yellowish red (5YR 5/6); moist; mostly medium to fine SAND; little fines; non-plastic (ARTIFICIAL FILL (af)).	S1					3			M, PA, R, CR
		- yellow (10YR 7/6) and dark yellowish brown (10YR 4/4); coarse to fine SAND.									
	5										
10.0		- medium dense; strong brown (7.5YR 4/6); medium to fine SAND.	S2	12 13 13	26	94					
			S3	5 10 11	21	83		5			M, PA
	10	POORLY GRADED SAND with SILT (SP-SM); medium dense; light brownish gray (2.5Y 6/1); moist; mostly fine SAND; little fines; non-plastic (RECENT ALLUVIAL DEPOSITS (Qa)).	S4	8 8 11	19	77					
5.0		- gray (2.5Y 5/1); medium to fine SAND; increase in moisture content.	S5	6 7 9	16	72		14			M, PA, PI
	15	- wet.	S6	7 11 14	25	89		25			M
0.0		POORLY GRADED SAND (SP); medium dense; gray (2.5Y 5/1); wet; medium to fine SAND; non-plastic; micaceous.	S7	6 8 9	17	89		27			M, PA
		POORLY GRADED SAND with SILT (SP-SM); medium dense; dark gray (2.5Y 4/1); wet; medium to fine SAND; little fines; non-plastic.	S8	3 7 5	12	77		26			Added water at 18 feet. M
	20	- loose; few coarse subrounded GRAVEL, 2 in. max. dia..	S9	1 3 3	6	39					Switch to mud rotary drilling from hollow stem auger at 20 feet. PA
-5.0											
	25										

(continued)

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REPORT TITLE **BORING RECORD**			HOLE ID **R-20-001**	
DIST. **11**	COUNTY **San Diego**	ROUTE **NA**	POSTMILE **NA**	EA **NA**
PROJECT OR BRIDGE NAME **Camino Del Mar Bridge Replacement**				
BRIDGE NUMBER **NA**	PREPARED BY **ST**		DATE **2-26-20**	SHEET **1 of 6**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25		SILTY SAND (SM); medium dense; dark gray (2.5Y 4/1); wet; mostly medium to fine SAND; little fines.	⧖	S10	7 7 6	13	72							
-10.0															
			LEAN CLAY (CL); very soft; dark gray (2.5Y 4/1); wet; few fine SAND; mostly fines; medium plasticity.												
	30			⧗	S11	2 4 3	7	33		55	66	PP=0.0			M, UW, PI
-15.0															
			SILTY SAND (SM); loose; dark gray (2.5Y 4/1); wet; mostly medium to fine SAND; little fines; non-plastic; trace shell fragments.												Rocky from 33 to 34 feet.
			FAT CLAY (CH); very soft; dark gray (2.5Y 4/1); wet; few medium SAND; mostly fines; medium to high plasticity; trace roots and shell fragments.												
	35			⧖	S12	4 4 4	8	55				PP=0.0			PA, PI
-20.0															
			SILTY SAND with GRAVEL (SM); wet; (inferred from drilling action).												Hard drilling due to gravel layers.
	40		- subrounded gravel (3") inside sampler.	⧗		17 18 34	52	NR							Rocky from 40 to 50 feet due to gravel layers. No sample recovery at 40 to 41.5 feet.
-25.0															
	45			⧖		14 15 16	31	NR							No sample recovery at 45 to 46.5 feet.
-30.0															
			SILTY SAND (SM); dense; dark gray (2.5Y 4/1); wet; mostly coarse to fine SAND; little fines; non-plastic; trace shell fragments (YOUNG ALLUVIAL DEPOSITS (Qya)).												
	50			⧗	S13	17 20 20	40	44		21	110				M, UW, PA
-35.0															
	55														

(continued)

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REPORT TITLE **BORING RECORD**				HOLE ID **R-20-001**
DIST. **11**	COUNTY **San Diego**	ROUTE **NA**	POSTMILE **NA**	EA **NA**
PROJECT OR BRIDGE NAME **Camino Del Mar Bridge Replacement**				
BRIDGE NUMBER **NA**	PREPARED BY **ST**		DATE **2-26-20**	SHEET **2 of 6**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
-40.0	55		SILTY SAND (SM); dense; dark gray (2.5Y 4/1); wet; mostly medium to fine SAND; little fines; non-plastic.		S14	15 20 21	41	77							
-45.0	60		- very dense; micaceous.		S15	17 26 40	66	55		25	102				M, UW, PA, PI
-50.0	65		- loose.		S16	11 5 3	8	55				PP=0.0			
			SANDY LEAN CLAY (CL); very soft; dark gray (2.5Y 4/1); wet; some fine SAND; mostly fines; low to medium plasticity.												
			POORLY GRADED SAND with SILT (SP-SM); very dense; dark gray (2.5Y 4/1); wet; mostly medium to fine SAND; little fines; non-plastic; trace shell fragments.												
-55.0	70				S17	11 20 31	51	66		21	108				M, UW, PA
-60.0	75		- dense.		S18	21 23 20	43	77							PI
-65.0	80		- very dense.		S19	29 40 31	71	66		21	107				M, UW, PA
	85														

(continued)

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REPORT TITLE **BORING RECORD**				HOLE ID **R-20-001**
DIST. **11**	COUNTY **San Diego**	ROUTE **NA**	POSTMILE **NA**	EA **NA**
PROJECT OR BRIDGE NAME **Camino Del Mar Bridge Replacement**				
BRIDGE NUMBER **NA**	PREPARED BY **ST**		DATE **2-26-20**	SHEET **3 of 6**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85		SILTY SAND (SM); medium dense; dark gray (2.5Y 4/1); wet; mostly medium to fine SAND; little fines; non-plastic (YOUNG ESTUARINE DEPOSITS (Qyes)).		S20	13 11 10	21	72							
-70.0															
			LEAN CLAY (CL); medium stiff; black (10YR 2/1); wet; few fine SAND; mostly fines; medium plasticity; micaceous, trace shell fragments.												
	90				S21	4 5 7	12	66		47	77	PP=0.5			
-75.0															M, UW, PI, WA
	95		SILTY SAND (SM); medium dense; very dark gray (10YR 3/1); wet; mostly medium to fine SAND; some fines; non-plastic (OLD ALLUVIAL DEPOSITS (Qoa)).		S22	4 6 14	20	100							
-80.0															
	100		- dense.		S23	7 13 22	35	66							
-85.0															DS
	105		POORLY GRADED SAND with SILT (SP-SM); medium dense; dark gray (10YR 4/1); wet; mostly medium to fine SAND; little fines; non-plastic; micaceous.		S24	5 10 16	26	89							
-90.0															
	110		- SAA.		S25	12 11 13	24	66		32	92				M, UW, PA
-95.0															
	115														

(continued)

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REPORT TITLE **BORING RECORD**				HOLE ID **R-20-001**	
DIST. **11**	COUNTY **San Diego**	ROUTE **NA**	POSTMILE **NA**	EA **NA**	
PROJECT OR BRIDGE NAME **Camino Del Mar Bridge Replacement**					
BRIDGE NUMBER **NA**	PREPARED BY **ST**			DATE **2-26-20**	SHEET **4 of 6**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	115		SILTY SAND (SM); dense; dark gray (10YR 4/1); mostly medium to fine SAND; little fines; non-plastic.	☒	S26	10 17 16	33	100							
-100.0															
	120		- very dense.	⧓	S27	14 24 33	57	33		25	103				M, UW, PA
-105.0															
	125		- SAA.	☒	S28	12 24 31	55	89							
-110.0															
	130		- SAA.	⧓	S29	14 24 29	53	66							
-115.0															DS
															Increase in drilling effort at 132 feet.
	135														
-120.0															
	140		- dense.	☒	S30	21 19 17	36	100							
-125.0															
	145														

(continued)

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REPORT TITLE				HOLE ID
BORING RECORD				**R-20-001**

DIST.	COUNTY	ROUTE	POSTMILE	EA
11	**San Diego**	**NA**	**NA**	**NA**

PROJECT OR BRIDGE NAME
Camino Del Mar Bridge Replacement

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
NA	**ST**	**2-26-20**	**5 of 6**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	145		SILTY SAND (SM); dense; dark gray (10YR 4/1); mostly medium to fine SAND; little fines; non-plastic.												
-130.0			CLAYSTONE; dark reddish brown (2.5YR 3/4) with grayish green (GLEY 1-5/5GY); medium plasticity (DEL MAR FORMATION (Td)).												Hard drilling at 146 feet.
	150				S31	30 50/5"	50/5	100				PP=4.5			PA, PI
-135.0			Bottom of borehole at 151.0 ft bgs												
	155														
-140.0															
	160														
-145.0															
	165														
-150.0															
	170														
-155.0															
	175														

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REPORT TITLE
BORING RECORD

HOLE ID
R-20-001

DIST.
11

COUNTY
San Diego

ROUTE
NA

POSTMILE
NA

EA
NA

PROJECT OR BRIDGE NAME
Camino Del Mar Bridge Replacement

BRIDGE NUMBER
NA

PREPARED BY
ST

DATE
2-26-20

SHEET
6 of 6

LOGGED BY **S.Tena**	BEGIN DATE **2-10-20**	COMPLETION DATE **2-13-20**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **32.97396° / -117.26878° WGS84**	HOLE ID **R-20-002**
DRILLING CONTRACTOR **Pacific Drilling**			BOREHOLE LOCATION (Offset, Station, Line) **Sta N/A**	SURFACE ELEVATION **~16.00 ft NAVD88**
DRILLING METHOD **Mud Rotary**			DRILL RIG **Marl 10**	BOREHOLE DIAMETER **8 in / 4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4"), CAL (2.5")**			SPT HAMMER TYPE **Auto; 140 lbs / 30-inch drop**	HAMMER EFFICIENCY, ERi **94%**
BOREHOLE BACKFILL AND COMPLETION **Bentonite and grout**			GROUNDWATER READINGS DURING DRILLING **11.0 ft** AFTER DRILLING (DATE) **Not Applicable**	TOTAL DEPTH OF BORING **208.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0		ASPHALT CONCRETE; (6").												
15.0			POORLY GRADED SAND (SP); light brownish gray (10YR 6/2); moist; trace subrounded GRAVEL, 2 in. max. dia.; mostly medium to fine SAND; non-plastic; micaceous (ARTIFICIAL FILL (af)).	▨	S1					4					M, PA, R, CR
10.0	5		POORLY GRADED SAND with SILT (SP-SM); medium dense; light brownish gray (10YR 6/2); moist; mostly medium to fine SAND; little fines; non-plastic.		S2	5 4 7	11	100							
			- loose; trace shell fragments.		S3	5 5 5	10	66		4					M, PA, PI
	10		POORLY GRADED SAND with SILT (SP-SM); very loose; brown (10YR 5/3); moist; mostly medium to fine SAND; little fines; non-plastic; micaceous (RECENT ALLUVIAL DEPOSITS (Qa)).		S4	3 3 1	4	55							
5.0			- dark grayish brown (10YR 4/2); moist to wet.		S5	2 1 1	2	55		21					M, PA
			- very dark gray (10YR 3/1); wet; some fines; non-plastic to low plasticity; trace of odor.		S6	1 2 2	4	44							
0.0	15		POORLY GRADED SAND (SP); loose; dark gray (10YR 4/1); wet; mostly medium to fine SAND; non-plastic; trace shell fragments, no odor, micaceous. -gravelly layers from 16 to 18 feet.		S7	2 1 4	5	77		27					Added water at 15 feet. M, PA
			- medium dense.		S8	8 9 9	18	77		27					Switch to mud rotary drilling from hollow stem auger at 16.5 feet. M
	20					1 4 8	12	NR							No sample recovery at 19 to 20.5 feet.
-5.0			SILTY SAND (SM); loose; dark gray (10YR 4/1); wet; mostly medium to fine SAND; some fines; non-plastic; trace shell fragments.		S9	6 3 2	5	55							PA, PI
	25														

(continued)

REPORT TITLE **BORING RECORD**				HOLE ID **R-20-002**	
DIST. **11**	COUNTY **San Diego**	ROUTE **NA**	POSTMILE **NA**	EA **NA**	
PROJECT OR BRIDGE NAME **Camino Del Mar Bridge Replacement**					
BRIDGE NUMBER **NA**	PREPARED BY **ST**			DATE **2-14-20**	SHEET **1 of 8**

PLOTTED: 04/21/2020 02:05 PM BY: JBonfiglio

PROJECT NUMBER: 20180876.001A OFFICE FILTER: SAN DIEGO

gINT FILE: Klf_gint_master_2018

gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2018.GLB [CLIENT_CALTRANS BORING RECORD MET/ENG]

PLOTTED: 04/21/2020 02:05 PM BY: JBonfiglio

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
-10.0	25		SILTY SAND (SM); dense; dark gray (10YR 4/1); trace subrounded GRAVEL, 3 in. max. dia.; mostly medium to fine SAND; some fines; non-plastic.		S10	12 17 17	34	33		26	100				M, UW
-15.0	30		POORLY GRADED SAND with SILT (SP-SM); dense; dark gray (10YR 4/1); wet; mostly medium to fine SAND; little fines; non-plastic (YOUNG ALLUVIAL DEPOSITS (Qya)).		S11	14 18 16	34	77							PA
-20.0	35		- SAA.		S12	17 21 20	41	55		24	101				M, UW
-25.0	40		SILTY SAND (SM); medium dense; dark gray (10YR 4/1); wet; mostly medium to fine SAND; some fines; non-plastic; increase in SILT content.		S13	6 9 5	14	77							PA
-30.0	45		- very dense; trace GRAVEL, 3 in. max. dia.; medium SAND; little fines.		S14	22 37 44	81	66		24	105				M, UW
-35.0	50		POORLY GRADED SAND (SP); medium dense; dark gray; wet; mostly medium to fine SAND; non-plastic; trace shell fragments.		S15	11 14 15	29	100							PA, PI
	55														

(continued)

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REPORT TITLE **BORING RECORD**				HOLE ID **R-20-002**	
DIST. **11**	COUNTY **San Diego**	ROUTE **NA**	POSTMILE **NA**	EA **NA**	
PROJECT OR BRIDGE NAME **Camino Del Mar Bridge Replacement**					
BRIDGE NUMBER **NA**	PREPARED BY **ST**			DATE **2-14-20**	SHEET **2 of 8**

gINT FILE: Klf_gint_master_2018 PROJECT NUMBER: 20180876.001A OFFICE FILTER: SAN DIEGO
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2018.GLB [CLIENT_CALTRANS BORING RECORD MET/ENG]

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55		POORLY GRADED SAND with SILT (SP-SM); very dense; dark gray (10YR 4/1); mostly medium SAND; little fines; non-plastic.	⧗	S16	16 34 39	73	66		37	92				M, UW
-40.0															
	60		- medium dense.	☒	S17	10 9 12	21	77							PA
-45.0															
	65		- very dense.	⧗	S18	24 30 26	56	66		29	96				M, UW
-50.0															
	70		- medium dense.	☒	S19	19 15 14	29	77							PA
-55.0															
	75		- very dense.	⧗	S20	14 24 31	55	66		30	96				M, UW
-60.0															
			SILTY SAND (SM); loose; dark gray (10YR 4/1); wet; mostly fine SAND; some fines; non-plastic to low plasticity; micaceous, trace shell fragments (YOUNG ESTUARINE DEPOSITS (Qyes)).												
	80			☒	S21	5 3 5	8	100				PP=0.5			PA, PI
-65.0															
	85														

(continued)

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REPORT TITLE
BORING RECORD

HOLE ID
R-20-002

DIST. **11**
COUNTY **San Diego**
ROUTE **NA**
POSTMILE **NA**
EA **NA**

PROJECT OR BRIDGE NAME
Camino Del Mar Bridge Replacement

BRIDGE NUMBER **NA**
PREPARED BY **ST**
DATE **2-14-20**
SHEET **3 of 8**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
-70.0	85		SANDY SILT (ML); medium stiff; dark gray (10YR 4/1); wet; some fine SAND; mostly fines; non-plastic to low plasticity.		S22	8 11 16	27	100		48	74	PP=0.5			Hole caved to 20 feet bgs on 2/11/2020 prior to start of drilling activities. M, UW
			LEAN CLAY (CL); medium stiff; dark gray (10YR 4/1); wet; few fine SAND; mostly fines; low plasticity; micaceous, trace shell fragments.												
-75.0	90				S23	3 4 5	9	100				PP=0.5			PA
			SILTY SAND (SM); dense; very dark gray (10YR 3/1); wet; mostly medium to fine SAND; little fines; non-plastic; micaceous, trace shell fragments (OLD ALLUVIAL DEPOSITS (Qoa)).												
-80.0	95				S24	10 18 30	48	66		28	95				M, UW, UC
-85.0	100		- medium dense; some fines; non-plastic to low plasticity; interbeded layer (1") of Silty Clay material.		S25	7 10 9	19	100							PA
-90.0	105		- dense; little fines; non-plastic.		S26	20 25 17	42	33							DS
-95.0	110		- medium dense.		S27	12 13 11	24	83							PA, PI
	115														

(continued)

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REPORT TITLE **BORING RECORD**				HOLE ID **R-20-002**	
DIST. **11**	COUNTY **San Diego**	ROUTE **NA**	POSTMILE **NA**		EA **NA**
PROJECT OR BRIDGE NAME **Camino Del Mar Bridge Replacement**					
BRIDGE NUMBER **NA**	PREPARED BY **ST**			DATE **2-14-20**	SHEET **4 of 8**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
-100.0	115		SILTY SAND (SM); dense; very dark gray (10YR 3/1); wet; mostly medium to fine SAND; little fines; non-plastic.		S28	12 24 26	50	66		29	104				M, UW
-105.0	120		- medium dense; coarse to medium SAND.		S29	13 14 13	27	77							
-110.0	125		- very dense.		S30	15 23 31	54	44		31	94				Hole caved to 115 feet bgs on 2/12/2020 prior to start of drilling activities. M, UW, PA
-115.0	130		- medium dense.		S31	8 9 12	21	89							
-120.0	135		- very dense.		S32	32 37 27	64	44							DS
-125.0	140		- loose; some medium to fine SAND; some fines; non-plastic to low plasticity; micaceous, increase in SILT content.		S33	5 2 8	10	94				PP=0.5			PA, PI
	145														

(continued)

REPORT TITLE
BORING RECORD

HOLE ID
R-20-002

DIST.	COUNTY	ROUTE	POSTMILE	EA
11	**San Diego**	**NA**	**NA**	**NA**

PROJECT OR BRIDGE NAME
Camino Del Mar Bridge Replacement

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
NA	**ST**	**2-14-20**	**5 of 8**

KLEINFELDER
Bright People. Right Solutions.

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	145		SILTY SAND (SM); very dense; very dark gray (10YR 3/1); wet; mostly medium to fine SAND; little fines; non-plastic.		S34	23 29 30	59	44		29	92				M, UW
-130.0															
	150		POORLY GRADED SAND with SILT (SP-SM); dense; very dark gray; wet; mostly medium to fine SAND; little fines; non-plastic.		S35	15 16 17	33	77							PA
-135.0															
	155														
-140.0															
	160		- very dense; coarse to medium SAND.		S36	25 37 38	75	44							
-145.0															DS
	165														
-150.0															
	170		SILTY SAND (SM); medium dense; very dark gray; wet; some fines; non-plastic to low plasticity.		S37	4 7 9	16	100							PA
-155.0															
	175														

(continued)

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REPORT TITLE
BORING RECORD

HOLE ID
R-20-002

DIST.	COUNTY	ROUTE	POSTMILE	EA
11	**San Diego**	**NA**	**NA**	**NA**

PROJECT OR BRIDGE NAME
Camino Del Mar Bridge Replacement

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
NA	**ST**	**2-14-20**	**6 of 8**

PLOTTED: 04/21/2020 02:05 PM BY: JBonfiglio

PROJECT NUMBER: 20180876.001A OFFICE FILTER: SAN DIEGO

gINT FILE: Klf_gint_master_2018

gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2018.GLB [CLIENT_CALTRANS BORING RECORD MET/ENG]

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	175		SILTY SAND (SM); medium dense; very dark gray (10YR 3/1); wet; mostly coarse to medium SAND; some fines; non-plastic to low plasticity.												
-160.0															
			SILTY CLAY with SAND (CL-ML); stiff; very dark gray (10YR 3/1); wet; little SAND; mostly fines; medium plasticity.												
	180				S38 S39	19 28 40	68	61		24 18	105 107	PP=1.0			M, UW, PI M, UW
-165.0															
			SILTY SAND (SM); very dense; very dark gray (10YR 3/1); wet; mostly medium to fine SAND; little fines; non-plastic.												
	185														
-170.0															Hole caved to 145 feet bgs on 2/13/2020 prior to start of drilling activities.
	190		- dense.		S40	19 17 19	36	94							
-175.0															
	195														
-180.0															
	200		- very dense; olive gray (5Y 5/2); trace subrounded GRAVEL, 1 in. max. dia.; coarse to medium SAND; little fines; iron oxide staining.		S41	34 50/5"	50/5	45							
-185.0															
	205														

(continued)

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REPORT TITLE
BORING RECORD

HOLE ID
R-20-002

DIST.	COUNTY	ROUTE	POSTMILE	EA
11	**San Diego**	**NA**	**NA**	**NA**

PROJECT OR BRIDGE NAME
Camino Del Mar Bridge Replacement

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
NA	**ST**	**2-14-20**	**7 of 8**

PLOTTED: 04/21/2020 02:05 PM BY: JBonfiglio
OFFICE FILTER: SAN DIEGO
PROJECT NUMBER: 20180876.001A
gINT FILE: Klf_gint_master_2018
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2018.GLB [CLIENT_CALTRANS BORING RECORD MET/ENG]

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	205		SILTY SAND (SM); very dense; olive gray (5Y 5/2); trace subrounded GRAVEL, 1 in. max. dia.; mostly coarse to medium SAND; little fines.												
-190.0															Practical refusal at 208 feet.
			Bottom of borehole at 208.0 ft bgs												
	210														
-195.0															
	215														
-200.0															
	220														
-205.0															
	225														
-210.0															
	230														
-215.0															
	235														

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REPORT TITLE **BORING RECORD**			HOLE ID **R-20-002**	
DIST. **11**	COUNTY **San Diego**	ROUTE **NA**	POSTMILE **NA**	EA **NA**
PROJECT OR BRIDGE NAME **Camino Del Mar Bridge Replacement**				
BRIDGE NUMBER **NA**	PREPARED BY **ST**		DATE **2-14-20**	SHEET **8 of 8**

APPENDIX B

CONE PENETROMETER TEST (CPT) LOGS

FUGRO
Fugro USA Land, Inc.
6100 Hillcroft Ave.
Houston, Texas 77081
USA

March 3, 2020
Report Number 04.09200002

KLEINFELDER
550 West C Street
Suite 1200
San Diego, California 92101
USA

Attn.: Janna Bonfiglio

REPORT FOR
PIEZOCONE PENETRATION TESTING, SHEAR-WAVE VELOCITY MEASUREMENTS AND RELATED SERVICES
DEL MAR, CALIFORNIA

Dear. Ms. Bonfiglio,

Introduction

Fugro is pleased to present data report for Piezocone Penetration Testing, Seismic Shear-Wave Velocity Measurements and Related Services performed at the above-referenced site. This report contains the scope of services performed and the test results.

Scope of Services

We performed four (4) Piezocone Penetration Tests (PCPT) to depths ranging from 16 ft to 200 ft below ground surface and one (1) Seismic PCPT (SCPT) to a depth of 200 ft penetration. All PCPT sounding locations were grouted after the completion of the tests.

PCPT Testing

The PCPT soundings were conducted in general accordance with ASTM D5778-12, *Electronic Friction Cone and Piezocone Penetration Testing of Soils* using a 30-ton truck mounted CPT unit. The in-situ soil data was obtained by hydraulically advancing a cylindrical steel rod, with an instrumented probe at the base, vertically into the subsurface materials at a constant rate of 2 centimeters per second. The instrumented probe consists of a cone-shaped tip element, with an apex angle of 60 degrees with a base area of 15 square centimeters (cm^2) and a cylindrical-shaped side friction sleeve with a surface area of 200 cm^2. A pore transducer is mounted between the tip and friction sleeve. Measurements of penetration resistance at the cone tip (q_c), frictional resistance along the friction sleeve (f_s), and pore water pressure (u_2), were recorded with depth during penetration. PCPT sounding measurements collected for this project are presented on the logs attached at the end of this report.

PCPT methods test the soil *in situ* and soil samples are not obtained. There are several methods to identify the soil type using the PCPT data collected. For your reference, we have presented soil stratigraphy using the attached *Campanella and Robertson's Simplified Soil Behavior Chart (12-zone, 1986)*.

Shear Wave Velocity Measurements

The shear wave velocity measurements were conducted in general accordance with ASTM D7400-08, *Standard Test Methods for Downhole Seismic Testing* during the PCPT sounding. A PCPT tip with x, y, and z geophones located behind the friction sleeve was used. Seismic readings were taken at 5 foot depth intervals during the sounding. The energy source for the seismic readings was a metal shear beam struck horizontally. Multiple readings were stacked at each interval. The interval velocities were determined from arrival times and relative arrival times of horizontally polarized shear (SH) seismic waves.

Please note that because of the empirical nature of the soil behavior chart, the soil identification should be verified locally from soil borings and laboratory testing. Some soils, such as cemented or calcareous soils, or glacial tills are outside the limits of the soil behavior chart.

Closing

Fugro appreciates the opportunity to be of service to you. If you have any questions, please feel free to contact me at 713.346.4004.

Best Regards,

Sheldon Coll

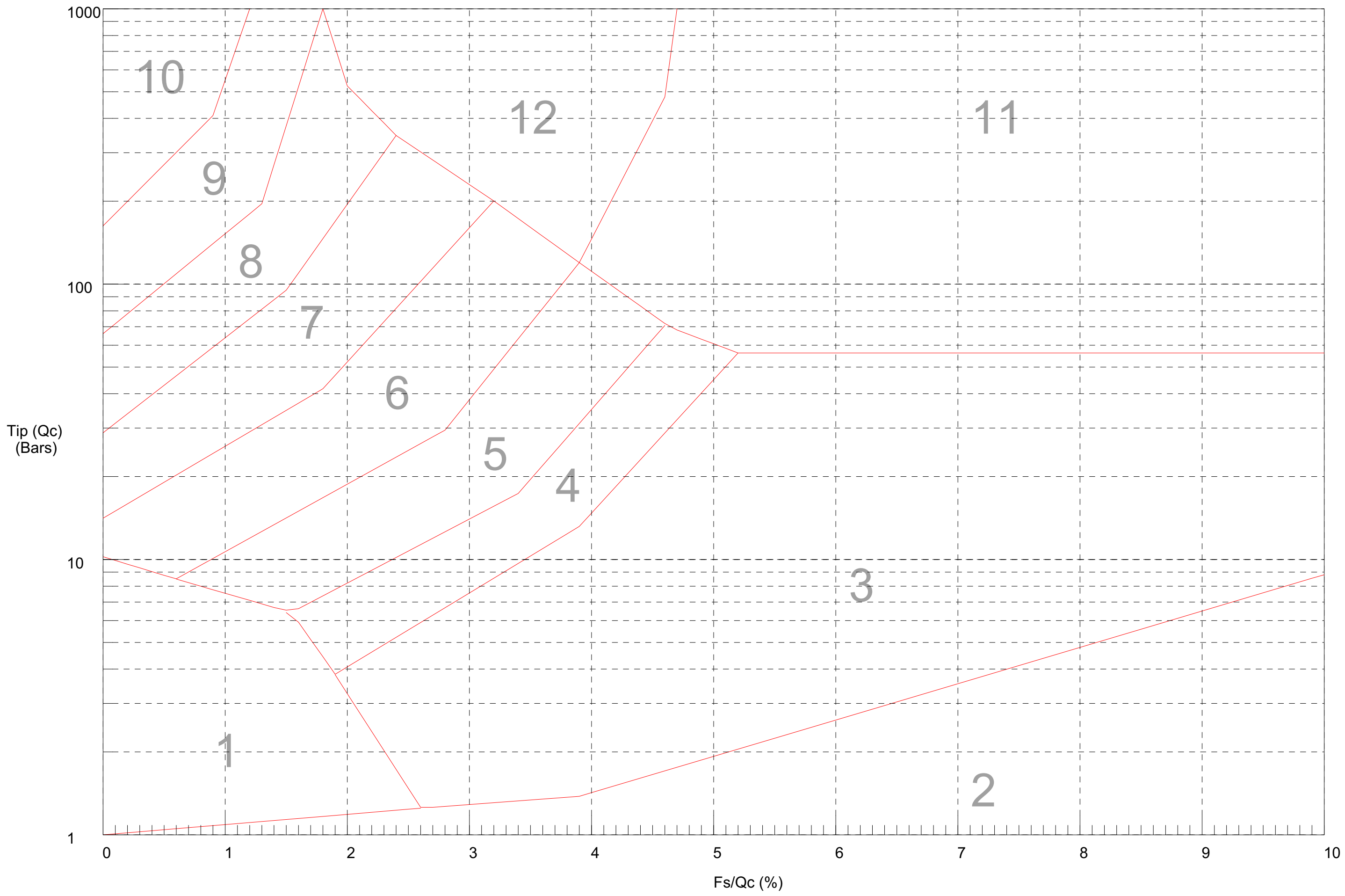
Sheldon Collins
Service Line Manager – CPT
North America

SC/am

Attachments: *Campanella and Robertson's Simplified Soil Behavior Chart* (1 page)
PCPT Sounding Logs (9 pages)
Four (4) Electronic Data Files
Plots of Shear Waves and Shear Waves Velocity (2 pages)
One (1) Shear Wave Velocity Spreadsheets

12 Zone Soil Behavior Chart

Classification Data:
Robertson and Campanella UBC-1986



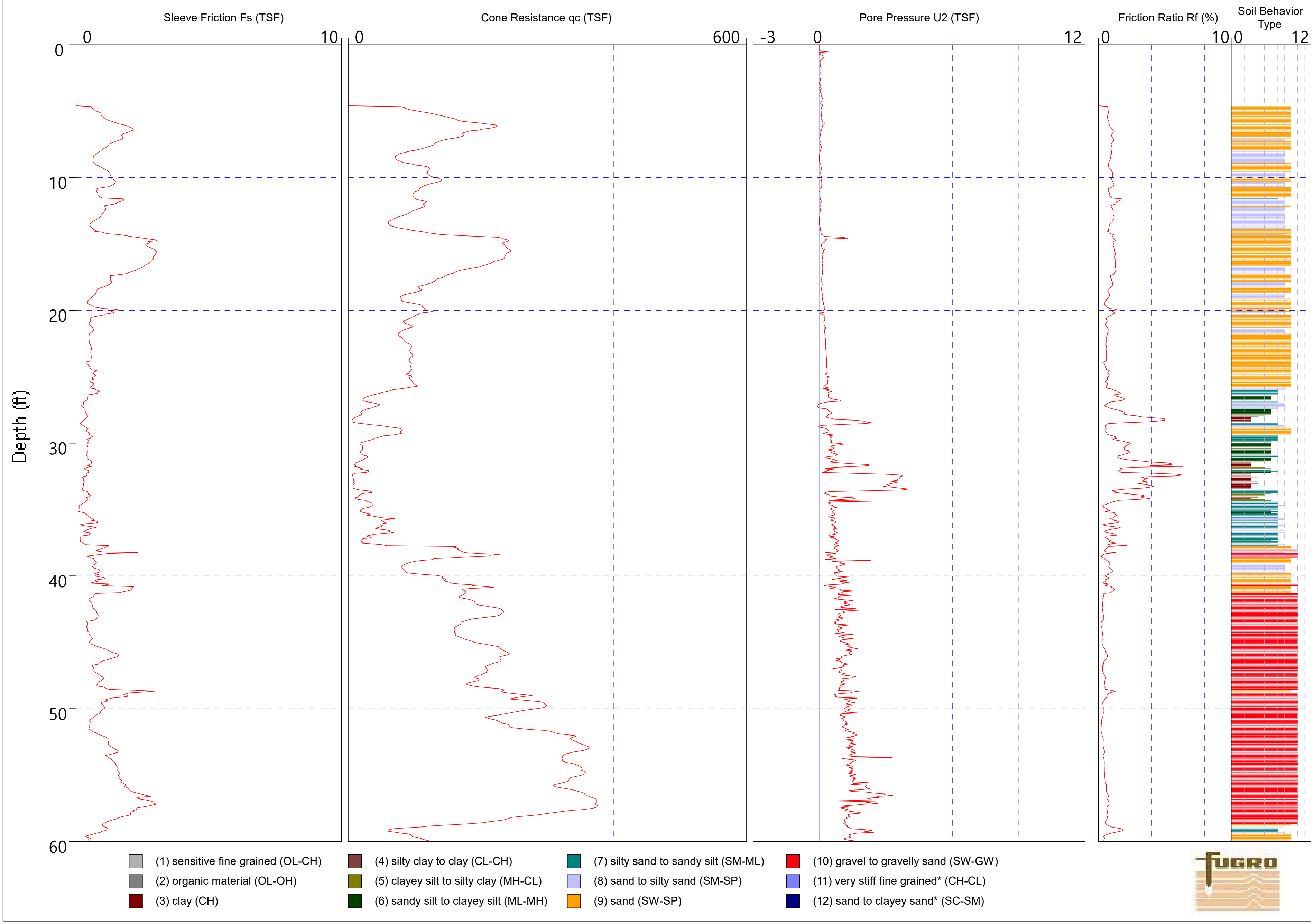
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

* Overconsolidated or cemented

Job Number: 04.09200002
Operator: D. Garza
Location: Del Mar, CA

CPT Number: CPT-20-001
Date: 18-Feb-2020
Elevation: 0.00

Coordinates: 32.976127, -117.269275
Cone Number: CP15-CF25PB7SN2-P1E1 2111

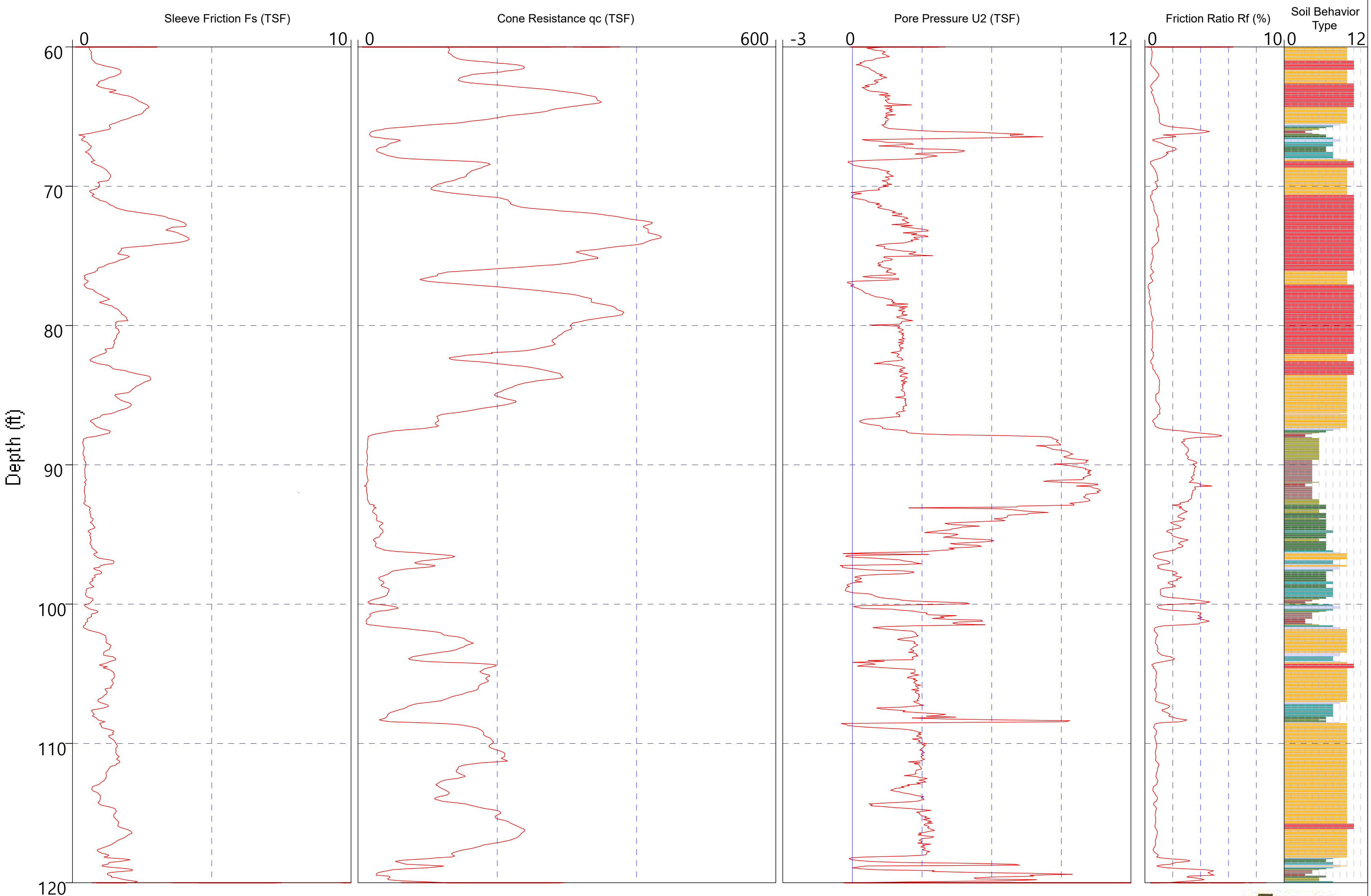


Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.09200002
Operator: D. Garza
Location: Del Mar, CA

CPT Number: CPT-20-001
Date: 18-Feb-2020
Elevation: 0.00

Coordinates: 32.976127, -117.269275
Cone Number: CP15-CF25PB7SN2-P1E1 2111



⬜ (1) sensitive fine grained (OL-CH)
⬛ (2) organic material (OL-OH)
🟥 (3) clay (CH)

🟫 (4) silty clay to clay (CL-CH)
🟩 (5) clayey silt to silty clay (MH-CL)
🟩 (6) sandy silt to clayey silt (ML-MH)

🟦 (7) silty sand to sandy silt (SM-ML)
🟪 (8) sand to silty sand (SM-SP)
🟧 (9) sand (SW-SP)

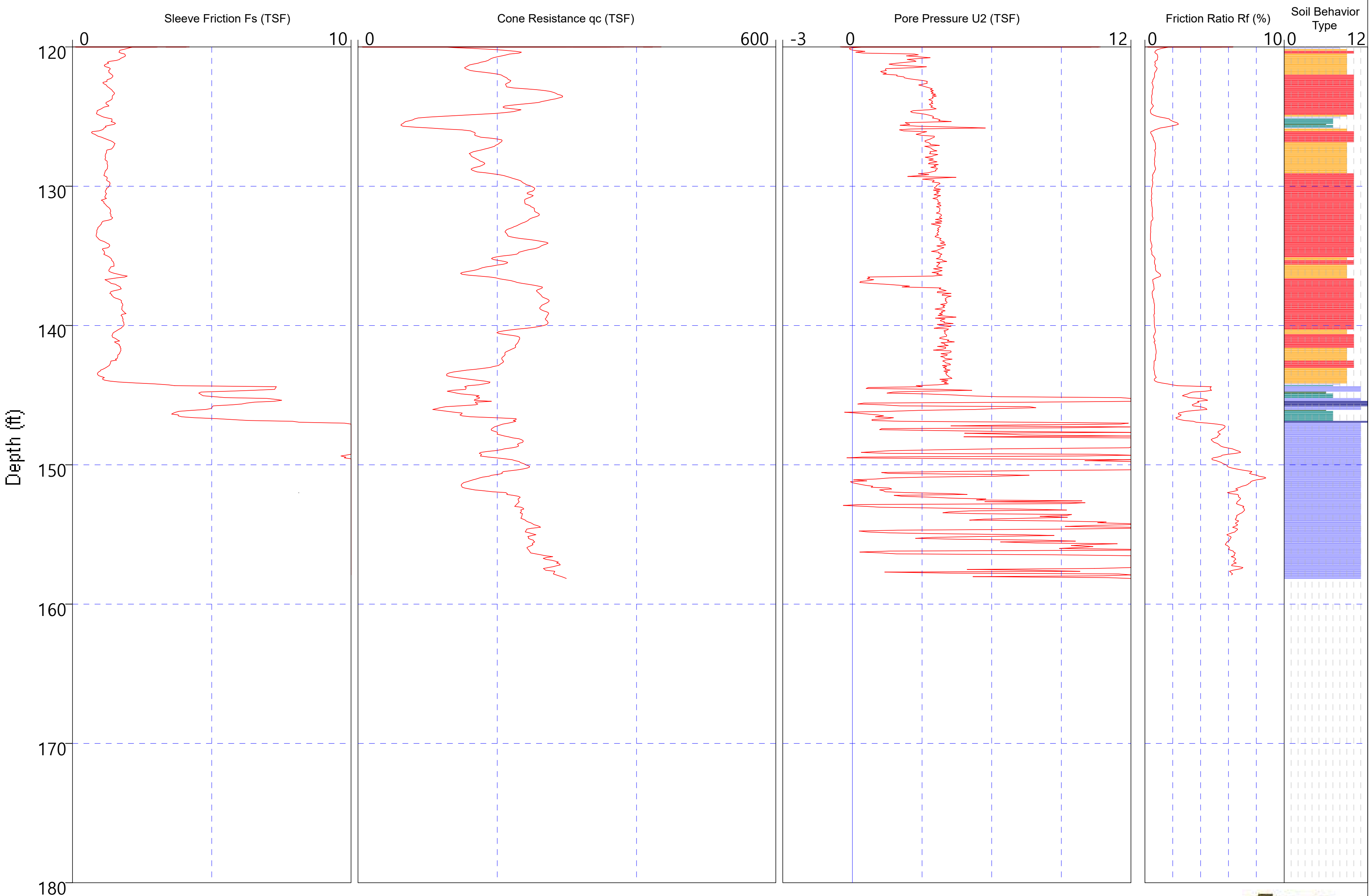
🟥 (10) gravel to gravelly sand (SW-GW)
🟦 (11) very stiff fine grained* (CH-CL)
🟦 (12) sand to clayey sand* (SC-SM)

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.09200002
Operator: D. Garza
Location: Del Mar, CA

CPT Number: CPT-20-001
Date: 18-Feb-2020
Elevation: 0.00

Coordinates: 32.976127, -117.269275
Cone Number: CP15-CF25PB7SN2-P1E1 2111



■ (1) sensitive fine grained (OL-CH)
■ (2) organic material (OL-OH)
■ (3) clay (CH)

■ (4) silty clay to clay (CL-CH)
■ (5) clayey silt to silty clay (MH-CL)
■ (6) sandy silt to clayey silt (ML-MH)

■ (7) silty sand to sandy silt (SM-ML)
■ (8) sand to silty sand (SM-SP)
■ (9) sand (SW-SP)

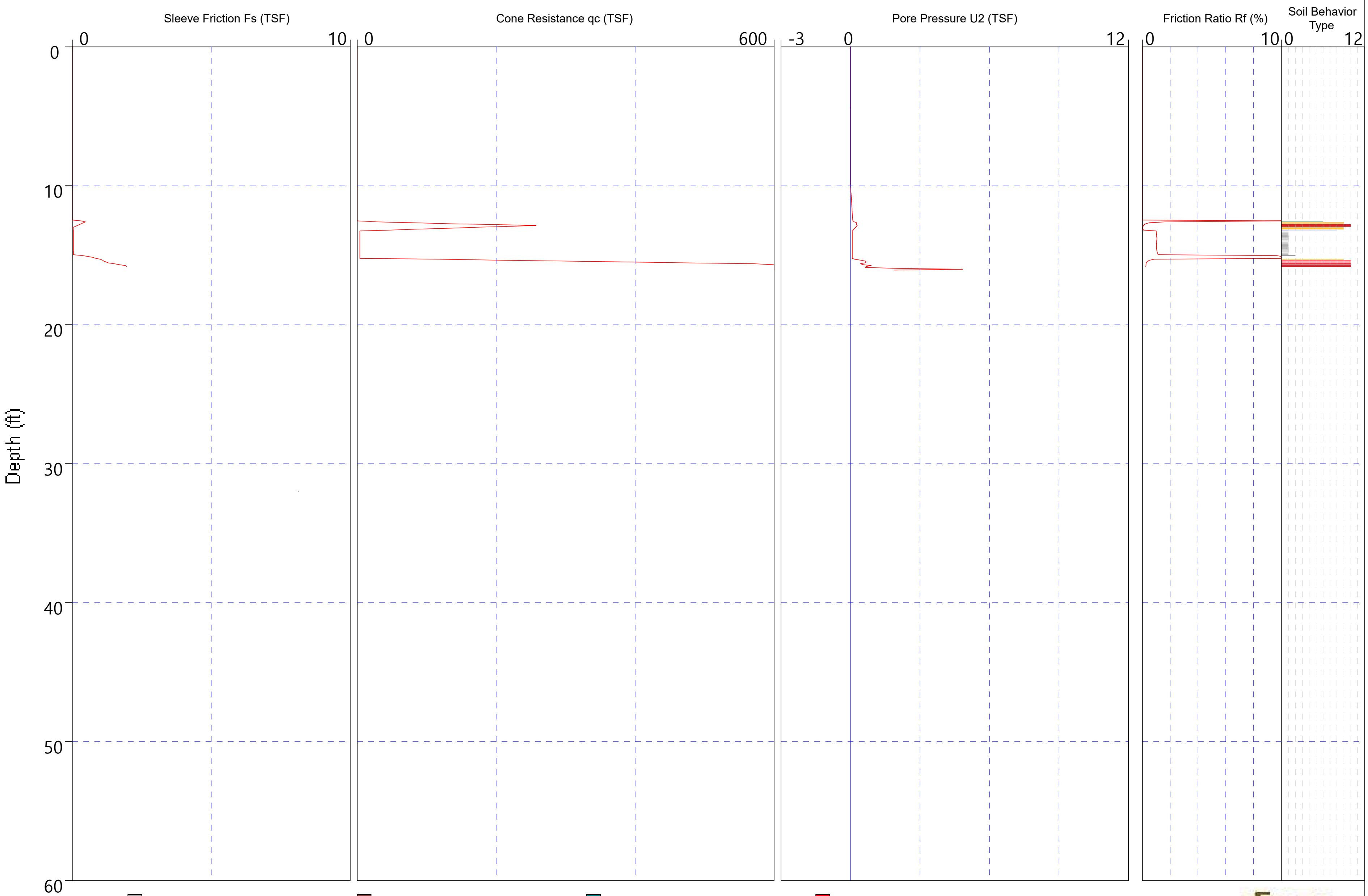
■ (10) gravel to gravelly sand (SW-GW)
■ (11) very stiff fine grained* (CH-CL)
■ (12) sand to clayey sand* (SC-SM)

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.09200002
Operator: D. Garza
Location: Del Mar, CA

CPT Number: CPT-20-002
Date: 21-Feb-2020
Elevation: 0.00

Coordinates: 32.975012, -117.268971
Cone Number: CP15-CF75PB7SN2-P1E1 2999



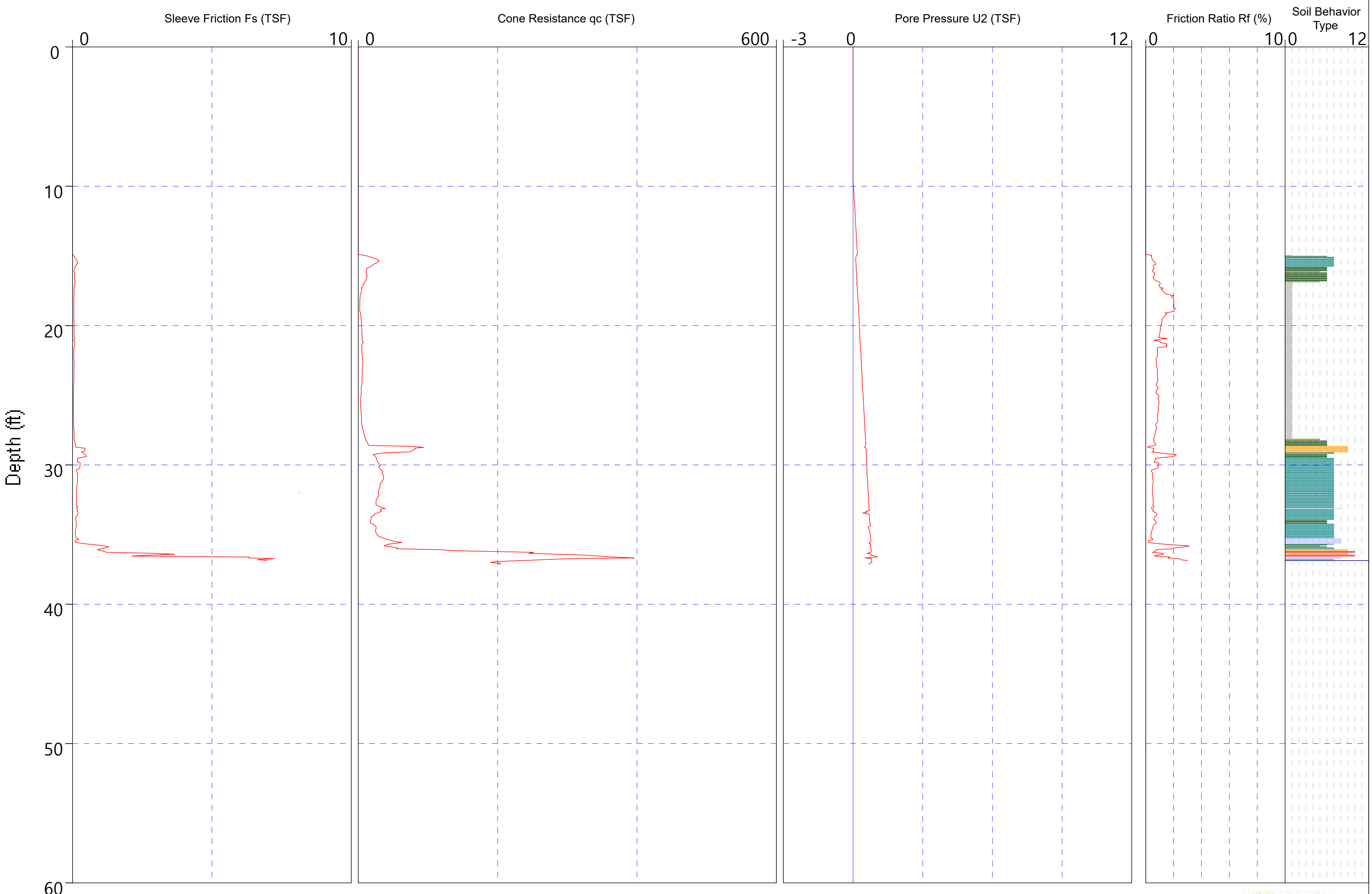
■ (1) sensitive fine grained (OL-CH)
■ (2) organic material (OL-OH)
■ (3) clay (CH)
■ (4) silty clay to clay (CL-CH)
■ (5) clayey silt to silty clay (MH-CL)
■ (6) sandy silt to clayey silt (ML-MH)
■ (7) silty sand to sandy silt (SM-ML)
■ (8) sand to silty sand (SM-SP)
■ (9) sand (SW-SP)
■ (10) gravel to gravelly sand (SW-GW)
■ (11) very stiff fine grained* (CH-CL)
■ (12) sand to clayey sand* (SC-SM)

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.09200002
Operator: D. Garza
Location: Del Mar, CA

CPT Number: CPT-20-002A
Date: 21-Feb-2020
Elevation: 0.00

Coordinates: 32.975012, -117.268971
Cone Number: CP15-CF75PB7SN2-P1E1 2999



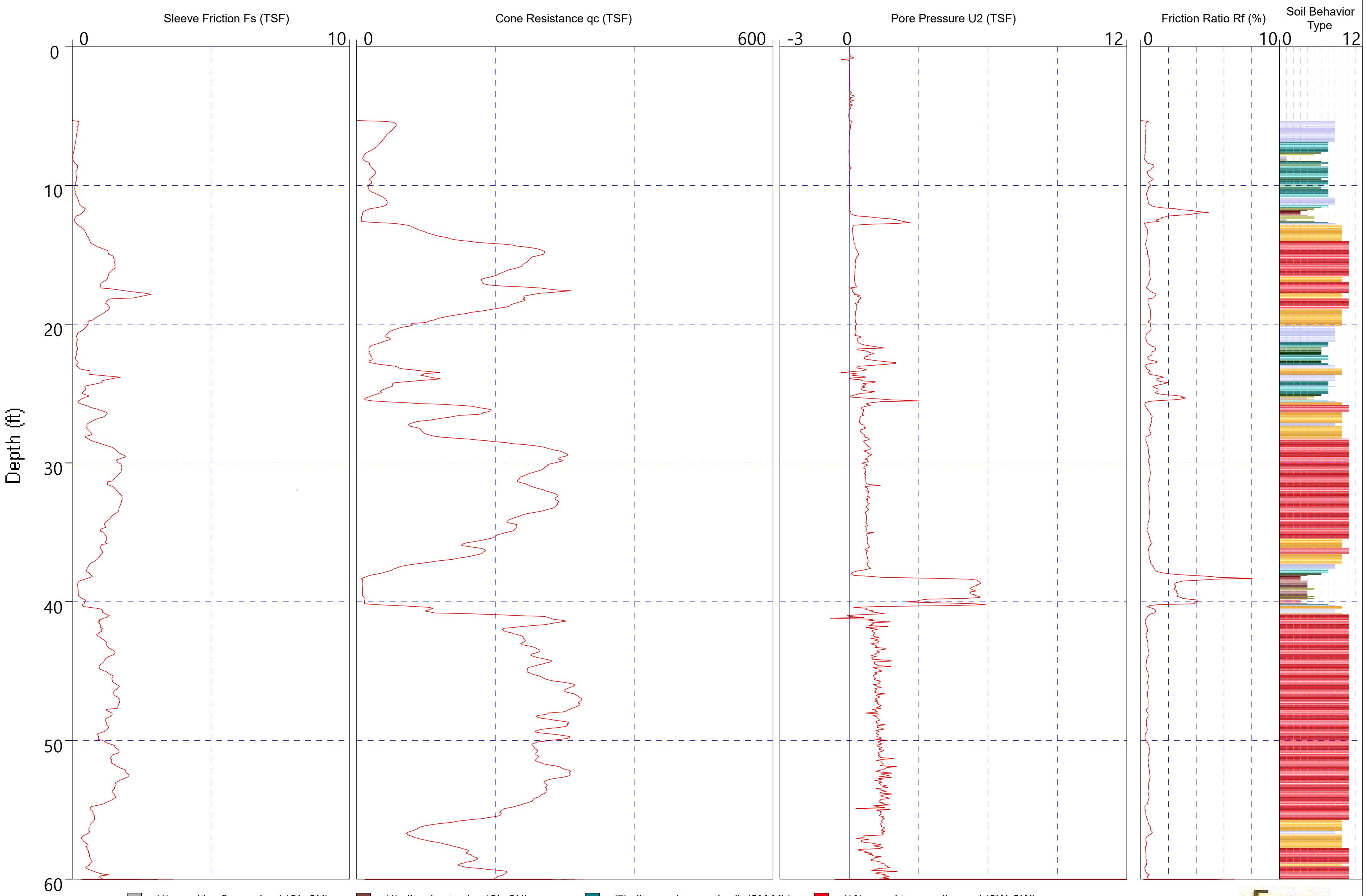
- 🟩 (1) sensitive fine grained (OL-CH)
- 🟩 (2) organic material (OL-OH)
- 🟥 (3) clay (CH)
- 🟥 (4) silty clay to clay (CL-CH)
- 🟩 (5) clayey silt to silty clay (MH-CL)
- 🟩 (6) sandy silt to clayey silt (ML-MH)
- 🟩 (7) silty sand to sandy silt (SM-ML)
- 🟦 (8) sand to silty sand (SM-SP)
- 🟧 (9) sand (SW-SP)
- 🟥 (10) gravel to gravelly sand (SW-GW)
- 🟦 (11) very stiff fine grained* (CH-CL)
- 🟦 (12) sand to clayey sand* (SC-SM)

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.09200002
Operator: D. Garza
Location: Del Mar, CA

CPT Number: CPT-20-003
Date: 19-Feb-2020
Elevation: 0.00

Coordinates: 32.973928, -117.268763
Cone Number: CP15-CF75PB7SN2-P1E1 2999



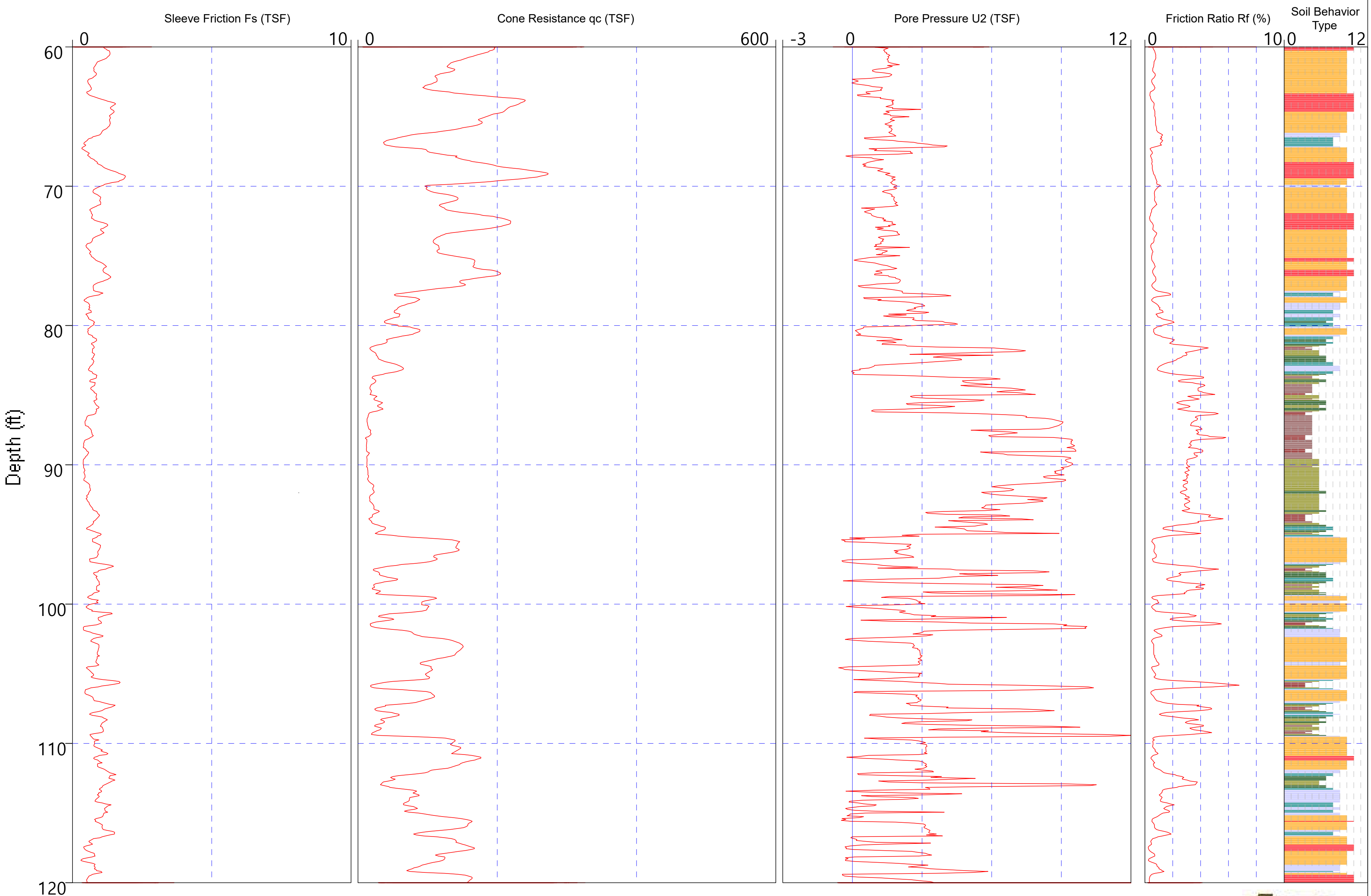
- ⬜ (1) sensitive fine grained (OL-CH)
- ⬛ (2) organic material (OL-OH)
- 🟥 (3) clay (CH)
- 🟫 (4) silty clay to clay (CL-CH)
- 🟫 (5) clayey silt to silty clay (MH-CL)
- 🟩 (6) sandy silt to clayey silt (ML-MH)
- 🟦 (7) silty sand to sandy silt (SM-ML)
- 🟪 (8) sand to silty sand (SM-SP)
- 🟧 (9) sand (SW-SP)
- 🟥 (10) gravel to gravelly sand (SW-GW)
- 🟦 (11) very stiff fine grained* (CH-CL)
- 🟦 (12) sand to clayey sand* (SC-SM)

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.09200002
Operator: D. Garza
Location: Del Mar, CA

CPT Number: CPT-20-003
Date: 19-Feb-2020
Elevation: 0.00

Coordinates: 32.973928, -117.268763
Cone Number: CP15-CF75PB7SN2-P1E1 2999



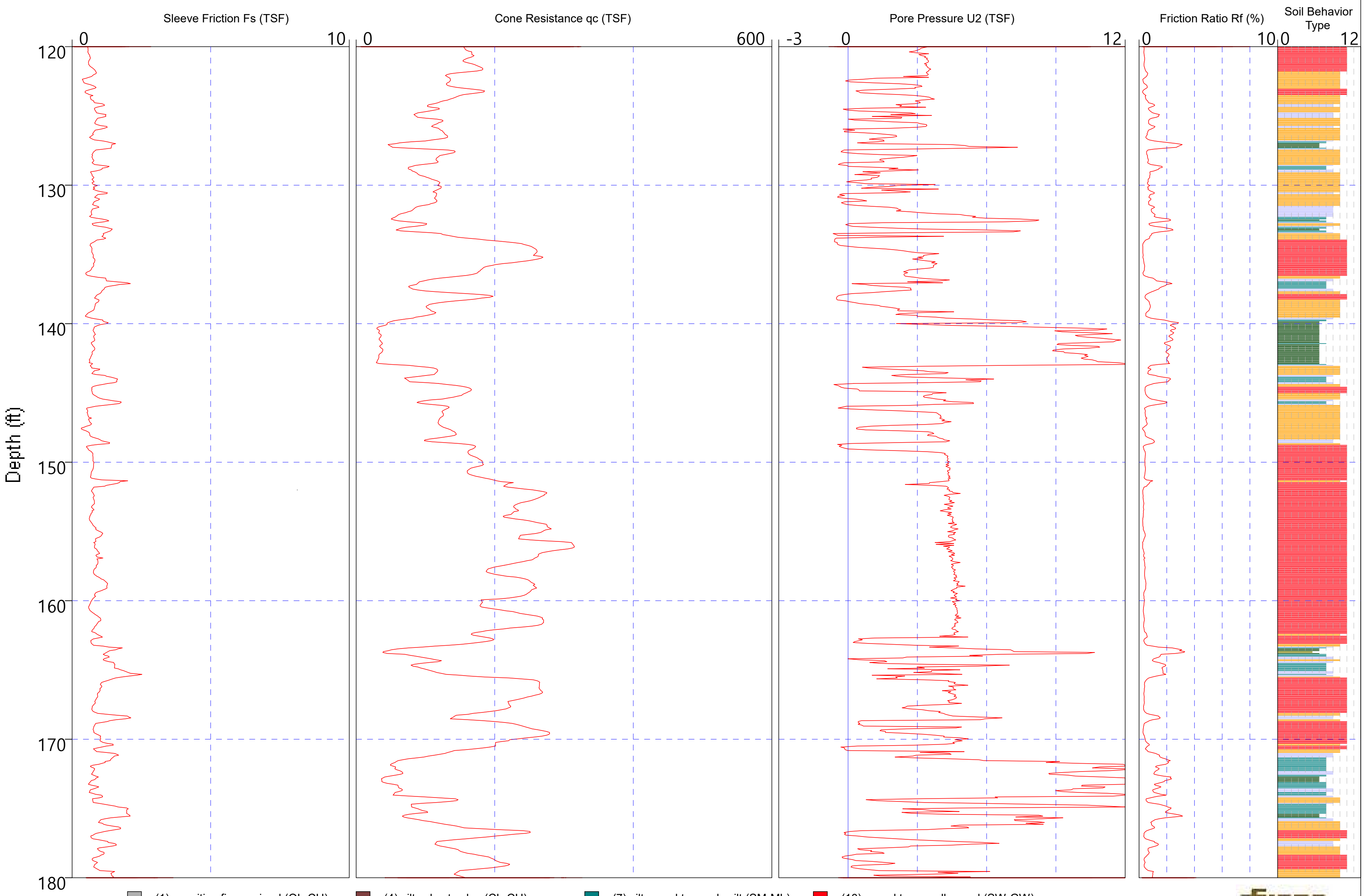
- ▨ (1) sensitive fine grained (OL-CH)
- ▨ (2) organic material (OL-OH)
- ▨ (3) clay (CH)
- ▨ (4) silty clay to clay (CL-CH)
- ▨ (5) clayey silt to silty clay (MH-CL)
- ▨ (6) sandy silt to clayey silt (ML-MH)
- ▨ (7) silty sand to sandy silt (SM-ML)
- ▨ (8) sand to silty sand (SM-SP)
- ▨ (9) sand (SW-SP)
- 🟥 (10) gravel to gravelly sand (SW-GW)
- ▨ (11) very stiff fine grained* (CH-CL)
- ▨ (12) sand to clayey sand* (SC-SM)

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.09200002
Operator: D. Garza
Location: Del Mar, CA

CPT Number: CPT-20-003
Date: 19-Feb-2020
Elevation: 0.00

Coordinates: 32.973928, -117.268763
Cone Number: CP15-CF75PB7SN2-P1E1 2999



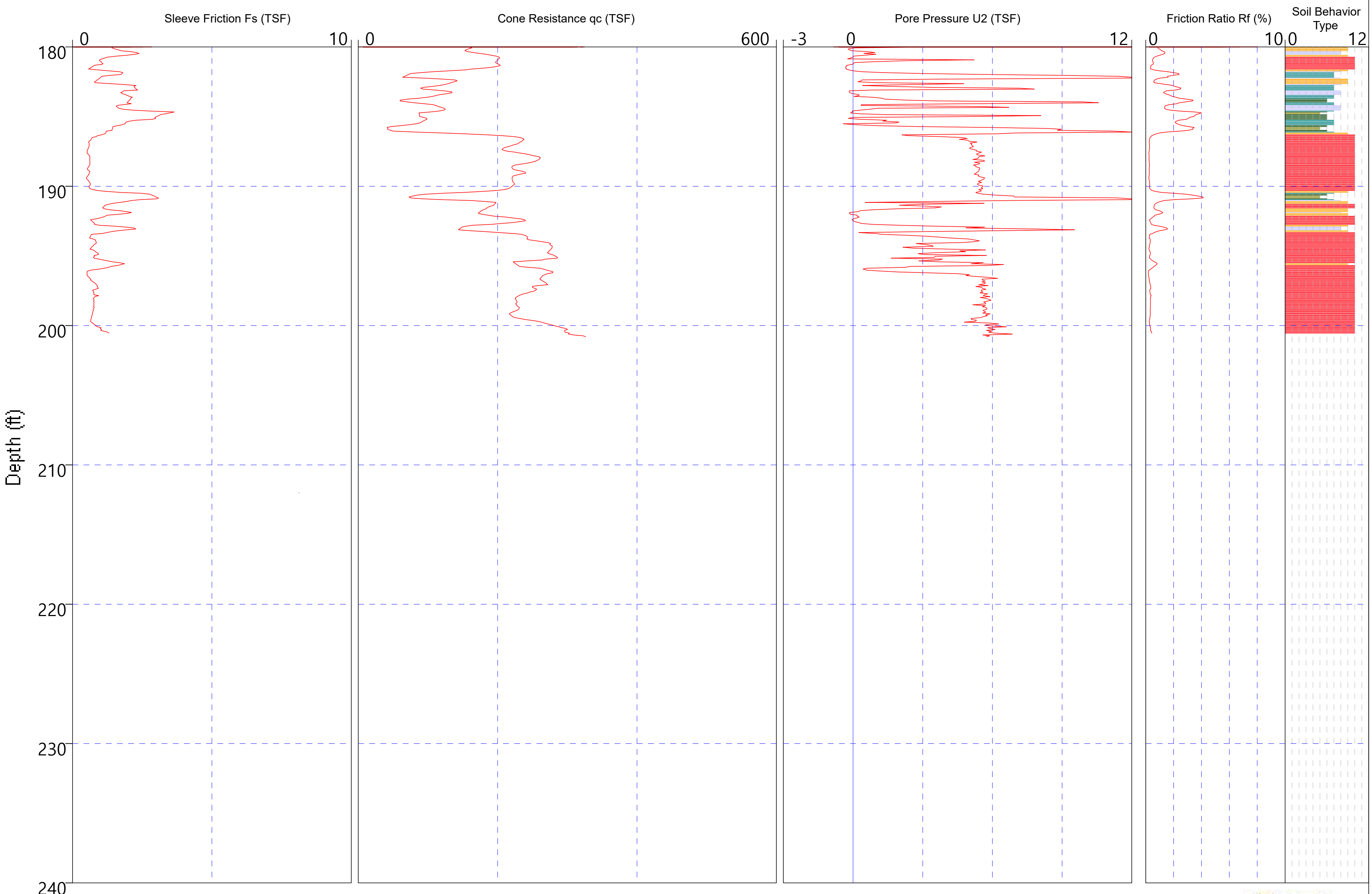
(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.09200002
Operator: D. Garza
Location: Del Mar, CA

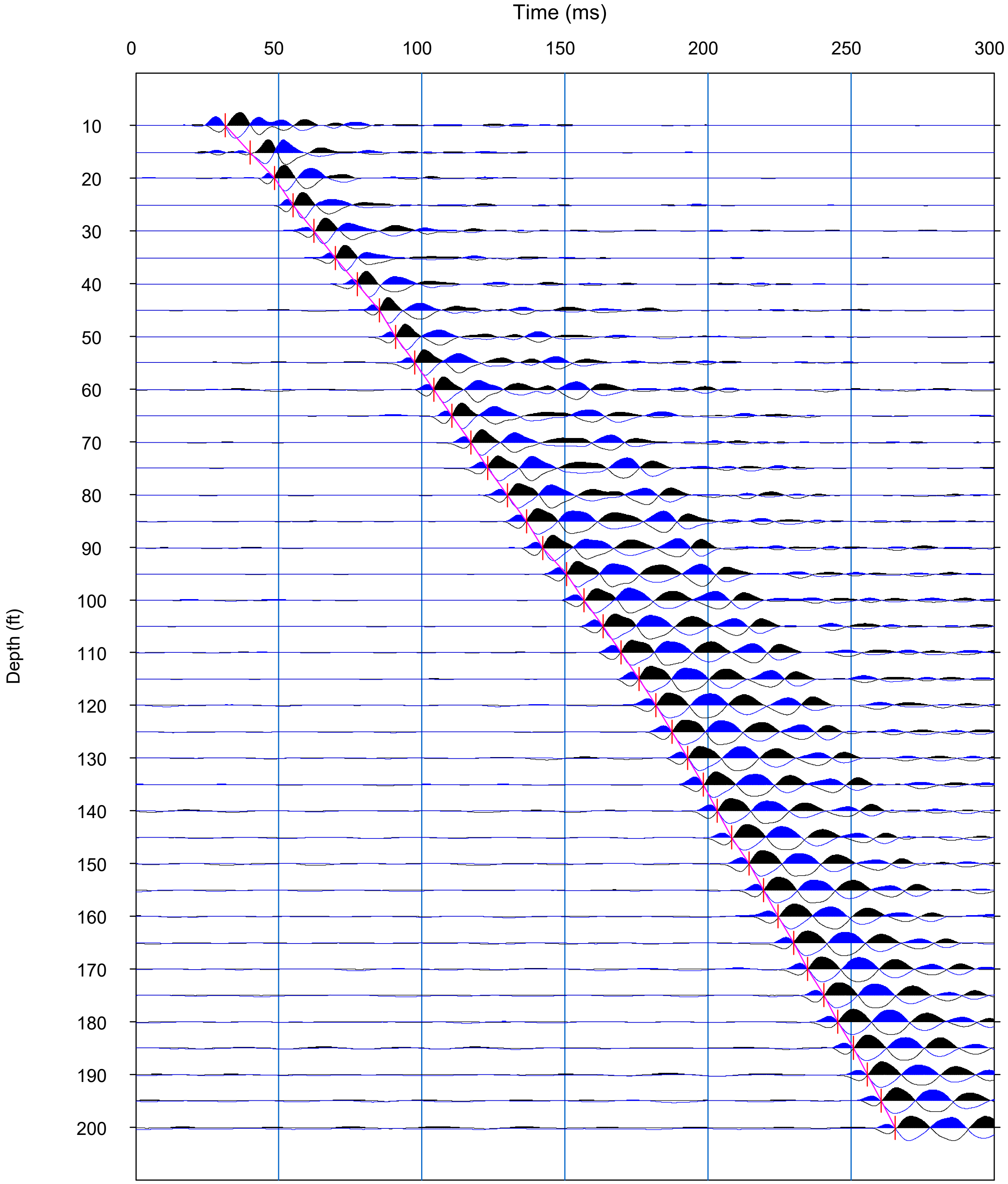
CPT Number: CPT-20-003
Date: 19-Feb-2020
Elevation: 0.00

Coordinates: 32.973928, -117.268763
Cone Number: CP15-CF75PB7SN2-P1E1 2999



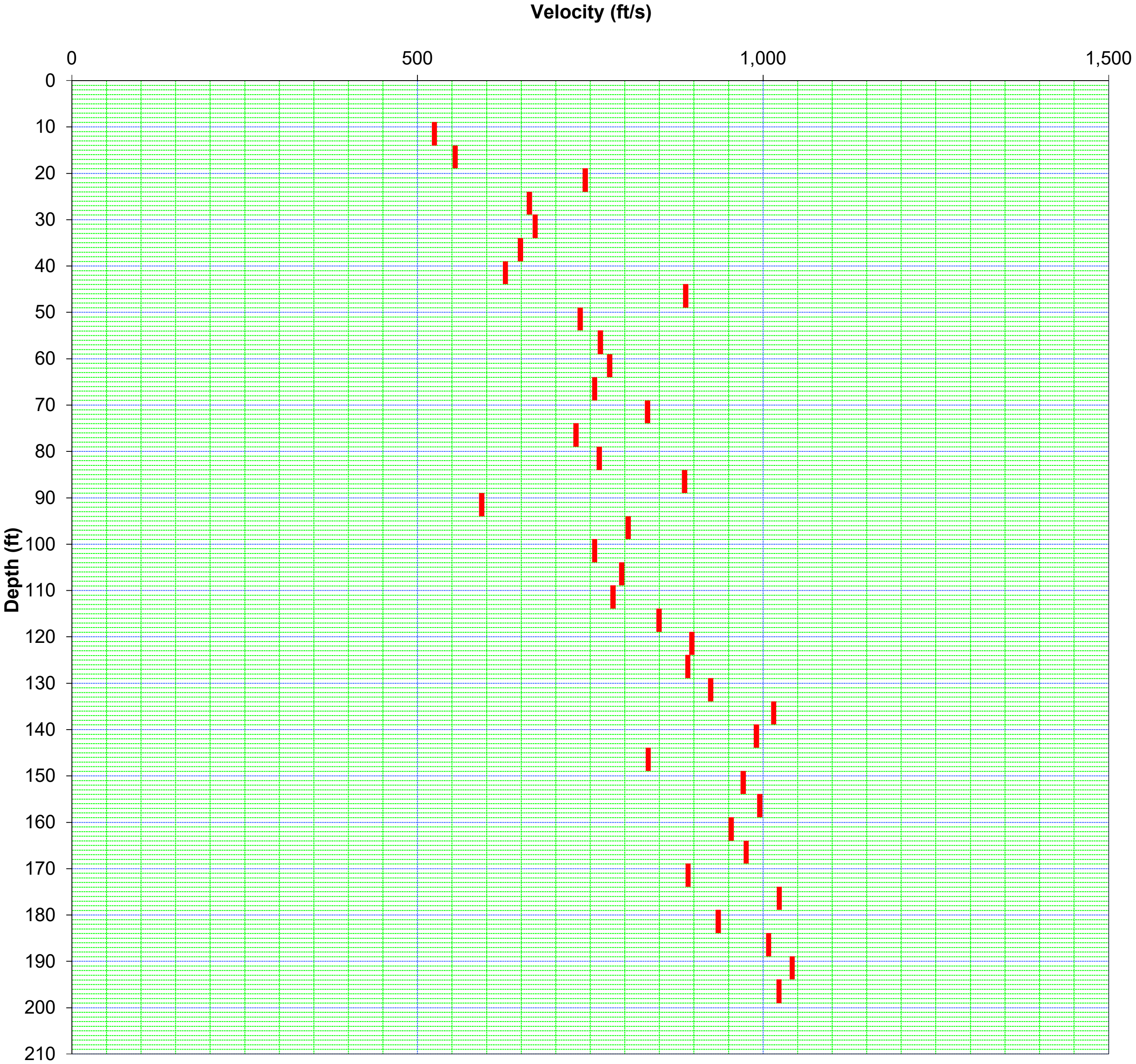
■ (1) sensitive fine grained (OL-CH)
■ (2) organic material (OL-OH)
■ (3) clay (CH)
■ (4) silty clay to clay (CL-CH)
■ (5) clayey silt to silty clay (MH-CL)
■ (6) sandy silt to clayey silt (ML-MH)
■ (7) silty sand to sandy silt (SM-ML)
■ (8) sand to silty sand (SM-SP)
■ (9) sand (SW-SP)
■ (10) gravel to gravelly sand (SW-GW)
■ (11) very stiff fine grained* (CH-CL)
■ (12) sand to clayey sand* (SC-SM)

Robertson et al. 1986 *Overconsolidated or Cemented



CPT-20-003
SHEAR WAVE WAVEFORMS

CAMINO DEL MAR BRIDGE
DEL MAR, CALIFORNIA
KLEINFELDER



CPT-20-003

SHEAR WAVE VELOCITIES

CAMINO DEL MAR BRIDGE
DEL MAR, CALIFORNIA
KLEINFELDER

APPENDIX C

LABORATORY TEST RESULTS

APPENDIX C
LABORATORY TEST RESULTS

Laboratory tests were performed on selected bulk and drive samples from our borehole explorations to estimate engineering characteristics of the various earth materials encountered. Testing was performed in accordance with ASTM and Caltrans standards and are presented in herein.

MOISTURE CONTENT AND DRY UNIT WEIGHT

Natural moisture content and dry unit weight tests were performed on selected bulk and drive samples collected from the boreholes in accordance with ASTM D2216 and D7263, respectively. The results are presented on the boring logs in Appendix A and in Appendix C as Figures C-1 through C-3.

GRADATION ANALYSIS

Sieve analyses were performed on selected samples of the materials encountered at the site to evaluate the gradation characteristics of the soil and to aid in classification. The tests were performed in general accordance with ASTM D1140 for percent finer than No. 200 sieve tests and ASTM D6913 for full gradation analyses. The results are presented in Appendix C as Figures C-4 through C-24.

ATTERBERG LIMITS

Atterberg limit tests were performed on fine-grained portions of selected soil samples to evaluate the plasticity characteristics (liquid limit, plastic limit, and plasticity index) of the soil and to aid in its classification. The tests were performed in general accordance with ASTM D4318. The results are presented in Appendix C as Figures C-25 and C-26.

TRIAXIAL COMPRESSION (UU) TEST

Three unconfined, unconsolidated (UU) triaxial compression tests were performed on selected soil samples from the borings performed at the site. The test procedures were performed in general accordance with the ASTM D2850. The results are presented in Appendix C as Figures C-27 through C-29.

UNCONFINED COMPRESSION TEST

An unconfined compression test was performed on a soil sample from boring R-20-002. The test procedures were performed in general accordance with the ASTM D2166. The results are presented in Appendix C as Figure C-30.

R-VALUE

Two R-Value tests were performed on selected bulk samples to evaluate resistance values of the near surface soils. The tests were performed using modified effort in general accordance with ASTM D2844. The results are presented in Appendix C and Figures C-31 and C-32.

CORROSION TESTS

A series of chemical tests were performed on four selected bulk and driven samples of the near surface and at-depth soils to estimate pH, minimum resistivity, and sulfate and chloride contents. The test procedures were in general accordance with the California Tests 417, 422, and 643. The test results are provided in Appendix C as Figures C-33 through C-36.

DIRECT SHEAR TEST

Five direct shear strength tests were performed on selected driven soil samples from the borings. The test procedures were performed in general accordance with the ASTM D3080. The results are presented in Appendix C as Figures C-37 through C-41.

Date Tested 3/16-20/2020

Boring No.	**R-20-001**	**R-20-001**	**R-20-001**	**R-20-001**	**R-20-001**
Sample No.	**S1**	**S3**	**S5**	**S6**	**S7**
Depth, ft.	**0.5-5**	**8-9.5**	**12-13.5**	**14-15.5**	**16-17.5**
Wet Weight, g	604.3	324.4	129.3	347.4	432.6
Dry Weight, g	587.1	309.4	113.2	278.3	340.1
Moisture Content, %	2.9	4.8	14.2	24.8	27.2
Sample Description	Dark brown poorly graded sand with silt	Brown poorly graded sand with silt	Dark brown poorly graded sand with silt	Dark gray poorly graded sand with silt	Dark gray poorly graded sand with silt

Boring No.	**R-20-001**	**R-20-002**	**R-20-002**	**R-20-002**	**R-20-002**
Sample No.	**S8**	**S1**	**S3**	**S5**	**S7**
Depth, ft.	**18-19.5**	**0.5-4**	**7-8.5**	**11-12.5**	**15-16.5**
Wet Weight, g	341.8	617.3	234.0	361.3	353.8
Dry Weight, g	272.3	594.4	224.4	297.9	279.6
Moisture Content, %	25.5	3.9	4.3	21.3	26.5
Sample Description	Dark gray poorly graded sand with silt	Dark brown poorly graded sand	Light gray poorly graded sand with silt	Dark brown poorly graded sand with silt	Dark gray poorly graded sand

Performed in General Accordance with ASTM D2216

KLEINFELDER
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CHECKED BY: J.B Tech T.C.
JOB NUMBER: 20180876.001A DATE: 6-Apr-20

Moisture Content Determination

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-1

Date Tested 3/16-20/2020

Boring No.	**R-20-002**				
Sample No.	**S8**				
Depth, ft.	**17-18.5**				
Wet Weight, g	349.6				
Dry Weight, g	276.1				
Moisture Content, %	26.6				
Sample Description	Dark gray poorly graded sand with silt				

Boring No.					
Sample No.					
Depth, ft.					
Wet Weight, g					
Dry Weight, g					
Moisture Content, %					
Sample Description					

Performed in General Accordance with ASTM D2216

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CHECKED BY: J.B. Tech T.C.
JOB NUMBER: 20180876.001A DATE: 2-Apr-20

Moisture Content Determination

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-2

Date Tested : 3/10-20/2020

Boring #	Sample #	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	Description
R-20-001	S11	30-31.5	65.6	55.2%	Dark gray sandy clay
R-20-001	S13	50-51.5	110.3	20.6%	Gray silty sand
R-20-001	S15	60-61.5	101.5	24.5%	Gray silty sand
R-20-001	S17	70-71.5	108.3	21.1%	Gray poorly graded sand with silt
R-20-001	S19	80-81.5	106.7	20.5%	Gray poorly graded sand with silt
R-20-001	S25	110-111.5	92.1	31.5%	Gray silty sand
R-20-001	S27	120-121.5	102.7	25.1%	Gray silty sand
R-20-002	S10	25-26.5	99.9	26.4%	Dark gray silty sand
R-20-002	S12	35-36.5	101.3	24.2%	Dark gray silty sand
R-20-002	S18	65-66.5	96.0	28.9%	Dark gray poorly graded sand with silt
R-20-002	S14	45-46.5	105.3	23.5%	Dark gray poorly graded sand with silt
R-20-002	S16	55-56.5	92.2	36.7%	Dark gray poorly graded sand with silt
R-20-002	S20	75-76.5	95.5	29.9%	Dark gray poorly graded sand with silt
R-20-002	S30	125-126.5	94.2	30.5%	Dark gray silty sand
R-20-002	S34	145-146.5	91.9	28.7%	Dark gray silty sand
R-20-002	S24	95-96.5	94.5	28.1%	Dark gray silty sand
R-20-002	S28	115-116.5	104.4	29.4%	Dark gray silty sand
R-20-002	S41	181-181.5	106.6	17.9%	Dark gray silty sand

Performed in General Accordance with ASTM D7263 B and D2216

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CHECKED BY: J.B. TECH: M.S.L
JOB NUMBER: 20180876.001A DATE: 2-Apr-20

Dry Density and Moisture Content

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-3

Date Tested 3/10-20/2020

Boring No	R-20-001	R-20-001	R-20-001	R-20-001	R-20-001
Sample No.	S3	S7	S12	S25	S31
Depth, ft.	8-9.5	16-17.5	35-36.5	110-111.5	150-151
Dry Weight before wash, g	309.4	340.1	148.2	265.4	235.1
Dry Weight After Wash, g	274.4	326.9	56.6	242.8	68.2
Weight Loss, No. 200, g	35.0	13.2	91.6	22.6	166.9
Wash No. 200, %	**11.3**	**3.9**	**61.8**	**8.5**	**71.0**
Sample Description	Brown poorly graded sand with silt	Dark gray poorly graded sand	Dark gray sandy fat clay	Dark gray poorly graded sand with silt	Gray brown sandy fat clay

Boring No	R-20-002	R-20-002	R-20-002	R-20-002	R-20-002
Sample No.	S3	S7	S11	S17	S21
Depth, ft.	7-8.5	15-16.5	30-31.5	60-61.5	80-81.5
Dry Weight before wash, g	224.4	279.6	255.2	318.5	286.4
Dry Weight After Wash, g	211.9	270.6	235.9	282.5	170.6
Weight Loss, No. 200, g	12.5	9.0	19.3	36.0	115.8
Wash No. 200, %	**5.6**	**3.2**	**7.6**	**11.3**	**40.4**
Sample Description	Light gray poorly graded sand with silt	Dark gray poorly graded sand	Dark gray poorly graded sand with silt	Dark gray poorly graded sand with silt	Dark gray silty sand

TEST PERFORMED IN ACCORDANCE WITH ASTM D 1140

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Materials Finer than 75 um (No 200) Sieve

FIGURE

C-4

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

CHECKED BY: J.B. Tech T.C.

JOB NUMBER: 20180876.001A DATE: 1-Apr-20

Date Tested 3/10-20/2020

Boring No	R-20-002	R-20-002			
Sample No.	S23	S33			
Depth, ft.	90-91.5	140-141.5			
Dry Weight before wash, g	253.5	288.9			
Dry Weight After Wash, g	78.3	180.2			
Weight Loss, No. 200, g	175.2	108.7			
Wash No. 200, %	**69.1**	**37.6**			
Sample Description	Dark gray sandy clay	Dark gray silty sand			

Boring No					
Sample No.					
Depth, ft.					
Dry Weight before wash, g					
Dry Weight After Wash, g					
Weight Loss, No. 200, g					
Wash No. 200, %					
Sample Description					

TEST PERFORMED IN ACCORDANCE WITH ASTM D 1140

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CHECKED BY: J.B. Tech T.C.

JOB NUMBER: 20180876.001A DATE: 1-Apr-20

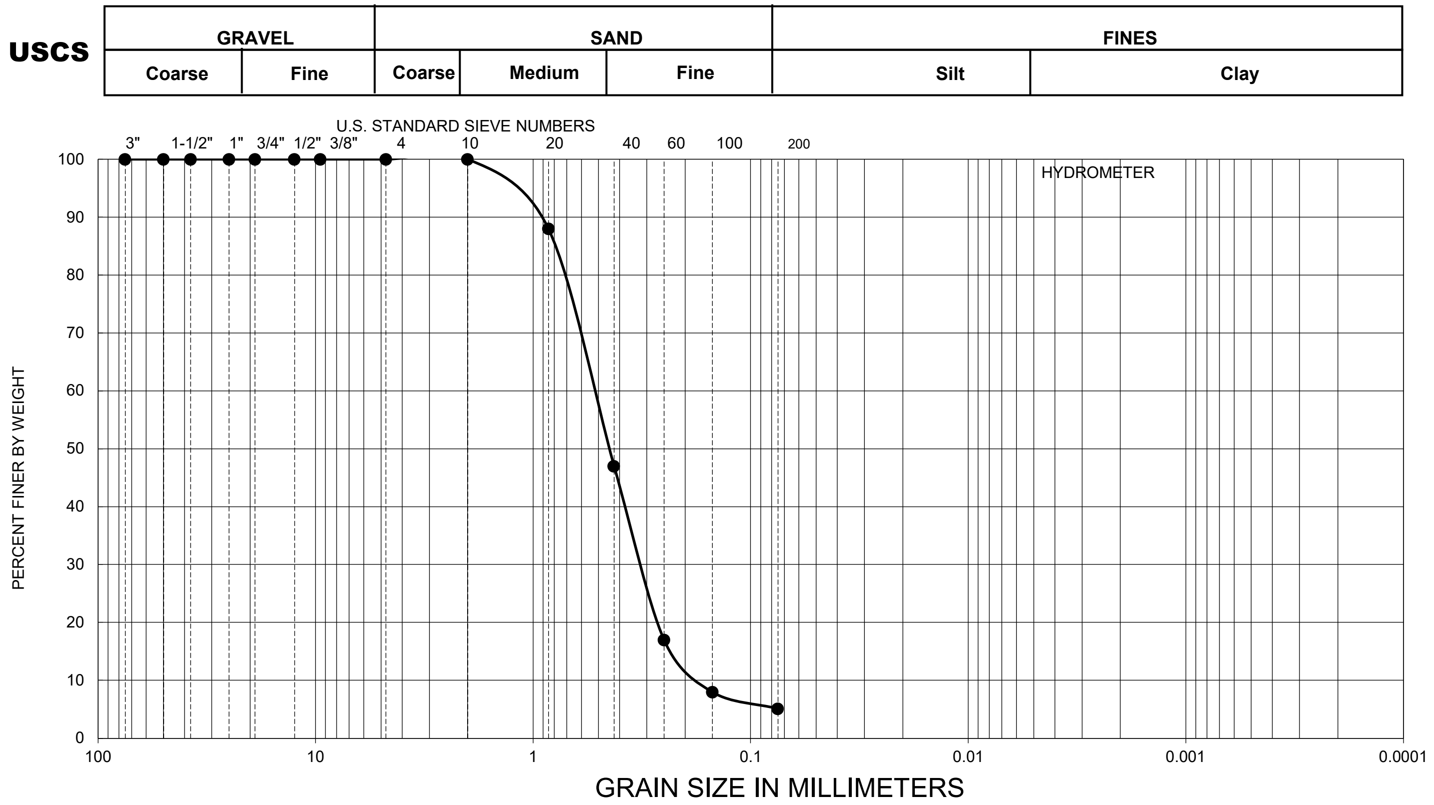
Materials Finer than 75 um (No 200) Sieve

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-5

Date Tested: **3/12/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-001	S1	0.5-5	5.1	SP-SM

Sample Description	Dark brown Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	88
	No. 40	0.425 mm	47
	No. 60	0.25 mm	17
	No 100	0.15 mm	8
	No 200	.075 mm	5.1

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	T.C.
Project No.	20180876.001A	Date:	1-Apr-20

GRADATION TEST RESULTS

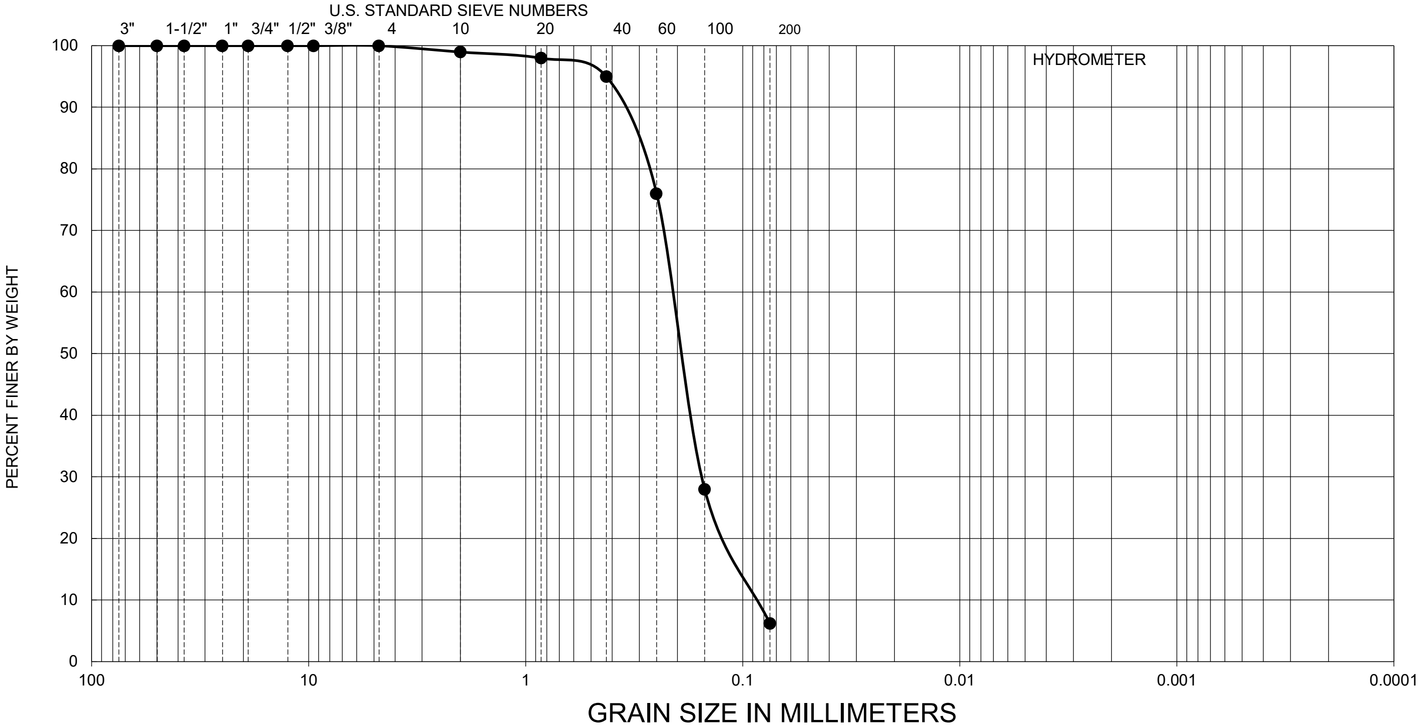
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-6

Date Tested: **3/12/2020**

USCS	**GRAVEL**		**SAND**			**FINES**	
	Coarse	**Fine**	**Coarse**	**Medium**	**Fine**	**Silt**	**Clay**



Boring No.	**Sample No.**	**Depth (ft)**	**Passing 200 (%)**	**USCS Classification**
R-20-001	S5	12-13.5	6.2	SP-SM

Sample Description	Dark brown Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	99
	No. 20	0.85 mm	98
	No. 40	0.425 mm	95
	No. 60	0.25 mm	76
	No 100	0.15 mm	28
	No 200	.075 mm	6.2

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Project No.	20180876.001A	Date:	2-Apr-20

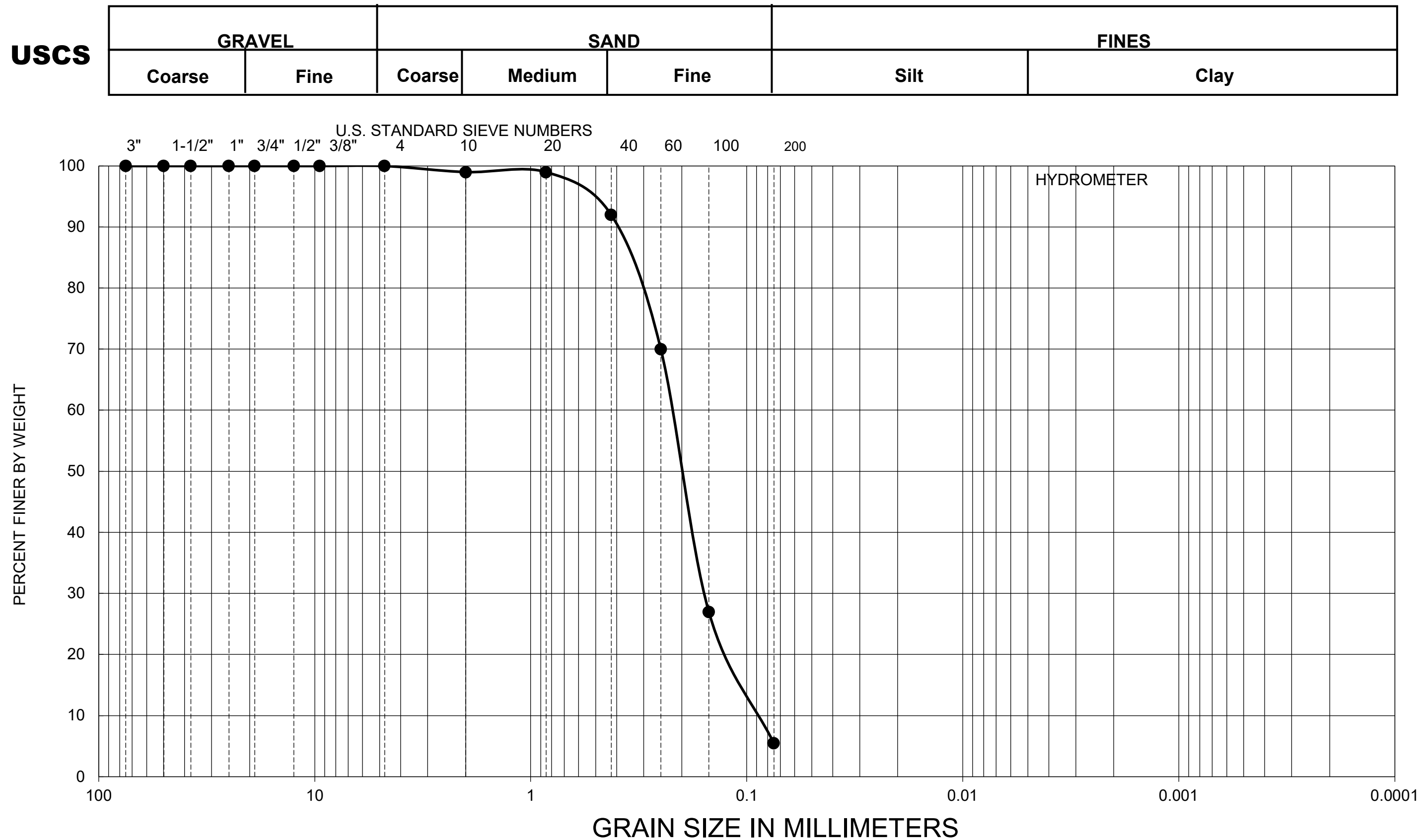
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-7

Date Tested: 3/11/2020



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-001	S9	20-21.5	5.5	SP-SM

Sample Description	Dark brown Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	99
	No. 20	0.85 mm	99
	No. 40	0.425 mm	92
	No. 60	0.25 mm	70
	No 100	0.15 mm	27
	No 200	.075 mm	5.5

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	T.C.
Project No.	20180876.001A	Date:	2-Apr-20

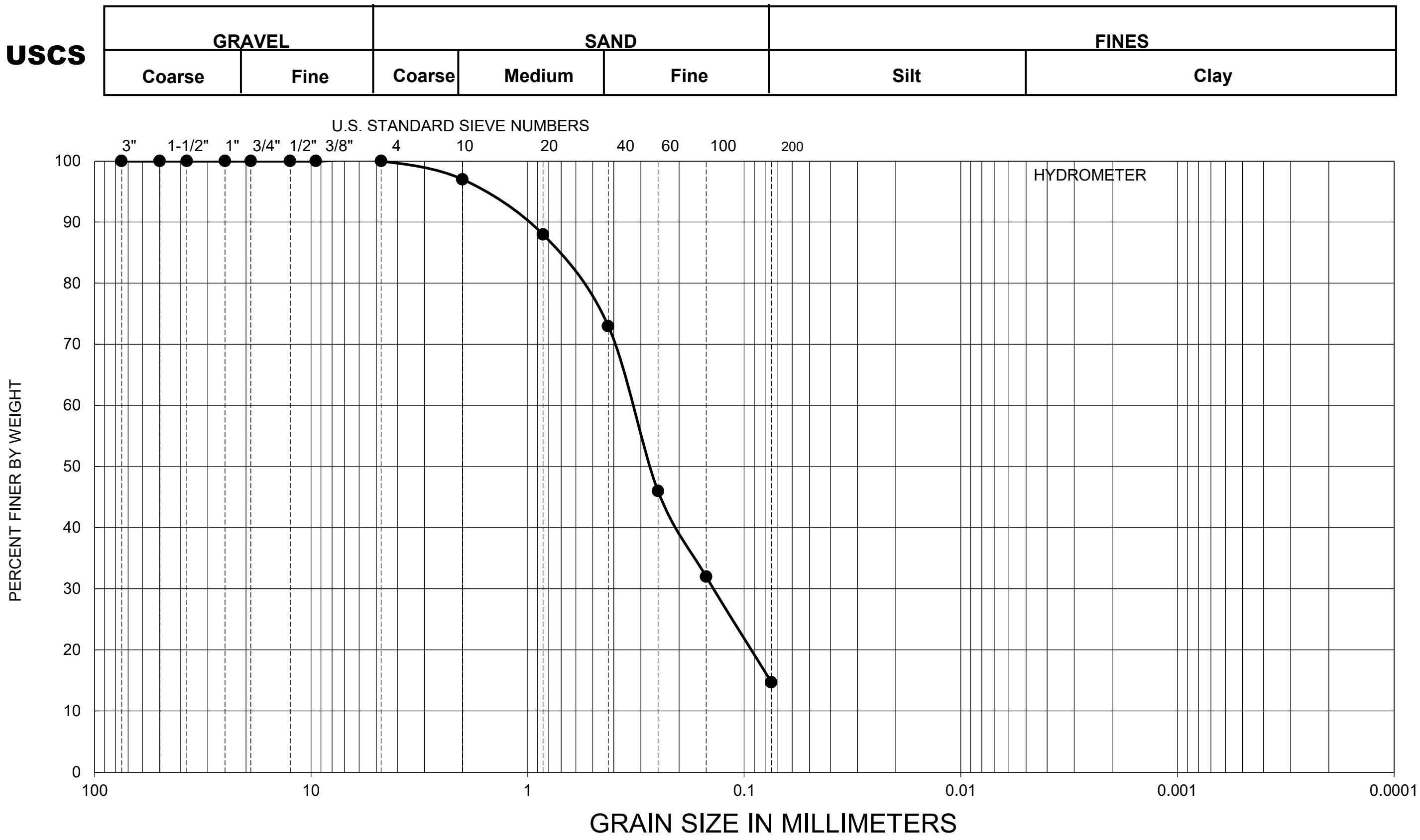
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-8

Date Tested: **3/19/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-001	S13	50-51.5	14.7	SM

Sample Description	Dark brown Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	97
	No. 20	0.85 mm	88
	No. 40	0.425 mm	73
	No. 60	0.25 mm	46
	No 100	0.15 mm	32
	No 200	.075 mm	14.7

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

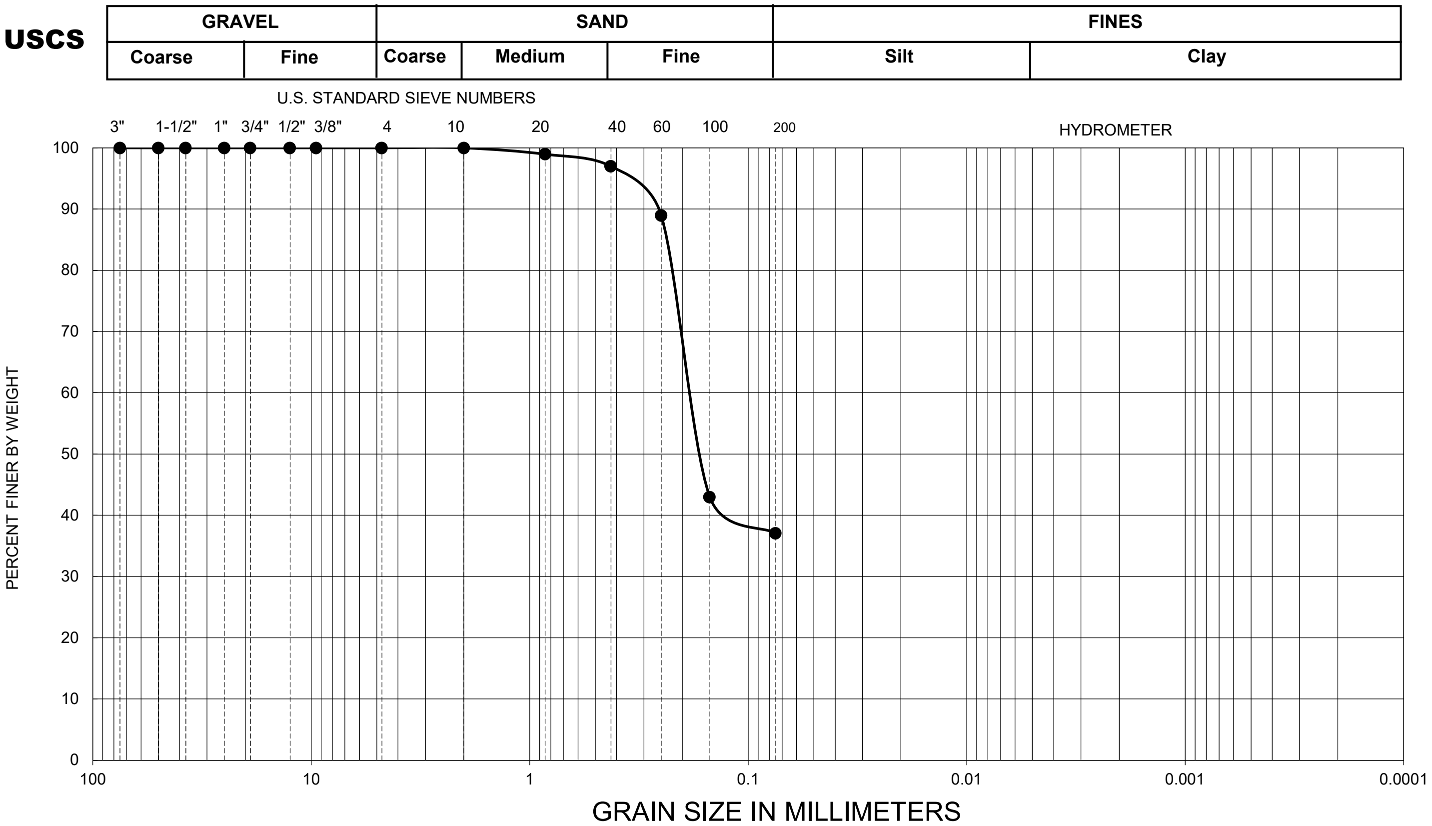
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-9

Date Tested: **3/19/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-001	S15	60-61.5	37.1	SM

Sample Description	Dark gray Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	99
	No. 40	0.425 mm	97
	No. 60	0.25 mm	89
	No 100	0.15 mm	43
	No 200	.075 mm	37.1

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Project No.	20180876.001A	Date:	2-Apr-20

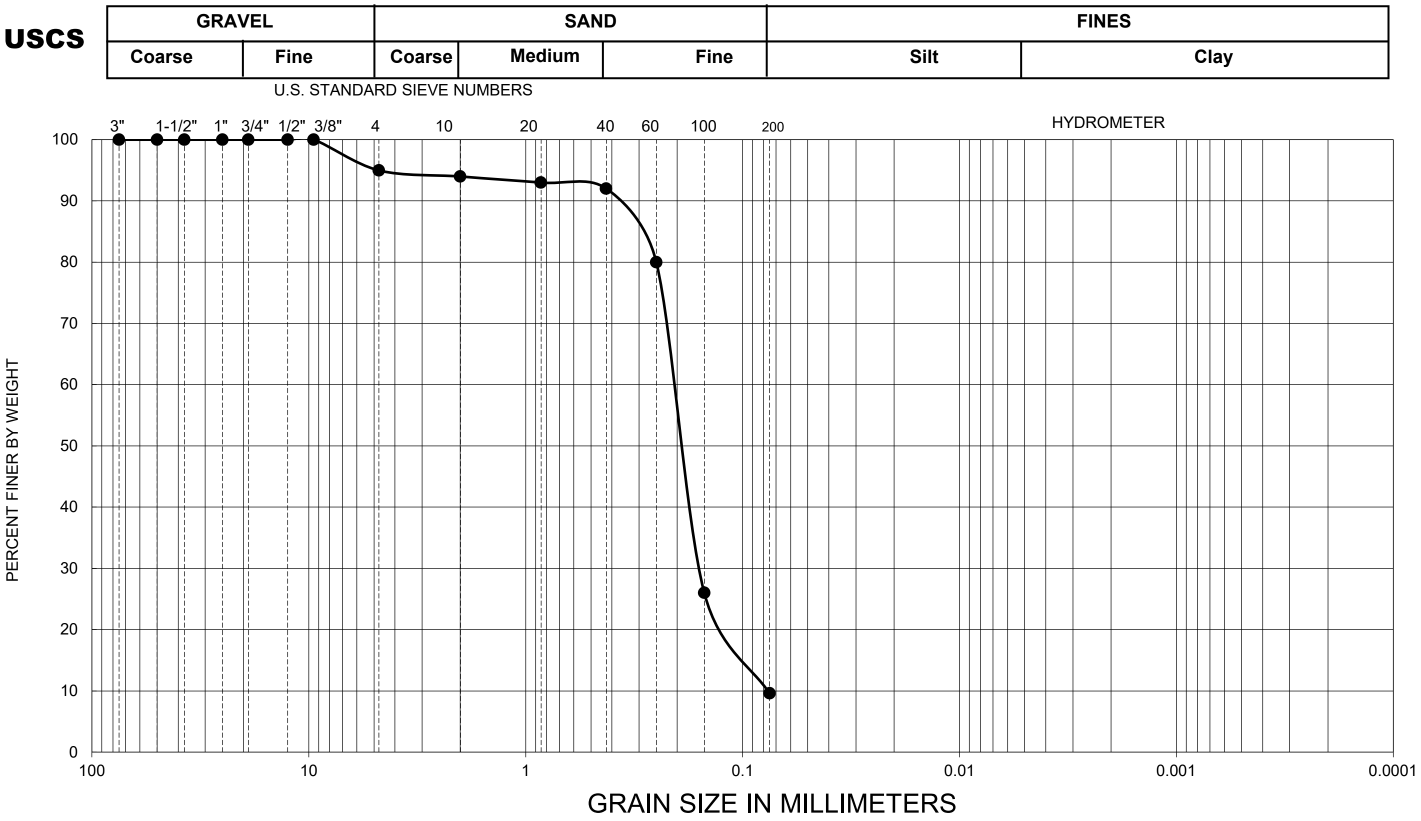
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-10

Date Tested: **3/19/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-001	S17	70-71.5	9.6	SP-SM

Sample Description	Dark gray Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	95
	No. 10	2.0 mm	94
	No. 20	0.85 mm	93
	No. 40	0.425 mm	92
	No. 60	0.25 mm	80
	No 100	0.15 mm	26
	No 200	.075 mm	9.6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Project No.	20180876.001A	Date:	2-Apr-20

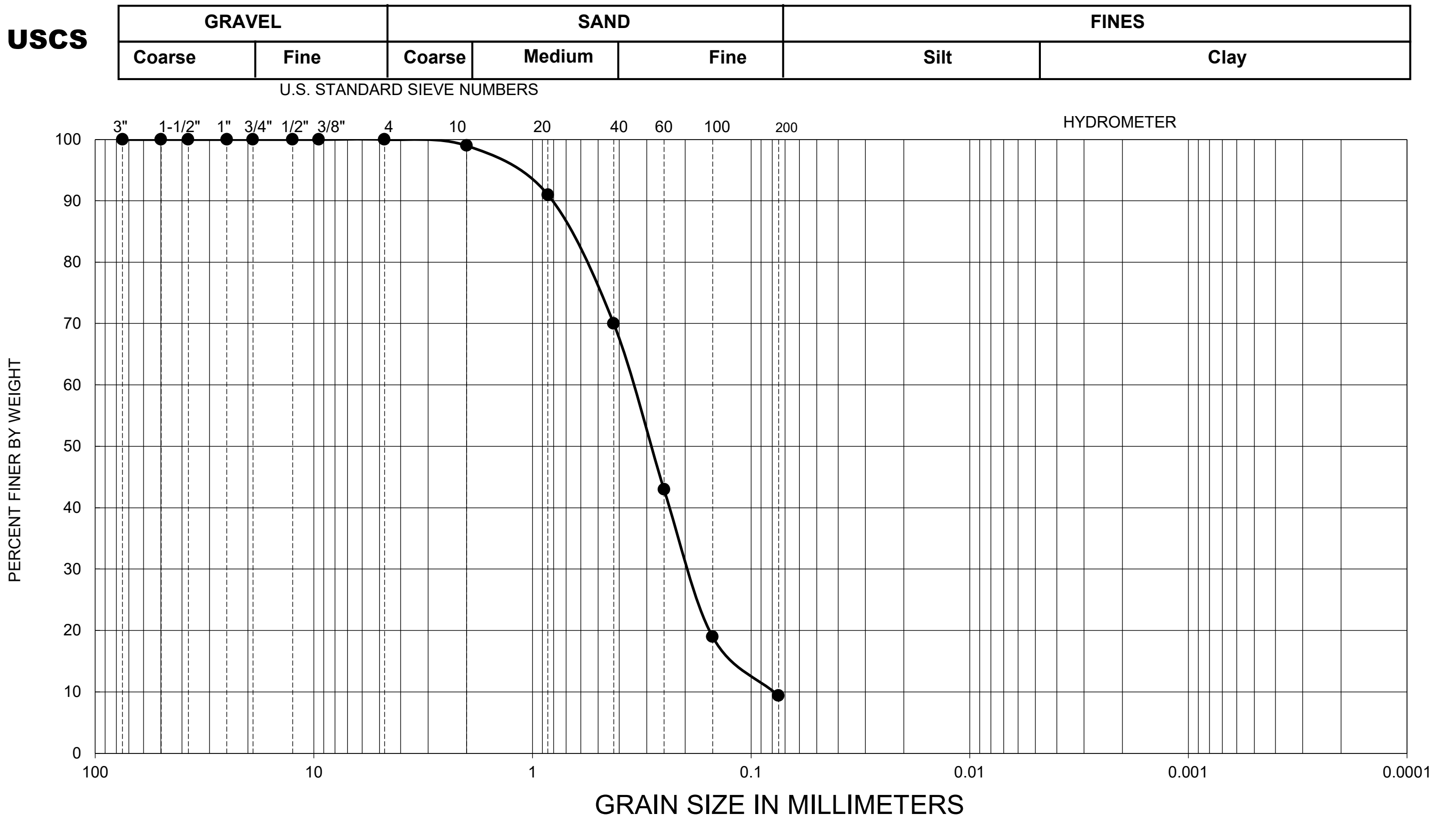
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-11

Date Tested: **3/19/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-001	S19	80-81.5	9.4	SP-SM

Sample Description	Dark gray Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	99
	No. 20	0.85 mm	91
	No. 40	0.425 mm	70
	No. 60	0.25 mm	43
	No 100	0.15 mm	19
	No 200	.075 mm	9.4

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Project No.	20180876.001A	Date:	2-Apr-20

GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

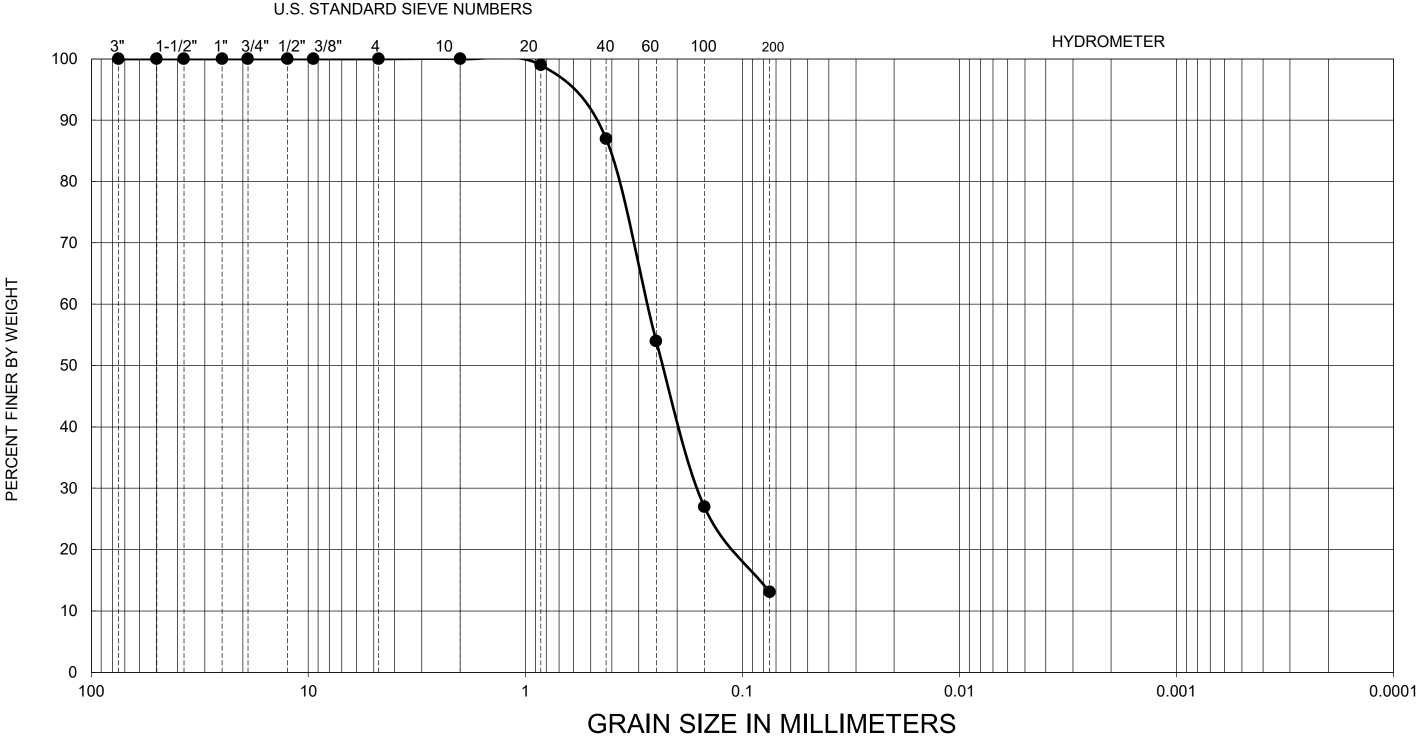
FIGURE

C-12

Date Tested: **3/19/2020**

USCS

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-001	S27	120-121.5	13.1	SM

Sample Description	Dark gray Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	99
	No. 40	0.425 mm	87
	No. 60	0.25 mm	54
	No 100	0.15 mm	27
	No 200	.075 mm	13.1

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

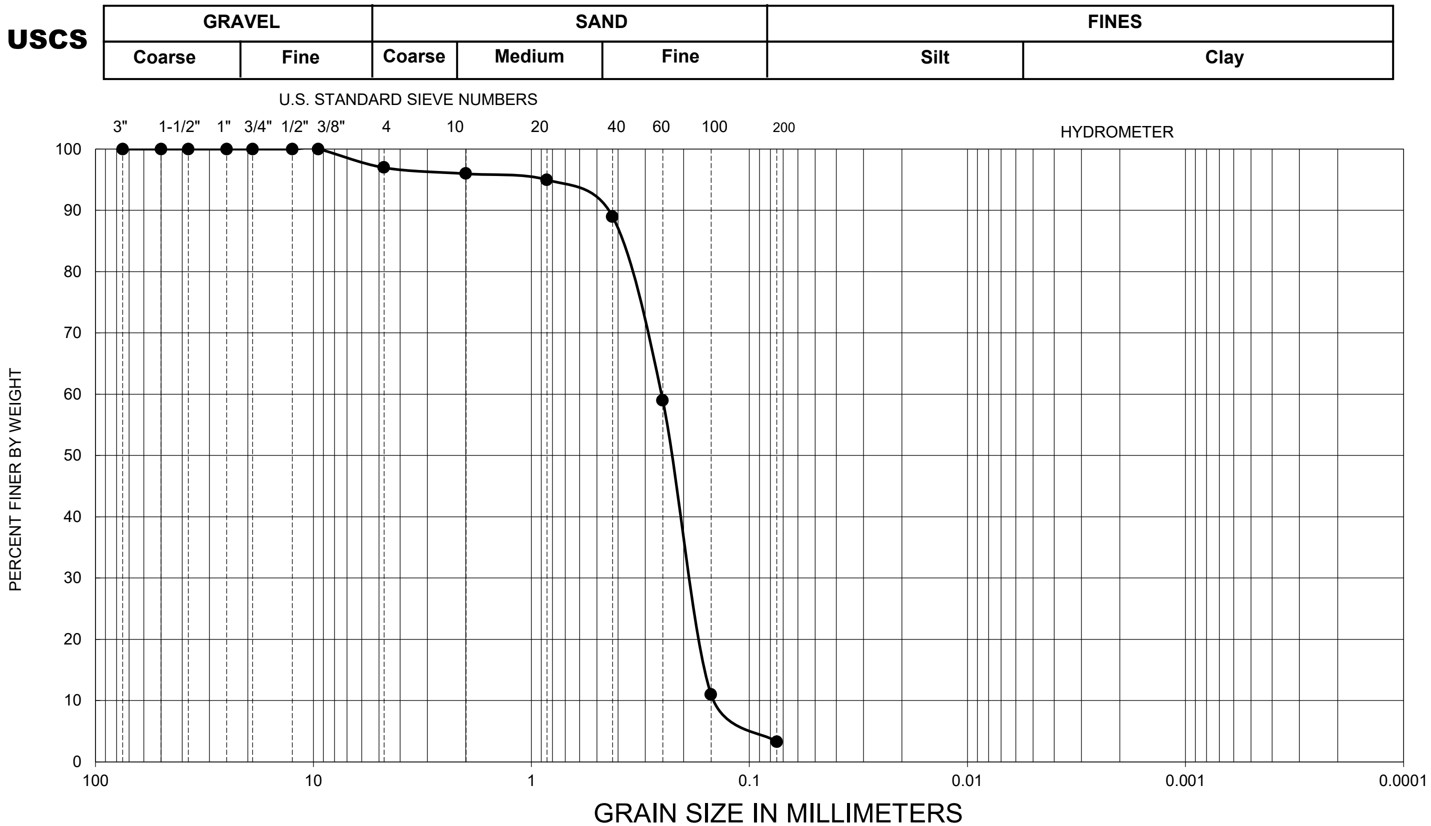
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-13

Date Tested: **3/11/2020**



Boring No.	**Sample No.**	**Depth (ft)**	**Passing 200 (%)**	**USCS Classification**
R-20-002	S1	0.5-4	3.3	SP

Sample Description	Brown Poorly graded sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	97
	No. 10	2.0 mm	96
	No. 20	0.85 mm	95
	No. 40	0.425 mm	89
	No. 60	0.25 mm	59
	No 100	0.15 mm	11
	No 200	.075 mm	3.3

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Project No.	20180876.001A	Date:	2-Apr-20

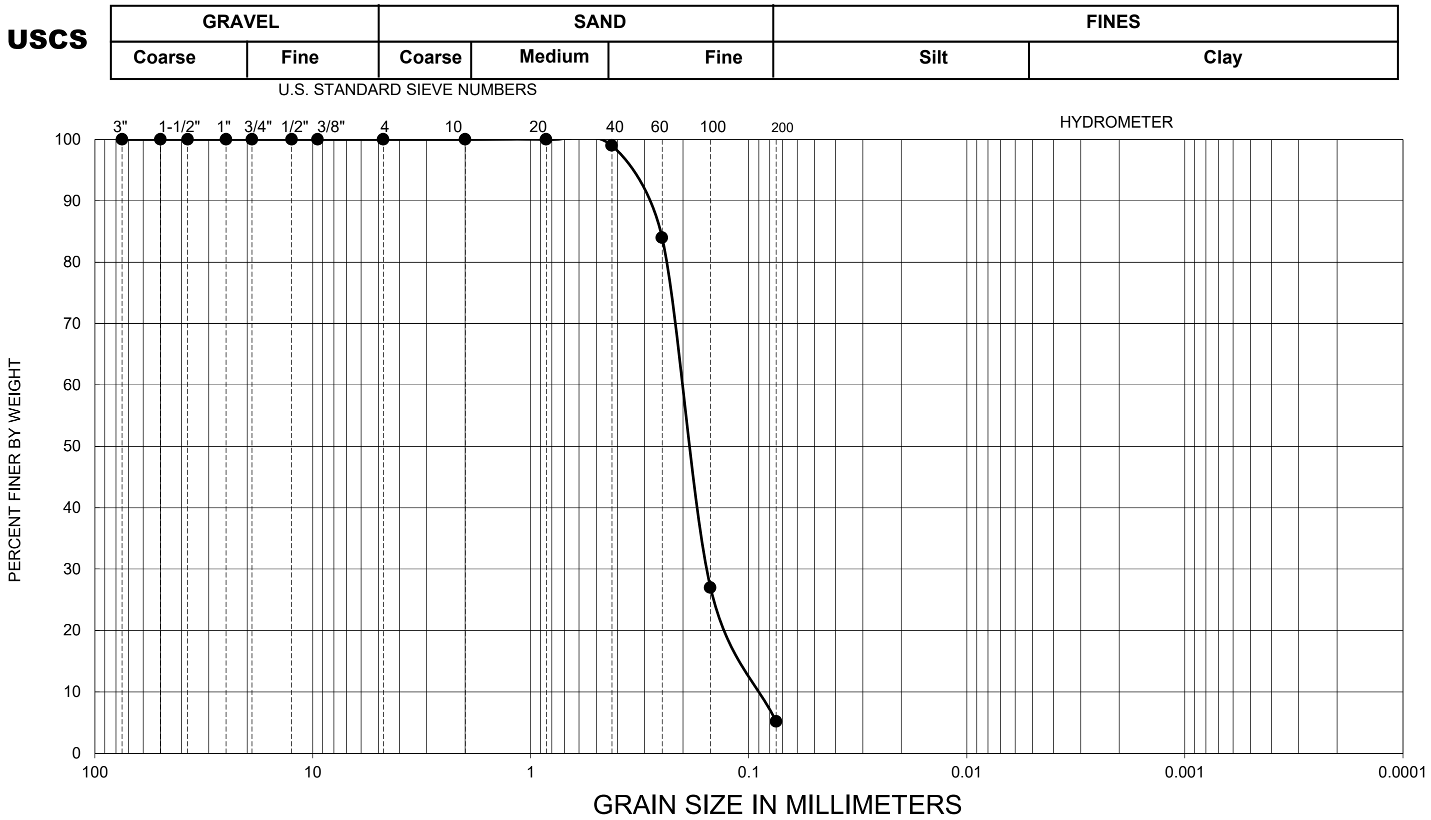
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-14

Date Tested: **3/11/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-002	S5	11-12.5	5.2	SP-SM

Sample Description	Dark gray Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	99
	No. 60	0.25 mm	84
	No 100	0.15 mm	27
	No 200	.075 mm	5.2

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Project No.	20180876.001A	Date:	2-Apr-20

GRADATION TEST RESULTS

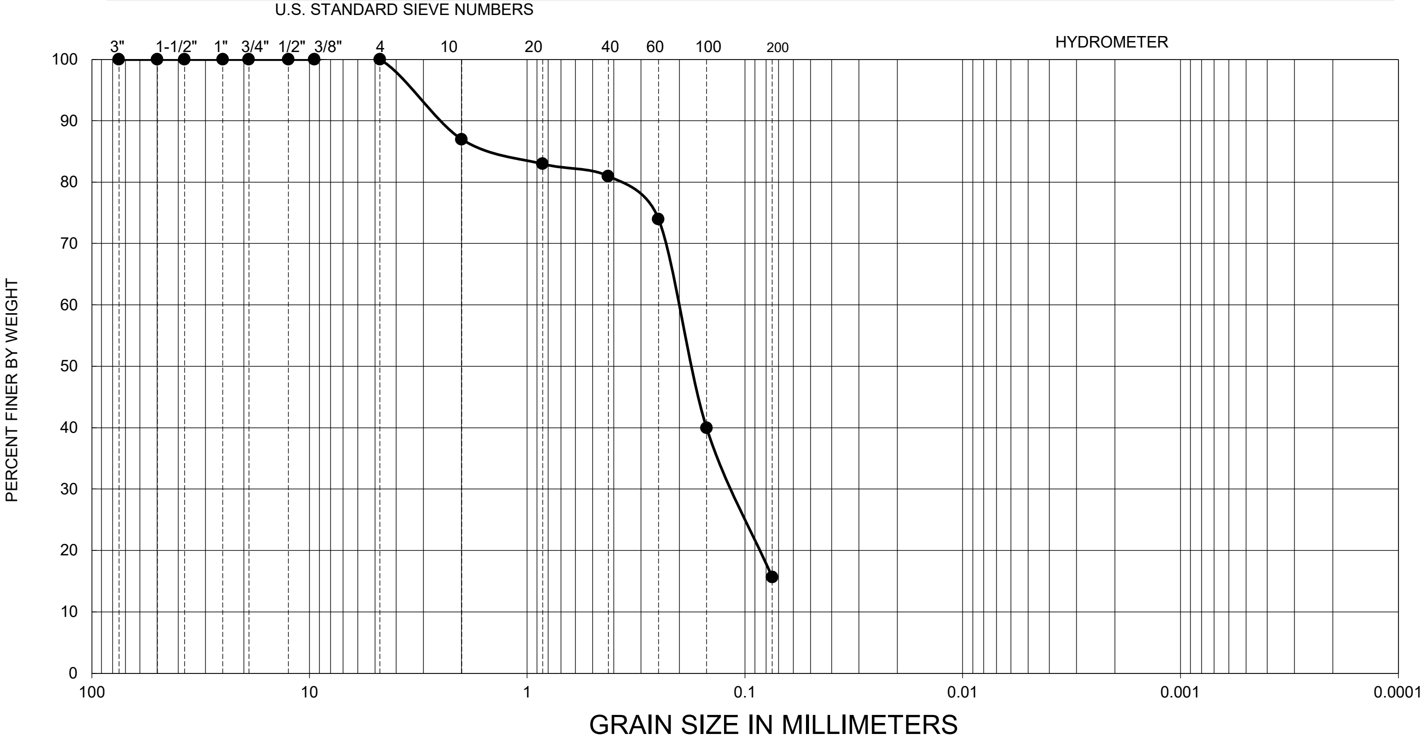
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-15

Date Tested: **3/11/2020**

USCS	**GRAVEL**		**SAND**			**FINES**	
	Coarse	**Fine**	**Coarse**	**Medium**	**Fine**	**Silt**	**Clay**



Boring No.	**Sample No.**	**Depth (ft)**	**Passing 200 (%)**	**USCS Classification**
R-20-002	S9	21-22.5	15.7	SM

Sample Description	Dark gray Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	87
	No. 20	0.85 mm	83
	No. 40	0.425 mm	81
	No. 60	0.25 mm	74
	No 100	0.15 mm	40
	No 200	.075 mm	15.7

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

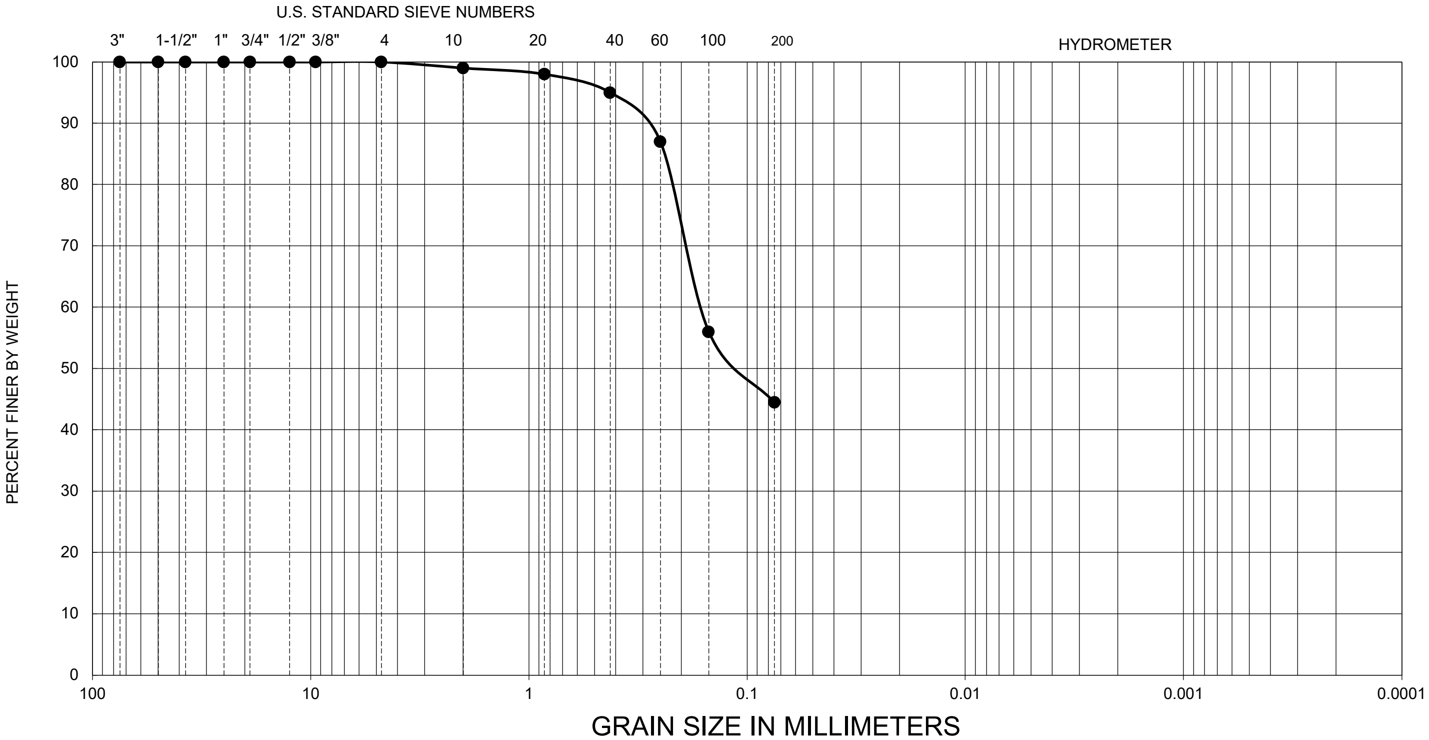
FIGURE

C-16

Date Tested: **3/12/2020**

USCS

GRAVEL		SAND			FINES	
Coarse	**Fine**	**Coarse**	**Medium**	**Fine**	**Silt**	**Clay**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-002	S13	40-41.5	44.5	SM

Sample Description	Dark gray Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	99
	No. 20	0.85 mm	98
	No. 40	0.425 mm	95
	No. 60	0.25 mm	87
	No 100	0.15 mm	56
	No 200	.075 mm	44.5

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

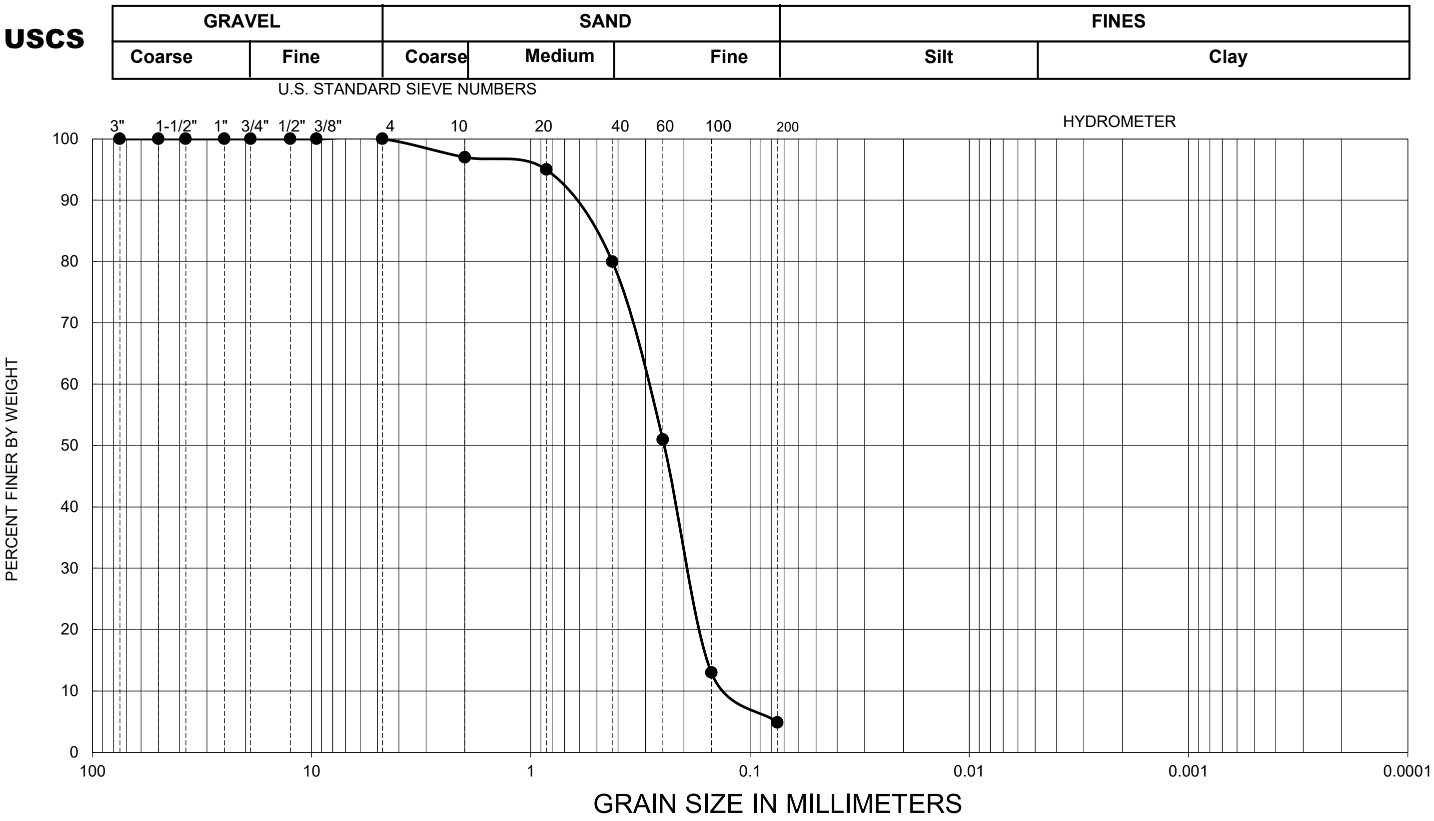
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-17

Date Tested: **3/12/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-002	S15	50-51.5	4.9	SP

Sample Description	Dark gray Poorly graded sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	97
	No. 20	0.85 mm	95
	No. 40	0.425 mm	80
	No. 60	0.25 mm	51
	No 100	0.15 mm	13
	No 200	.075 mm	4.9

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

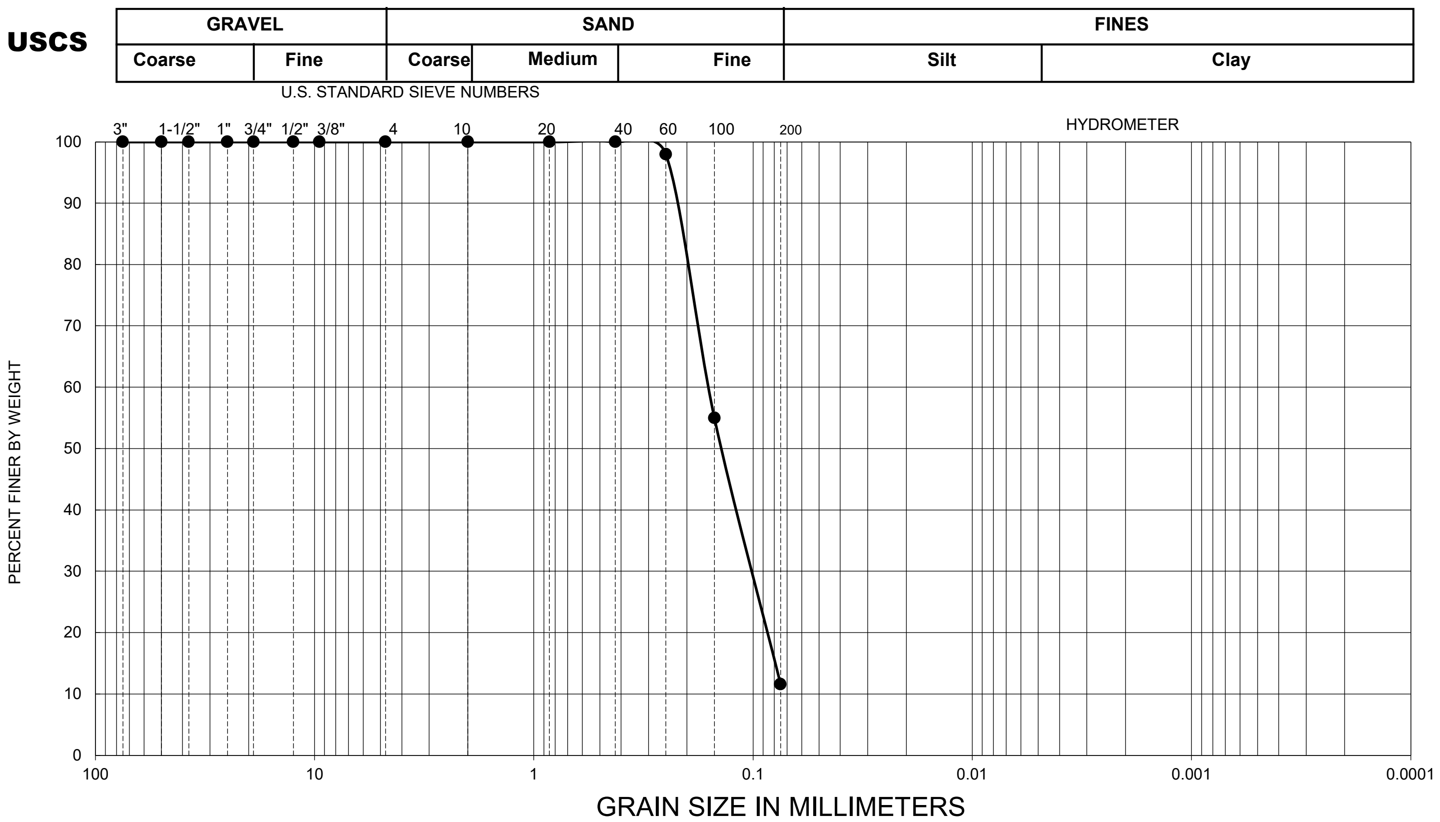
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-18

Date Tested: **3/12/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-002	S19	70-71.5	11.6	SP-SM

Sample Description	Dark gray Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	100
	No. 60	0.25 mm	98
	No 100	0.15 mm	55
	No 200	.075 mm	11.6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

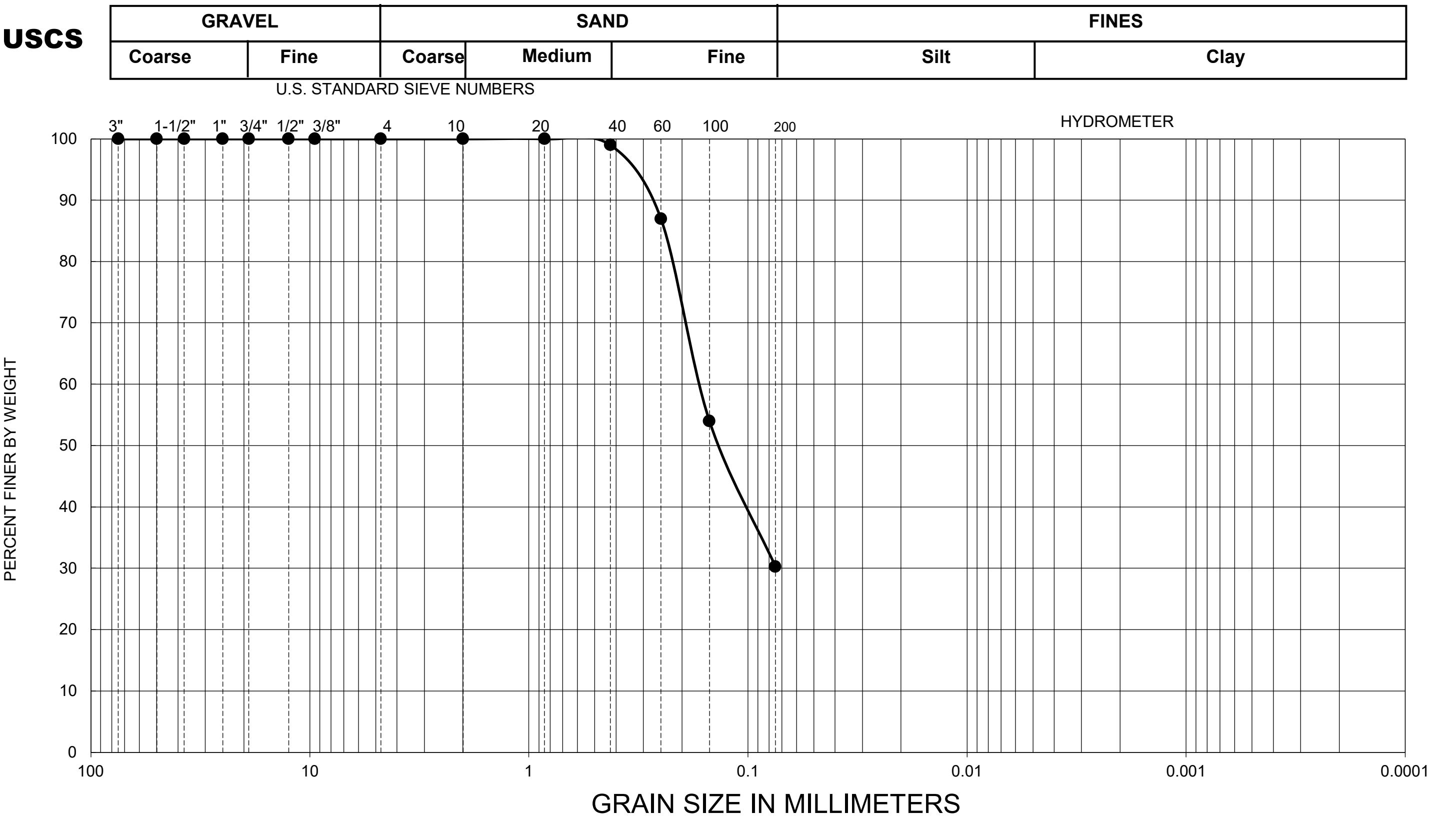
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-19

Date Tested: 3/11/2020



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-002	S25	100-101.5	30.3	SM

Sample Description	Dark gray Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	99
	No. 60	0.25 mm	87
	No 100	0.15 mm	54
	No 200	.075 mm	30.3

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

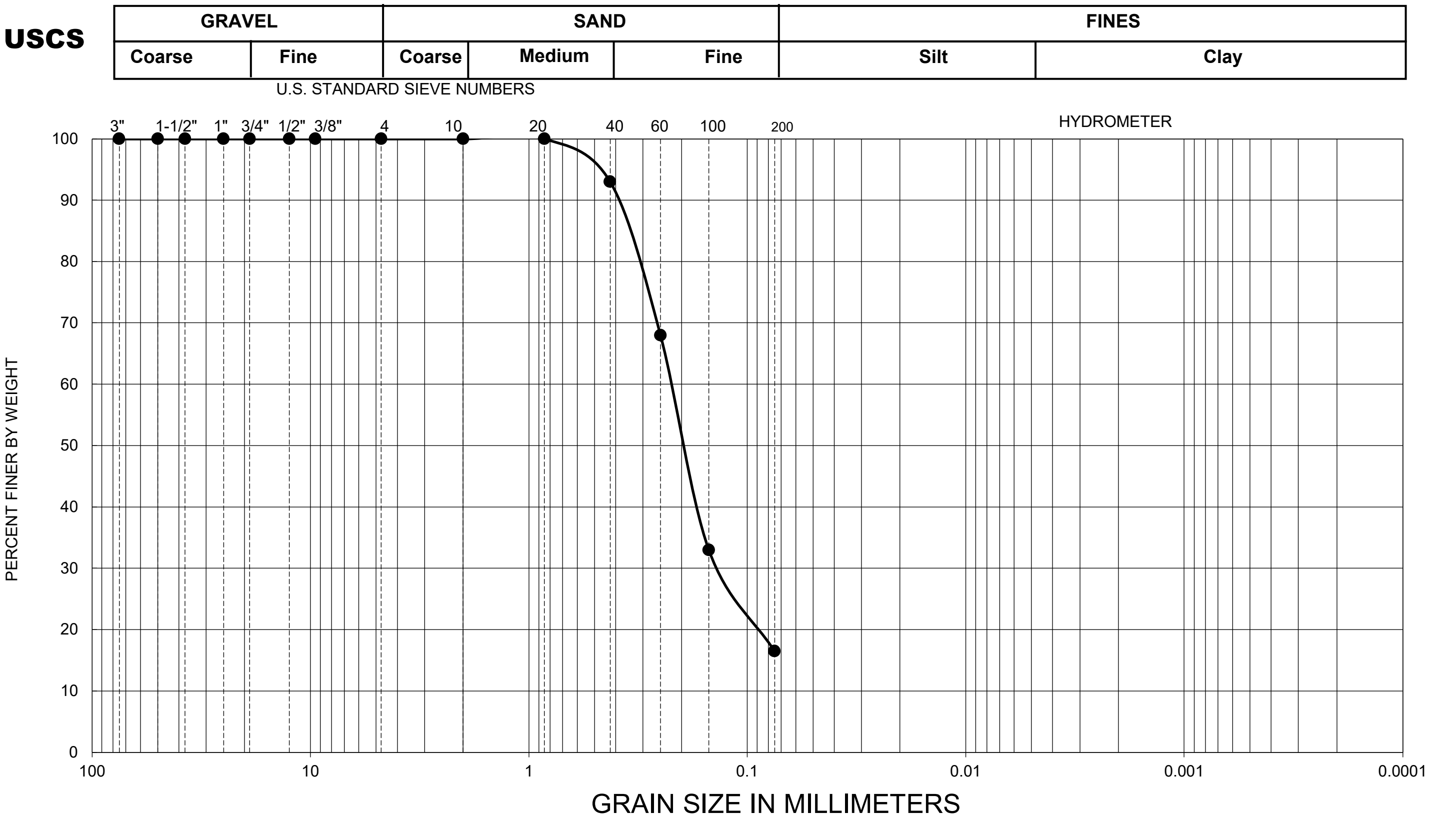
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-20

Date Tested: **3/11/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-002	S27	110-111.5	16.5	SM

Sample Description	Dark gray Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	93
	No. 60	0.25 mm	68
	No 100	0.15 mm	33
	No 200	.075 mm	16.5

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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GRADATION TEST RESULTS

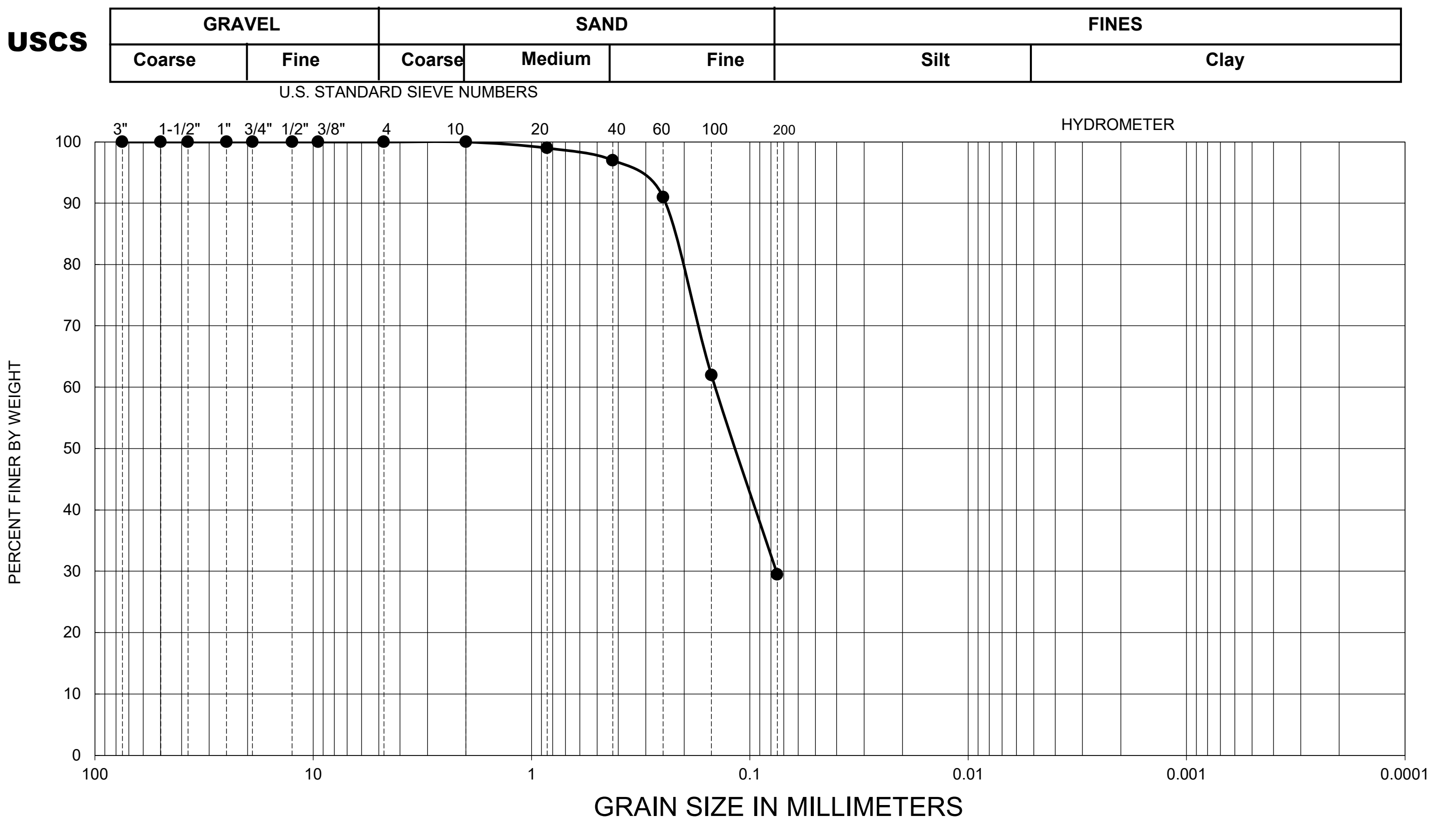
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-21

Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

Date Tested: **3/19/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-002	30	125-126.5	29.5	SM

Sample Description	Dark gray Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	99
	No. 40	0.425 mm	97
	No. 60	0.25 mm	91
	No 100	0.15 mm	62
	No 200	.075 mm	29.5

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Project No.	20180876.001A	Date:	2-Apr-20

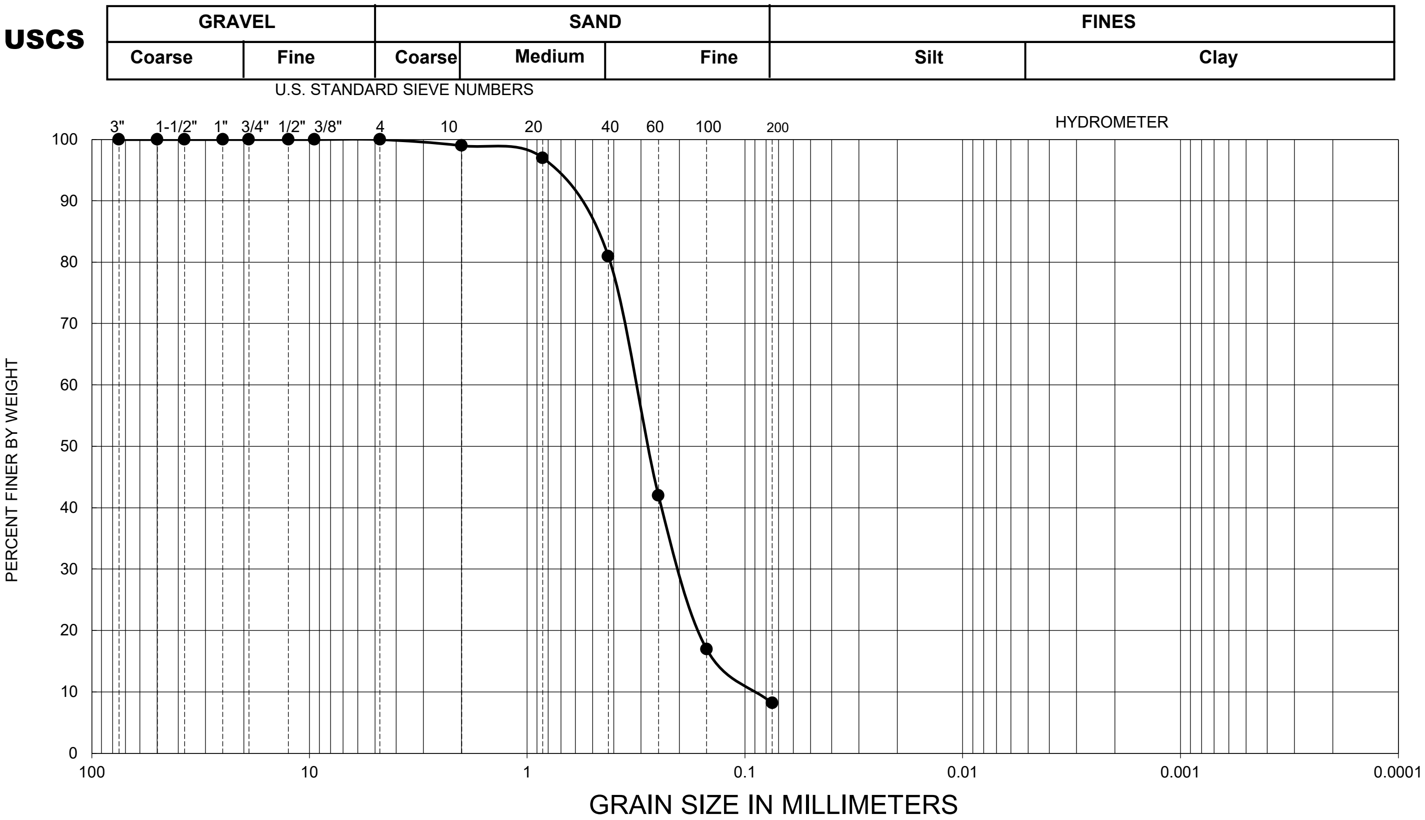
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-22

Date Tested: **3/19/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-002	35	150-151.5	8.2	SP-SM

Sample Description	Dark gray Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	99
	No. 20	0.85 mm	97
	No. 40	0.425 mm	81
	No. 60	0.25 mm	42
	No 100	0.15 mm	17
	No 200	.075 mm	8.2

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

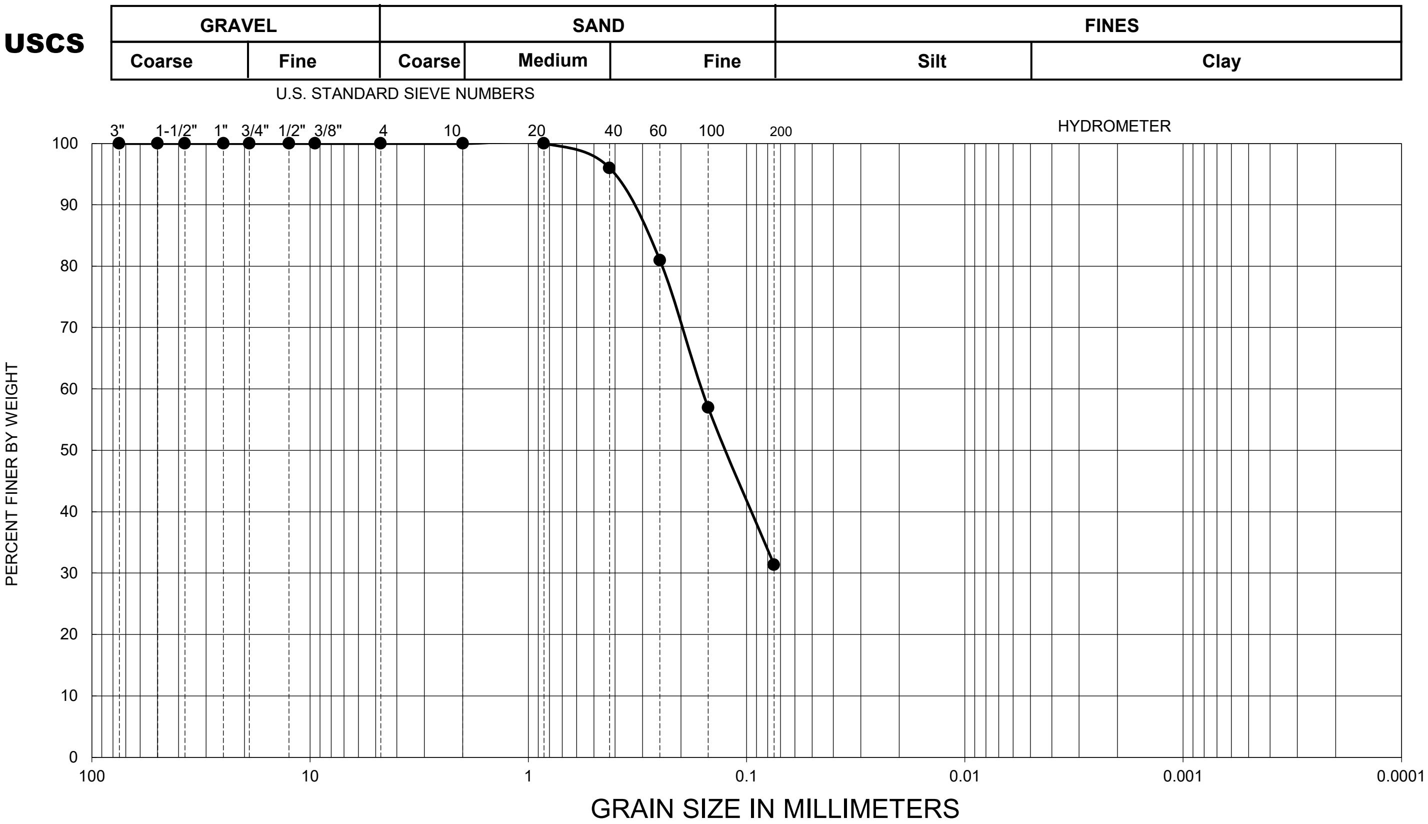
GRADATION TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-23

Date Tested: **3/12/2020**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
R-20-002	37	170-171.5	31.4	SM

Sample Description	Dark gray Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	96
	No. 60	0.25 mm	81
	No 100	0.15 mm	57
	No 200	.075 mm	31.4

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	J.B.	Tech:	MSL
Project No.	20180876.001A	Date:	2-Apr-20

GRADATION TEST RESULTS

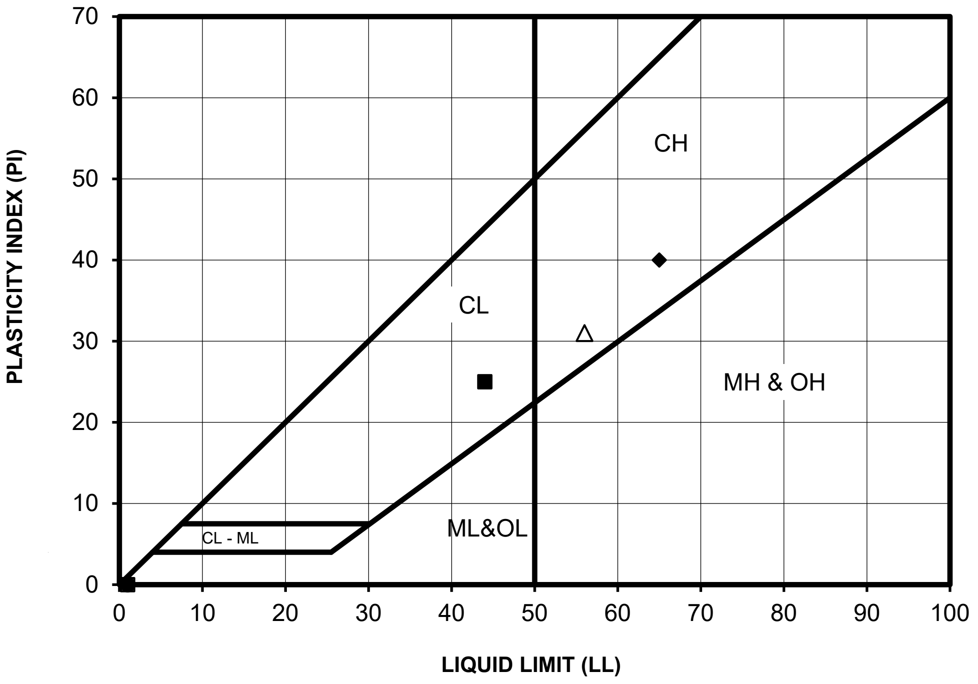
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-24

Date Tested: 3/12/2020 to
3/24/2020

SYMBOL	SAMPLE NAME	DEPTH (ft)	LL	PL	PI	USCS CLASSIFICATION (Minus No. 40 Sieve Fraction)	USCS (Entire Sample)
•	R-20-001/S5	12-13.5	NP	NP	NP	ML	SP-SM
■	R-20-001/S11	30-31.5	44	19	25	CL	CL
◆	R-20-001/S12	35-36.5	65	25	40	CH	CH
○	R-20-001/S15	60-61.5	NP	NP	NP	ML	SM
□	R-20-001/S18	75-76.5	NP	NP	NP	ML	SP-SM
△	R-20-001/S31	150-151	56	25	31	CH	CH
+	R-20-002/S3	7-8.5	NP	NP	NP	ML	SM
◇	R-20-002/S9	21-22.5	NP	NP	NP	ML	SP-SM



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ATTERBERG LIMITS TEST RESULTS

FIGURE

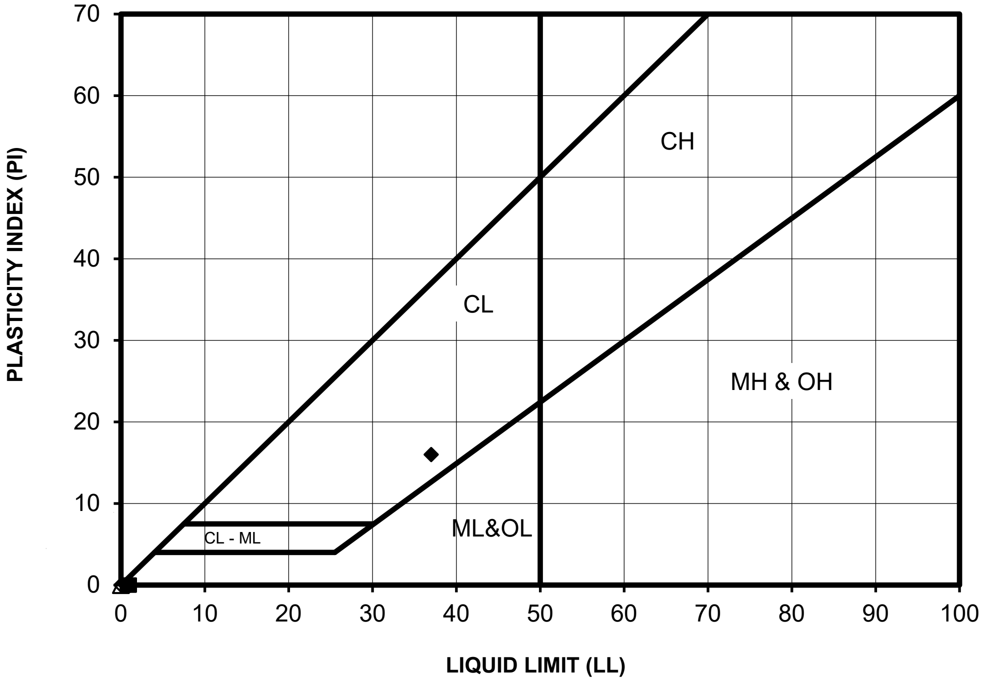
C-25

**Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California**

Checked by J.B.
TECH UP/TC/RH
PROJECT NO: 20180876.001A
2-Apr-20

Date Tested: 3/16/2020 to 3/18/2020

SYMBOL	SAMPLE NAME	DEPTH (ft)	LL	PL	PI	USCS CLASSIFICATION (Minus No. 40 Sieve Fraction)	USCS (Entire Sample)
●	R-20-002/S15	50-51.5	NP	NP	NP	ML	SP
■	R-20-002/S21	80-81.5	NP	NP	NP	ML	SM
◆	R-20-002/S23	90-91.5	37	21	16	CL	CL
○	R-20-002/S27	110-111.5	NP	NP	NP	ML	SM
□	R-20-002/S33	140-141.5	NP	NP	NP	ML	SM



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Limitations: Pursuant to applicable codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specification were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided. This report may not be reproduced, except in full, without written approval of Kleinfelder.

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ATTERBERG LIMITS TEST RESULTS

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

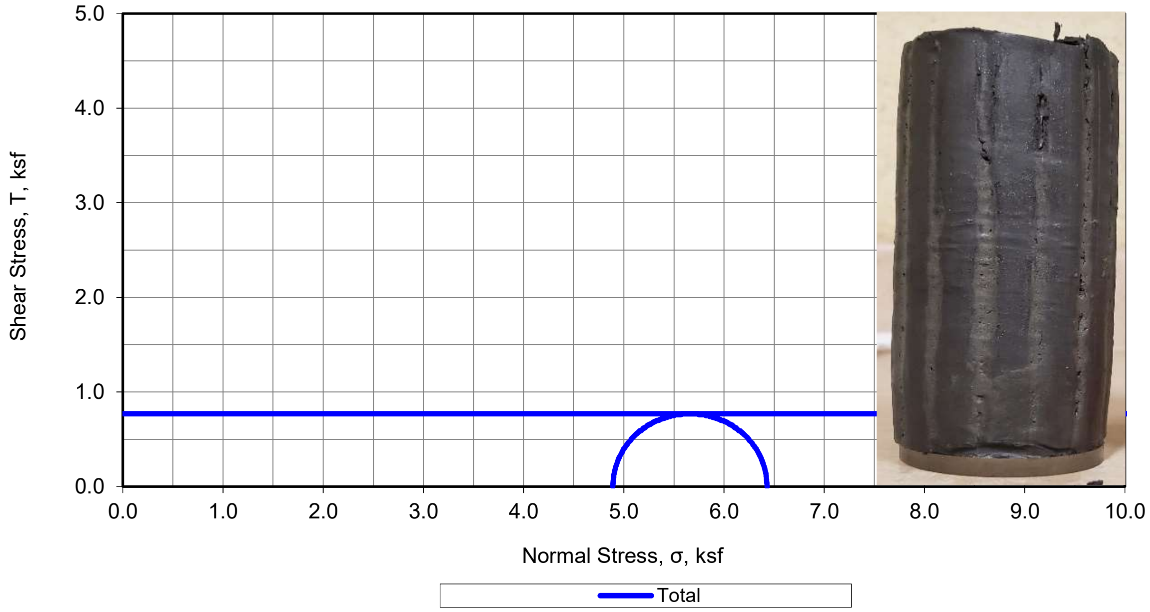
FIGURE

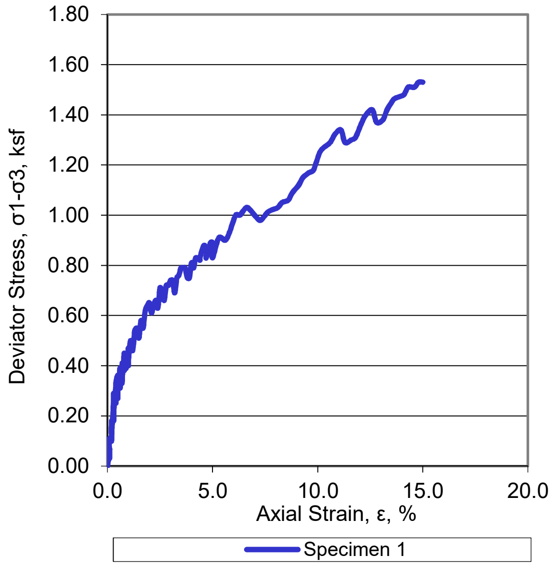
C-26

Checked by J.B
TECH UP/TC/RH
PROJECT NO: 20180876.001A
2-Apr-20

Total	
c = 0.77	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.49
	Water Content, %	ω_O	46.5
	Dry Density, lbs/ft^3	$^{\gamma}d_o$	76.6
	Saturation, %	S_O	106
	Void Ratio	e_O	1.159
Minor Principal Stress, ksf		σ_3	4.90
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	1.53
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.85
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	1.53
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	14.85

Description of Specimen: Black Fat Clay (CH)

Amount of Material Finer than the No. 200, %: 88

LL: 56	PL: 24	PI: 32	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

Boring:	R-20-001
Sample:	S21
Depth, ft:	91.0
Test Date:	3/16/20

Remarks: nm= not measured, na = not applicable

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2601 Barrington Ct, Hayward, CA 94545

Project No.:	20180876.001A
Date:	3/20/20
Entry By:	CP
Checked By:	CP
File Name:	HL12966

TRIAXIAL COMPRESSION TEST (UU)

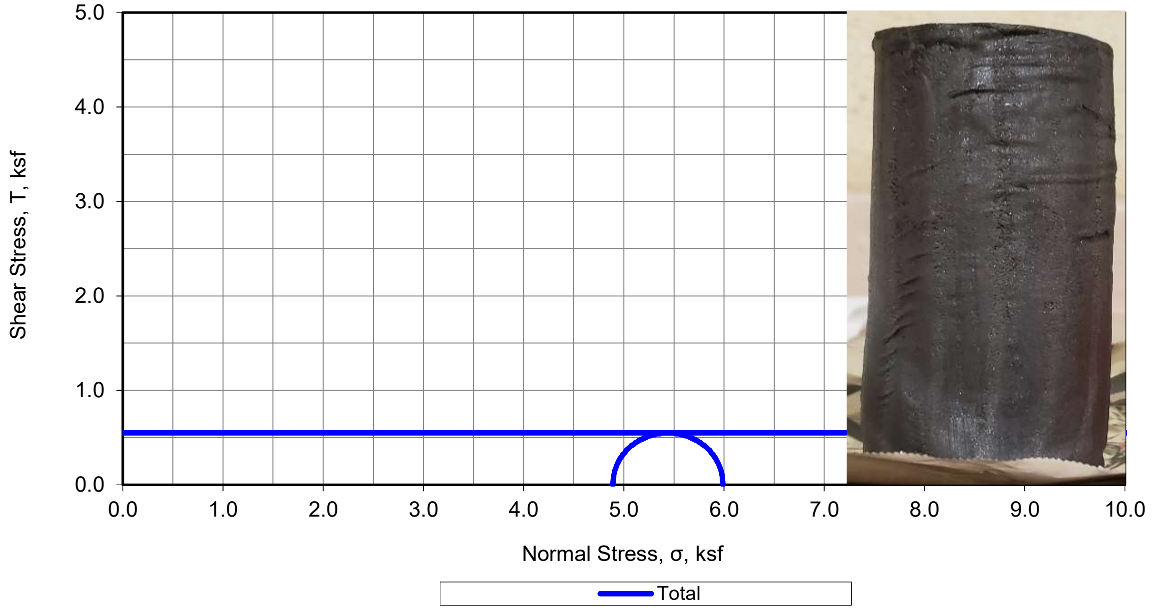
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

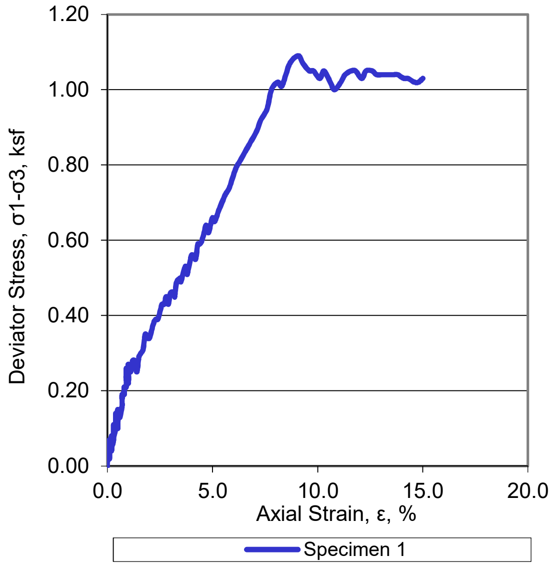
Figure

C-27

Total	
c = 0.55	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.39
	Height, in	H_O	5.38
	Water Content, %	ω_O	48.2
	Dry Density, lbs/ft^3	γ_{d_o}	73.8
	Saturation, %	S_O	103
	Void Ratio	e_O	1.240
Minor Principal Stress, ksf		σ_3	4.90
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	1.09
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	9.08
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	1.03
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	9.08

Description of Specimen: Black Silt (ML)

Amount of Material Finer than the No. 200, %: nm

LL: nm | PL: nm | PI: nm | G_S: 2.65 Assumed | Specimen Type: Undisturbed | Test Method: ASTM D2850

Membrane correction applied

Boring:	R-20-002
Sample:	S22
Depth, ft:	85.5
Test Date:	3/16/20

Remarks: nm= not measured, na = not applicable

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2601 Barrington Ct, Hayward, CA 94545

Project No.:	20180876.001A
Date:	3/20/20
Entry By:	CP
Checked By:	CP
File Name:	HL12966

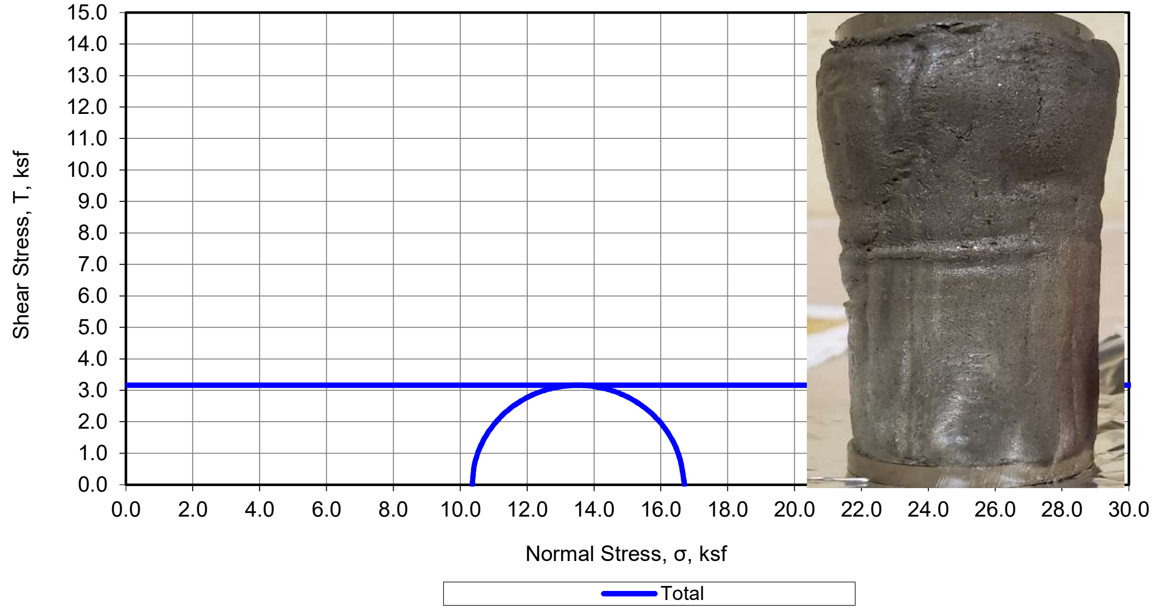
TRIAXIAL COMPRESSION TEST (UU)

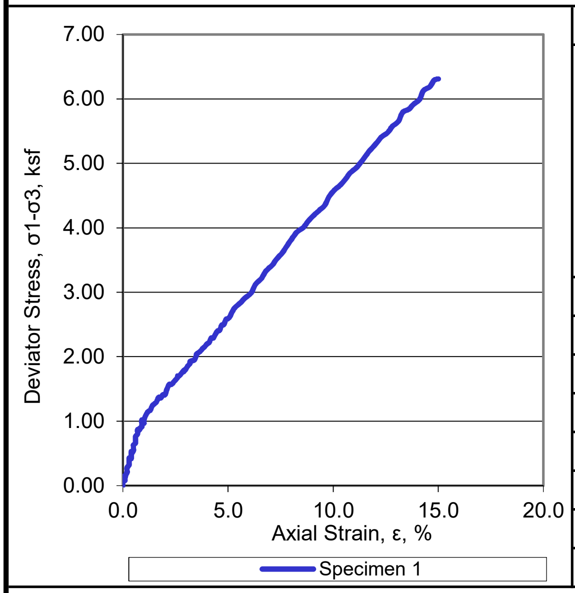
Camino Del Mar Bridge Replacement Over San Dieguito River - Phase 0
Del Mar, California

Figure
C-28

Total	
c = 3.16	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.38
	Height, in	H_O	4.79
	Water Content, %	ω_O	24.1
	Dry Density, lbs/ft^3	γ_{d_o}	104.6
	Saturation, %	S_O	110
	Void Ratio	e_O	0.581
Minor Principal Stress, ksf		σ_3	10.37
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	6.31
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	15.02
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	6.31
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	15.02

Description of Specimen: Black Slity Clay with Sand (CL-ML)

Amount of Material Finer than the No. 200, %: nm

LL: 26 | PL: 19 | PI: 7 | G_S: 2.65 Assumed | Specimen Type: Undisturbed | Test Method: ASTM D2850

Membrane correction applied

Boring:	R-20-002
Sample:	S38
Depth, ft:	180.5
Test Date:	3/16/20

Remarks: nm= not measured, na = not applicable

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Project No.:	20180876.001A
Date:	3/20/20
Entry By:	CP
Checked By:	CP
File Name:	HL12966

TRIAXIAL COMPRESSION TEST (UU)

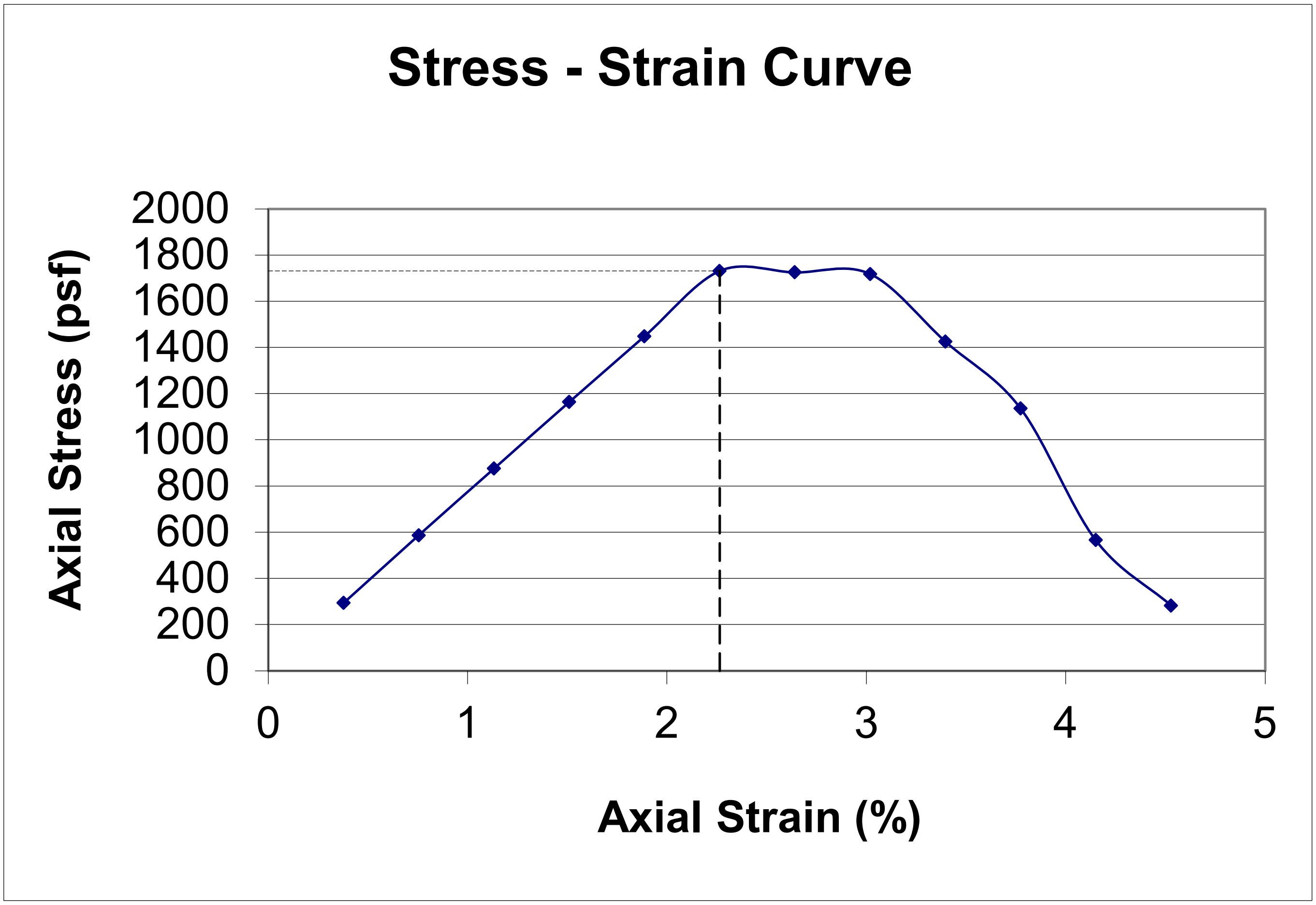
Camino Del Mar Bridge Replacement Over San Dieguito River - Phase 0
Del Mar, California

Figure

C-29

Sample Information			
Boring No.	R-20-002	Sample No.	24
Depth	95-96 ft		
Description	Dark gray silty sand		

Unit Weight	Diameter	2.42	in
	Length	5.3	in
	Wet Wt.	774.6	g
Moisture Content	Wet Wt.	410.9	
	Dry Wt.	320.8	
	Moisture	28.1%	
Wet Unit Weight	(pcf)	121.0	
Dry Unit Weight	(pcf)	94.5	



Unconfined Compressive Strength (psf) = **1731**

Unconfined Shear Strength (psf) = **865**

Loading Rate : 1%/min

Date Tested 3/25/2020

Performed in General Accordance with ASTM D2166

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CHECKED BY :	J.B.	TECH:	Uly P.
PROJECT NO:	20180876.001A	DATE:	2-Apr-20

UNCONFINED COMPRESSION TEST

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

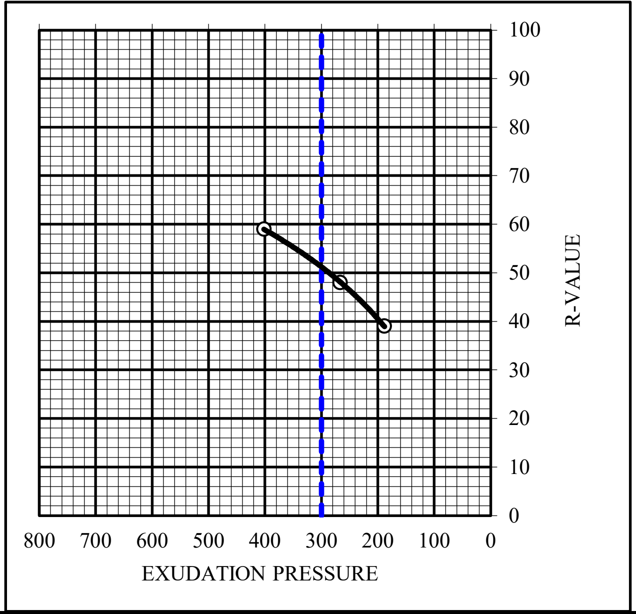
C-30

Boring No.	Sample No.	Depth	Description	Date Tested
R-20-001	S-1	0.5'-5'	Brown sand with silt	3/19/2020

TEST SPECIMEN				
MOLD NO.	6	2	9	
FOOT PRESSURE, psi	**280**	**210**	**150**	
INITIAL MOISTURE, %	4.0	4.0	4.0	
"AS-IS" WEIGHT, g	1200	1200	1200	
DRY WEIGHT, g	1154.4	1154.4	1154.4	
WATER ADDED, ml	**120**	**130**	**140**	
COMPACTION MOISTURE, %	14.3	15.2	16.1	
HEIGHT OF BRIQUETTE, in.	**2.5**	**2.49**	**2.48**	
WEIGHT BRIQUETTE/MOLD,	**3088**	**3089.3**	**3088.8**	
WEIGHT OF MOLD, g	**2101.2**	**2107.9**	**2114.6**	
WEIGHT OF BRIQUETTE, g	986.8	981.4	974.2	
DRY DENSITY, pcf	104.7	103.8	102.6	
STABILOMETER, 1000 lbs	**19**	**25**	**38**	
2000lbs	**40**	**55**	**68**	
DISPLACEMENT, in	**5.22**	**5.26**	**5.35**	
EXUDATION LOAD, lbs	**5048**	**3346**	**2368**	
EXUDATION PRESSURE, psi	401.9	266.4	188.5	
R-VALUE	59	48	39	
CORRECTED R-VALUE	59	48	39	
DIAL READING, END	0.0426	0.0275	0.0275	
DIAL READING, START	0.0433	0.0280	0.0286	
DIFFERENCE	-0.0007	-0.0005	-0.0011	
EXPANSION PRESSURE, PS	0.0	0.0	0.0	

INITIAL MOISTURE	
WET WEIGHT, g	323.4
DRY WEIGHT, g	311.1
WEIGHT OF WATER	
WEIGHT OF SAMPLE	
MOISTURE CONTENT %	4.0

R-VALUE:	**51**
Location:	



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R-Value (ASTM D2844)

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-31

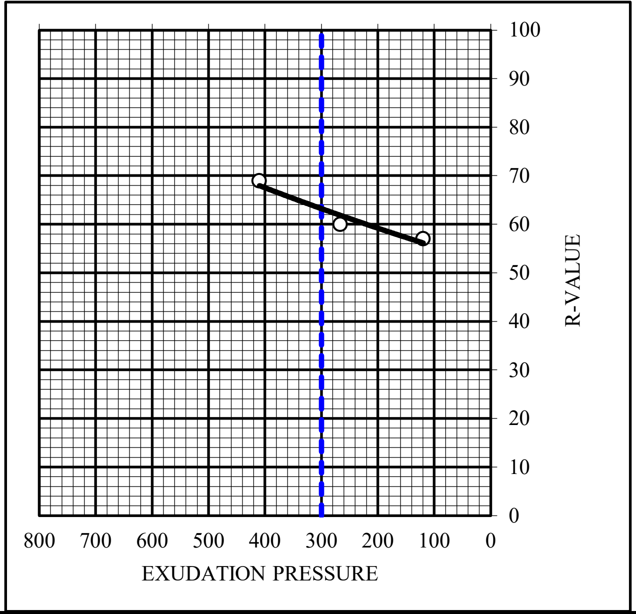
Checked By: J.B.	TECH: Uly P.
Job Number: 20180876.001A	DATE: 2-Apr-20

Boring No.	Sample No.	Depth	Description	Date Tested
R-20-002	S-1	0.5'-4'	Brown sand with silt	3/19/2020

TEST SPECIMEN				
MOLD NO.	10	5	8	
FOOT PRESSURE, psi	**250**	**210**	**150**	
INITIAL MOISTURE, %	4.3	4.3	4.3	
"AS-IS" WEIGHT, g	1200	1200	1200	
DRY WEIGHT, g	1150.3	1150.3	1150.3	
WATER ADDED, ml	**100**	**130**	**140**	
COMPACTION MOISTURE, %	13.0	15.6	16.5	
HEIGHT OF BRIQUETTE, in.	**2.55**	**2.56**	**2.56**	
WEIGHT BRIQUETTE/MOLD,	**3106.1**	**3112.7**	**3106.4**	
WEIGHT OF MOLD, g	**2109.2**	**2107.9**	**2112.7**	
WEIGHT OF BRIQUETTE, g	996.9	1004.8	993.7	
DRY DENSITY, pcf	104.9	103.0	101.1	
STABILOMETER, 1000 lbs	**14**	**19**	**19**	
2000lbs	**29**	**38**	**43**	
DISPLACEMENT, in	**5.03**	**5.44**	**5.07**	
EXUDATION LOAD, lbs	**5151**	**3346**	**1507**	
EXUDATION PRESSURE, psi	410.1	266.4	120.0	
R-VALUE	69	60	57	
CORRECTED R-VALUE	69	60	57	
DIAL READING, END	0.0295	0.0122	0.0300	
DIAL READING, START	0.0314	0.0126	0.0309	
DIFFERENCE	-0.0019	-0.0004	-0.0009	
EXPANSION PRESSURE, PS	0.0	0.0	0.0	

INITIAL MOISTURE	
WET WEIGHT, g	564.9
DRY WEIGHT, g	541.5
WEIGHT OF WATER	
WEIGHT OF SAMPLE	
MOISTURE CONTENT %	4.3

R-VALUE:	**63**
Location:	



KLEINFELDER
Bright People. Right Solutions.

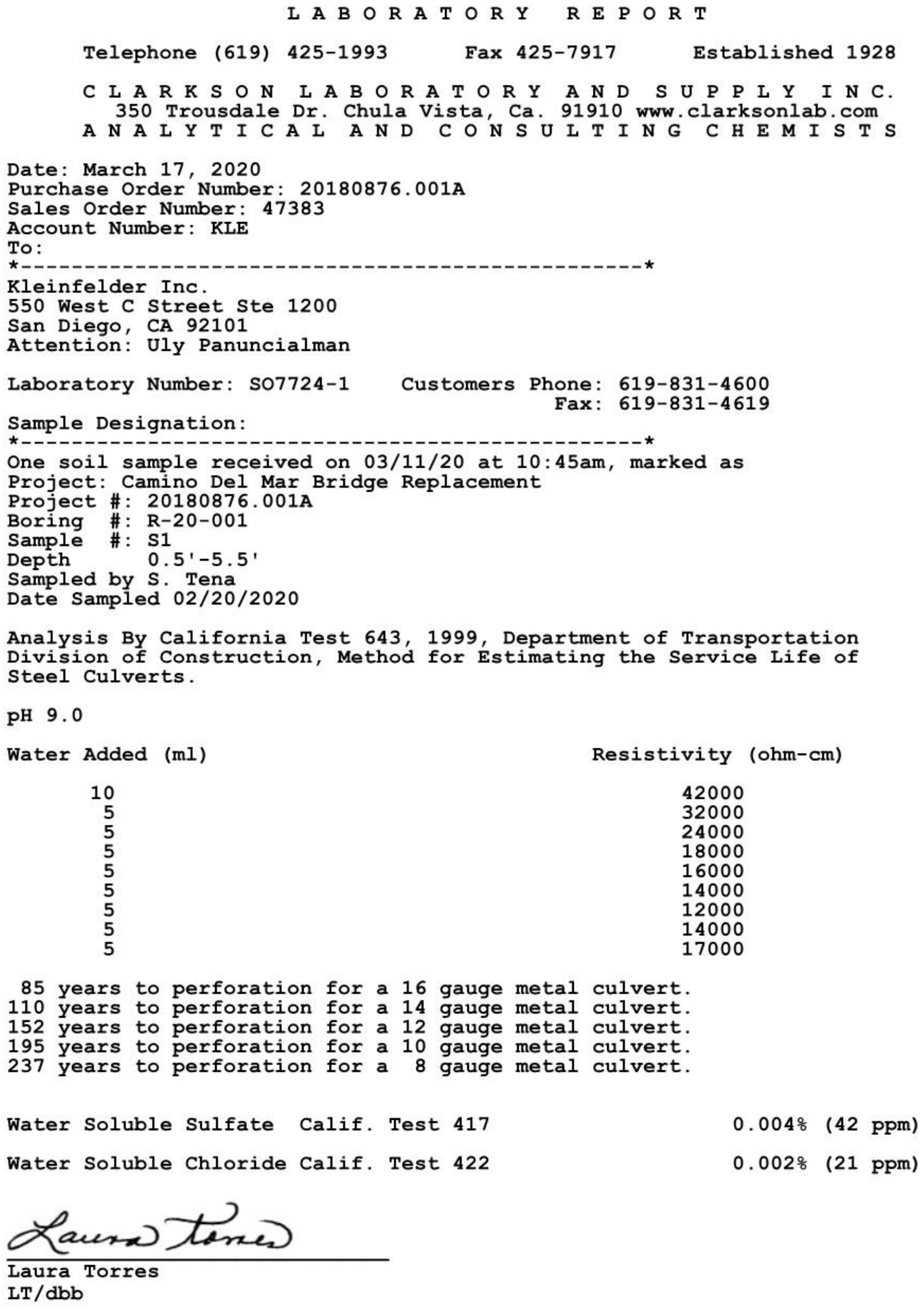
Checked By: J.B.	TECH: Uly P.
Job Number: 20180876.001A	DATE: 2-Apr-20

R-Value (ASTM D2844)

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-32



LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: March 17, 2020
Purchase Order Number: 20180876.001A
Sales Order Number: 47383
Account Number: KLE
To:

Kleinfelder Inc.
550 West C Street Ste 1200
San Diego, CA 92101
Attention: Uly Panuncialman

Laboratory Number: SO7724-1 Customers Phone: 619-831-4600
Fax: 619-831-4619
Sample Designation:

One soil sample received on 03/11/20 at 10:45am, marked as
Project: Camino Del Mar Bridge Replacement
Project #: 20180876.001A
Boring #: R-20-001
Sample #: S1
Depth 0.5'-5.5'
Sampled by S. Tena
Date Sampled 02/20/2020

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 9.0

Water Added (ml)	Resistivity (ohm-cm)
10	42000
5	32000
5	24000
5	18000
5	16000
5	14000
5	12000
5	14000
5	17000

85 years to perforation for a 16 gauge metal culvert.
110 years to perforation for a 14 gauge metal culvert.
152 years to perforation for a 12 gauge metal culvert.
195 years to perforation for a 10 gauge metal culvert.
237 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.004% (42 ppm)
Water Soluble Chloride	Calif. Test 422	0.002% (21 ppm)

Laura Torres

Laura Torres
LT/dbb

KLEINFELDER
Bright People. Right Solutions.

CHECKED BY: J.B. TECH: Clarkson Lab
JOB NUMBER: 20180876.001A DATE: 2-Apr-20

Corrosion Testing

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE
C-33

LABORATORY REPORT

Telephone (619) 425-1993   Fax 425-7917   Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: March 17, 2020
Purchase Order Number: 20180876.001A
Sales Order Number: 47383
Account Number: KLE
To:

Kleinfelder Inc.
550 West C Street Ste 1200
San Diego, CA 92101
Attention: Uly Panuncialman

Laboratory Number: SO7724-2   Customers Phone: 619-831-4600
Fax: 619-831-4619

Sample Designation:

One soil sample received on 03/11/20 at 10:45am,
marked as
Project: Camino Del Mar Bridge Replacement
Project #: 20180876.001A
Boring #: R-20-002
Sample #: S1
Depth 0.5'-4'
Sampled by S. Tena
Date Sampled 02/20/2020

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.7

Water Added (ml)	Resistivity (ohm-cm)
10	31000
5	23000
5	18000
5	13000
5	19000
5	23000

87 years to perforation for a 16 gauge metal culvert.
114 years to perforation for a 14 gauge metal culvert.
157 years to perforation for a 12 gauge metal culvert.
201 years to perforation for a 10 gauge metal culvert.
245 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.005% (45 ppm)

Water Soluble Chloride Calif. Test 422   0.002% (21 ppm)

Laura Torres

Laura Torres
LT/dbb

KLEINFELDER
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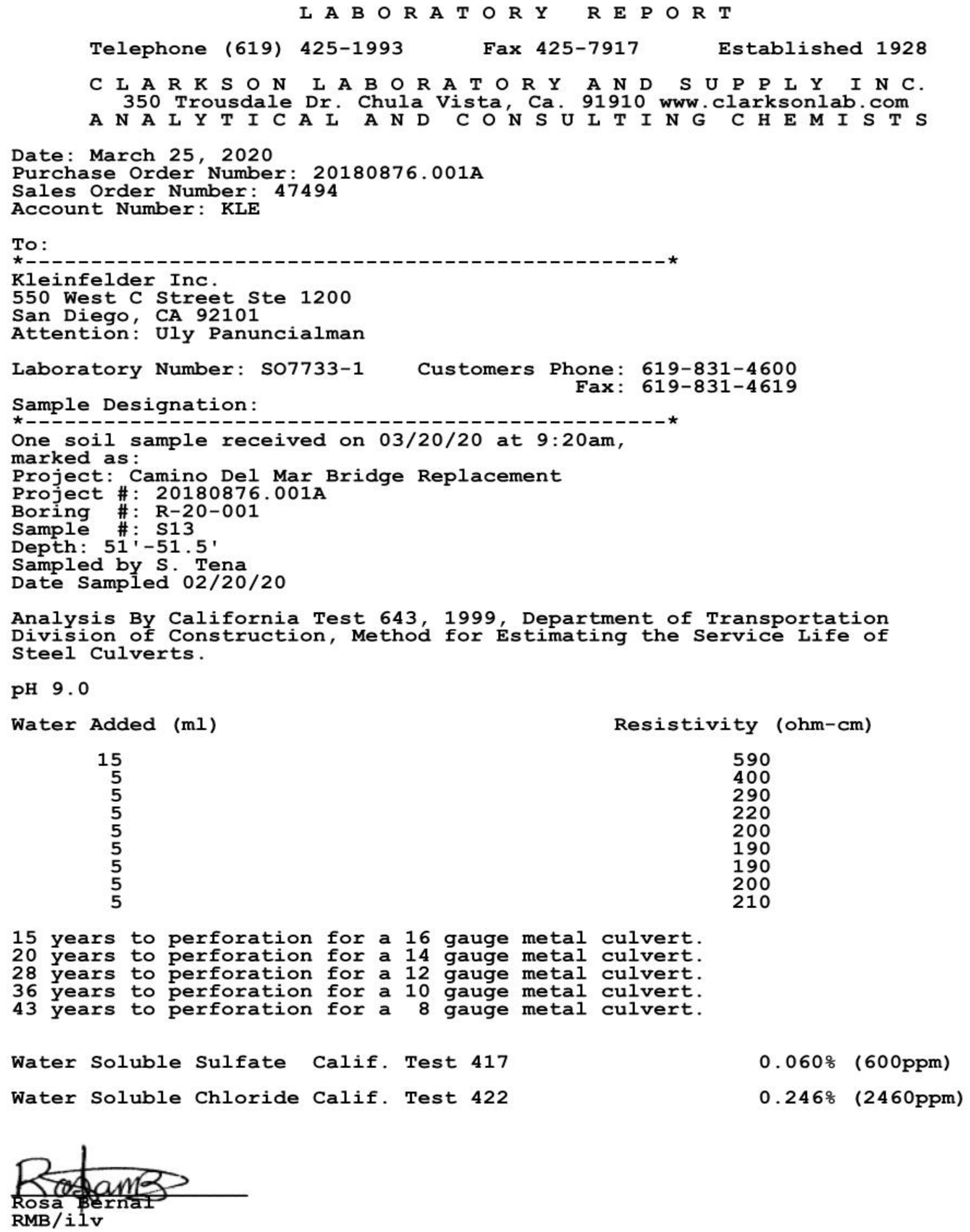
CHECKED BY: J.B.   TECH: Clarkson Lab
JOB NUMBER: 20180876.001A   DATE: 2-Apr-20

Corrosion Testing

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE
C-34



LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: March 25, 2020
Purchase Order Number: 20180876.001A
Sales Order Number: 47494
Account Number: KLE

To:

Kleinfelder Inc.
550 West C Street Ste 1200
San Diego, CA 92101
Attention: Uly Panuncialman

Laboratory Number: SO7733-1 Customers Phone: 619-831-4600
Fax: 619-831-4619

Sample Designation:

One soil sample received on 03/20/20 at 9:20am,
marked as:
Project: Camino Del Mar Bridge Replacement
Project #: 20180876.001A
Boring #: R-20-001
Sample #: S13
Depth: 51'-51.5'
Sampled by S. Tena
Date Sampled 02/20/20

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 9.0

Water Added (ml)	Resistivity (ohm-cm)
15	590
5	400
5	290
5	220
5	200
5	190
5	190
5	200
5	210

15 years to perforation for a 16 gauge metal culvert.
20 years to perforation for a 14 gauge metal culvert.
28 years to perforation for a 12 gauge metal culvert.
36 years to perforation for a 10 gauge metal culvert.
43 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.060% (600ppm)
Water Soluble Chloride	Calif. Test 422	0.246% (2460ppm)

Rosa Bernal
RMB/ilv

Corrosion Testing

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-35

CHECKED BY: J.B. TECH: Clarkson Lab
JOB NUMBER: 20180876.001A DATE: 13-May-20

LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: March 25, 2020
Purchase Order Number: 20180876.001A
Sales Order Number: 47494
Account Number: KLE

To:

Kleinfelder Inc.
550 West C Street Ste 1200
San Diego, CA 92101
Attention: Uly Panuncialman

Laboratory Number: SO7733-2 Customers Phone: 619-831-4600
Fax: 619-831-4619

Sample Designation:

One soil sample received on 03/20/20 at 9:20am,
marked as:
Project: Camino Del Mar Bridge Replacement
Project #: 20180876.001A
Boring #: R-20-002
Sample #: S30
Depth: 126'-126.5'
Sampled by S. Tena
Date Sampled 02/20/20

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.0

Water Added (ml)	Resistivity (ohm-cm)
20	220
5	200
5	150
5	110
5	100
5	93
5	85
5	120
5	140

11 years to perforation for a 16 gauge metal culvert.
14 years to perforation for a 14 gauge metal culvert.
20 years to perforation for a 12 gauge metal culvert.
26 years to perforation for a 10 gauge metal culvert.
31 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.087% (870ppm)
Water Soluble Chloride	Calif. Test 422	0.748% (7480ppm)

RosaMB
Rosa Bernal
RMB/ilv

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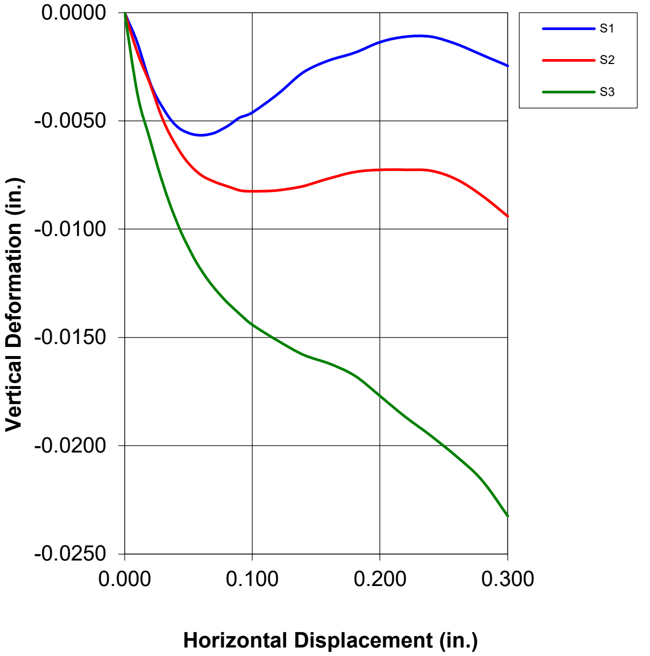
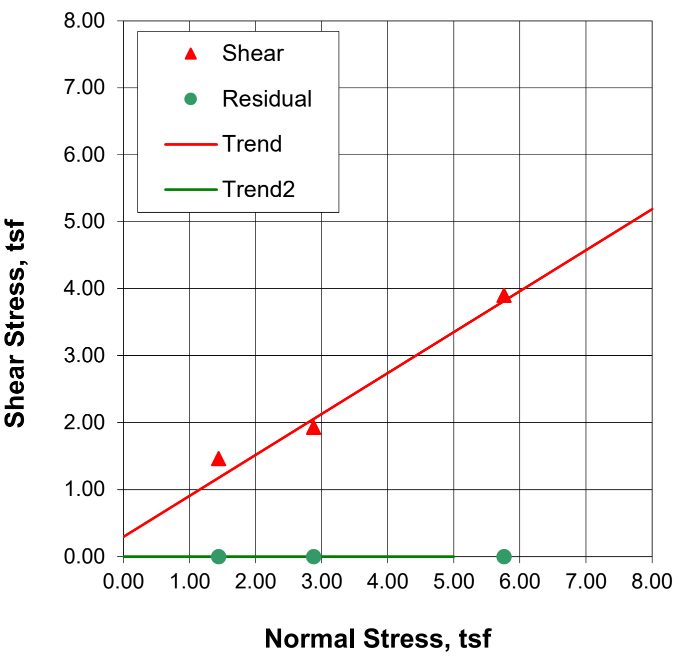
CHECKED BY: J.B. TECH: Clarkson Lab
JOB NUMBER: 20180876.001A DATE: 13-May-20

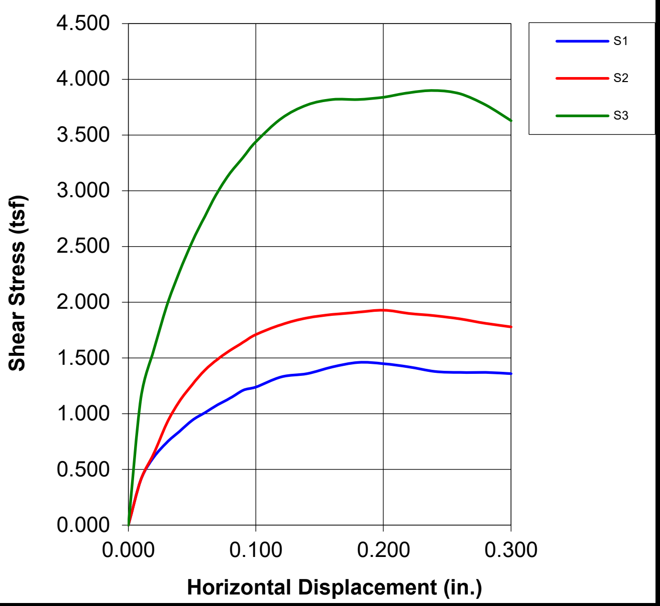
Corrosion Testing

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE
C-36







	Specimen Number	1	2	3	
Initial	Water Content, %	35.0	34.1	33.5	
	Dry Density, pcf	86.0	87.0	86.8	
	Void Ratio	0.960	0.936	0.941	
	Saturation, %	98.5	98.4	96.2	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.96	0.96	0.96	
At Test	Water Content, %	41.6	41.2	41.1	
	Dry Density, pcf	91.2	92.3	97.3	
	Void Ratio	0.851	0.840	0.771	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.91	0.91	0.88	
	Maximum Shear Stress, tsf	1.46	1.93	3.90	
	Residual Shear Stress, tsf	na	na	na	
	Hoizontal Displacment, in.	0.180	0.200	0.240	
	Normal Stress, tsf	1.44	2.88	5.76	
	Strain Rate, in./min.	0.001	0.001	0.001	

LL:	nm	PL:	nm	PI:	nm	G_S:	2.70	Assumed

	c, tsf	ϕ, deg.	Tan ϕ
Failure	0.3	31.4	0.61
Residual	na	na	na

Test Conditions: Undisturbed / Inundated

Specimen 1: Greenish Black Silt

Specimen 2: Greenish Black Silt

Specimen 3: Greenish Black Silt

Boring:	R-20-001
Sample:	S23
Depth, ft:	101.0-101.5
Test Date:	3/18/2020

Remarks: nm = not measured, na = not applicable

The determination of strength envelopes and the development of relationships to aid in interpreting and evaluating test results are beyond the scope of this test method. The user of this report retains the sole responsibility to evaluate and approve any interpreted values from the testing.

KLEINFELDER
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9969 Horn Rd., Sacramento, CA 95827

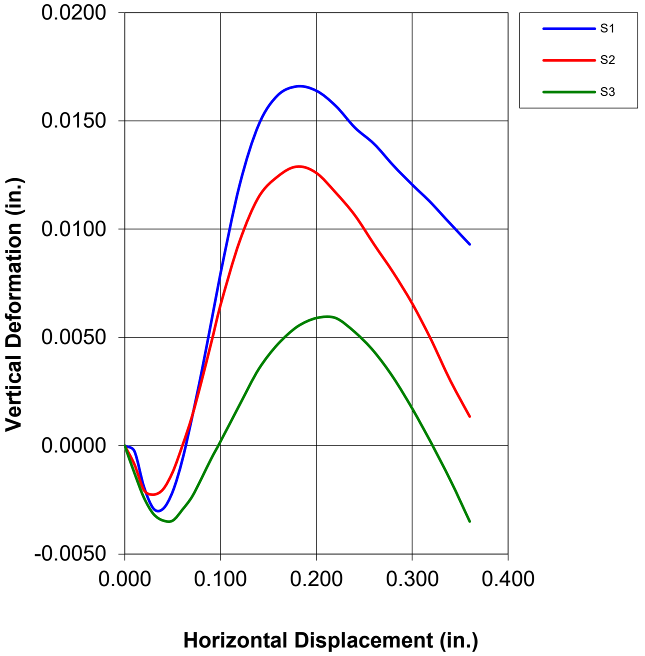
PROJECT NO.:	20180876
ENTRY BY:	A. Wohletz
CHECKED BY:	S. Rader
DATE:	3/23/2020

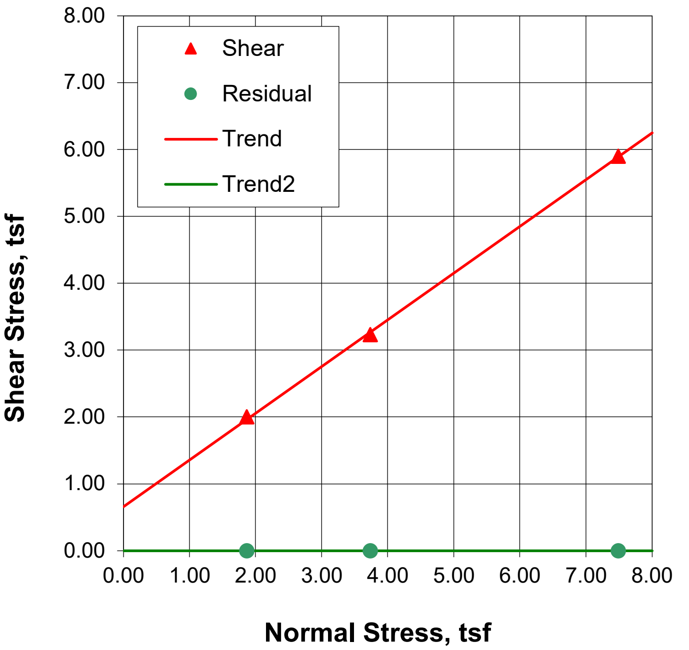
DIRECT SHEAR TEST ASTM D3080

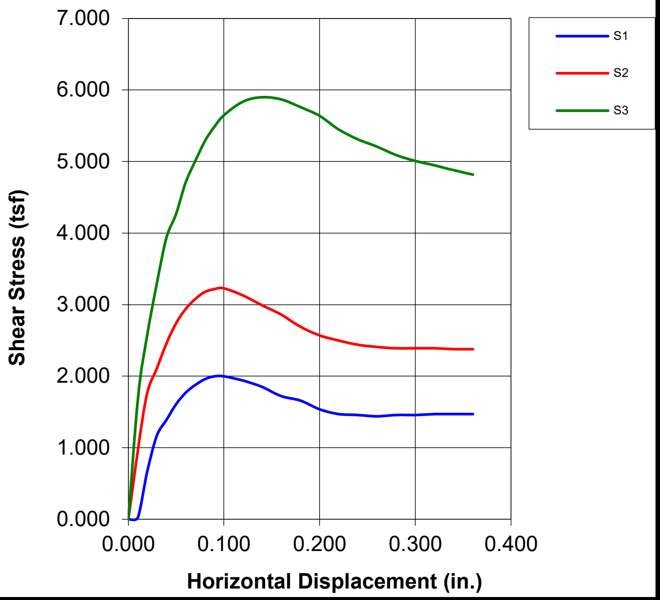
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-37







Specimen Number		1	2	3	
Initial	Water Content, %	20.6	21.5	21.0	
	Dry Density, pcf	103.5	102.7	103.4	
	Void Ratio	0.628	0.640	0.629	
	Saturation, %	88.7	90.9	90.0	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.96	0.96	0.96	
At Test	Water Content, %	22.7	23.9	23.1	
	Dry Density, pcf	105.4	105.3	107.4	
	Void Ratio	0.588	0.588	0.563	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.94	0.93	0.92	
Maximum Shear Stress, tsf		2.00	3.23	5.90	
Residual Shear Stress, tsf		na	na	na	
Hoizontal Displacment, in.		0.090	0.100	0.140	
Normal Stress, tsf		1.87	3.74	7.49	
Strain Rate, in./min.		0.005	0.005	0.005	

LL:	nm	PL:	nm	PI:	nm	G_S:	2.70	Assumed

Test Conditions: Undisturbed / Inundated

Specimen 1: Bluish Gray Poorly Graded Sand

Specimen 2: Bluish Gray Poorly Graded Sand

Specimen 3: Bluish Gray Poorly Graded Sand

	c, tsf	ϕ, deg.	Tan ϕ
Failure	0.7	34.9	0.70
Residual	na	na	na

Boring:	R-20-001
Sample:	S29
Depth, ft:	131.0-131.5
Test Date:	3/20/2020

Remarks: nm = not measured, na = not applicable

The determination of strength envelopes and the development of relationships to aid in interpreting and evaluating test results are beyond the scope of this test method. The user of this report retains the sole responsibility to evaluate and approve any interpreted values from the testing.

KLEINFELDER
Bright People. Right Solutions.
9969 Horn Rd., Sacramento, CA 95827

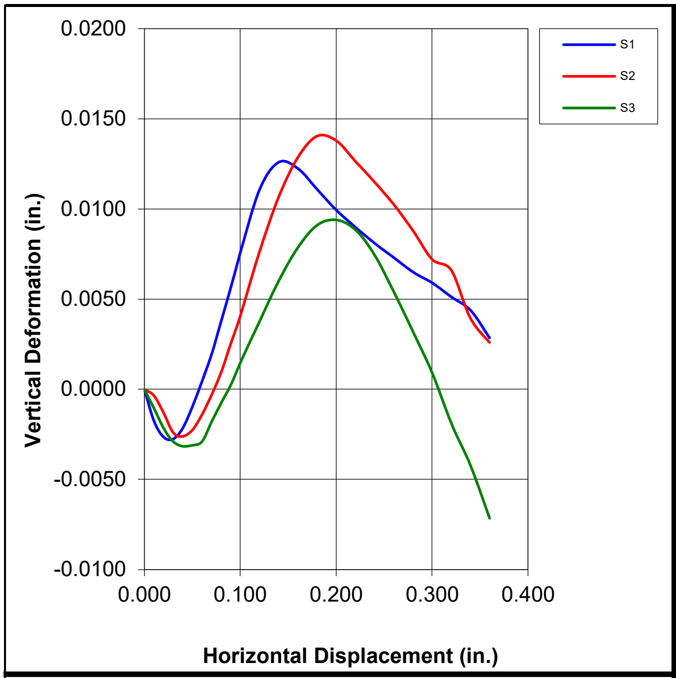
PROJECT NO.:	20180876
ENTRY BY:	A. Wohletz
CHECKED BY:	S. Rader
DATE:	3/23/2020

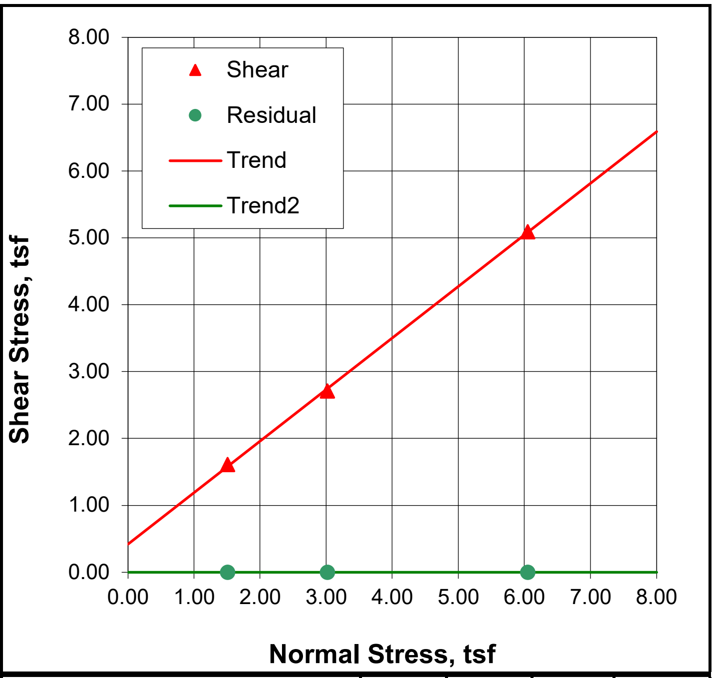
DIRECT SHEAR TEST ASTM D3080

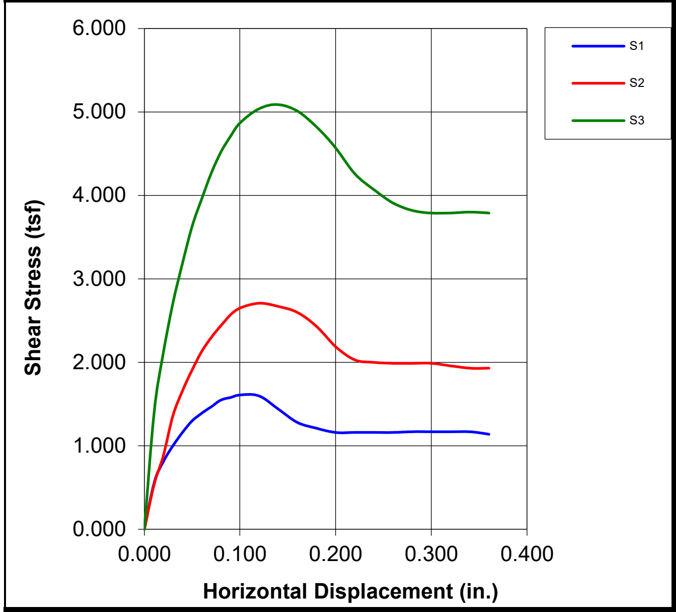
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-38







	Specimen Number	1	2	3	
Initial	Water Content, %	25.6	26.2	25.3	
	Dry Density, pcf	99.6	100.3	101.1	
	Void Ratio	0.660	0.649	0.636	
	Saturation, %	102.8	107.1	105.4	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.96	0.96	0.96	
At Test	Water Content, %	27.9	25.8	27.3	
	Dry Density, pcf	101.4	103.2	106.4	
	Void Ratio	0.617	0.589	0.544	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.94	0.93	0.91	
	Maximum Shear Stress, tsf	1.61	2.71	5.09	
	Residual Shear Stress, tsf	na	na	na	
	Hoizontal Displacment, in.	0.100	0.120	0.140	
	Normal Stress, tsf	1.51	3.02	6.05	
	Strain Rate, in./min.	0.005	0.005	0.005	

LL:	NM	PL:	NM	PI:	NM	G_S:	2.65	Assumed

Test Conditions: Undisturbed / Inundated

	c, tsf	ϕ, deg.	Tan ϕ
Failure	0.4	37.6	0.77
Residual	na	na	na

Specimen 1: Gray Poorly Graded Sand

Specimen 2: Gray Poorly Graded Sand

Specimen 3: Gray Poorly Graded Sand

Boring:	R-20-002
Sample:	S26
Depth, ft:	106.0-106.5
Test Date:	3/24/2020

Remarks: nm = not measured, na = not applicable

The determination of strength envelopes and the development of relationships to aid in interpreting and evaluating test results are beyond the scope of this test method. The user of this report retains the sole responsibility to evaluate and approve any interpreted values from the testing.

KLEINFELDER
Bright People. Right Solutions.
9969 Horn Rd., Sacramento, CA 95827

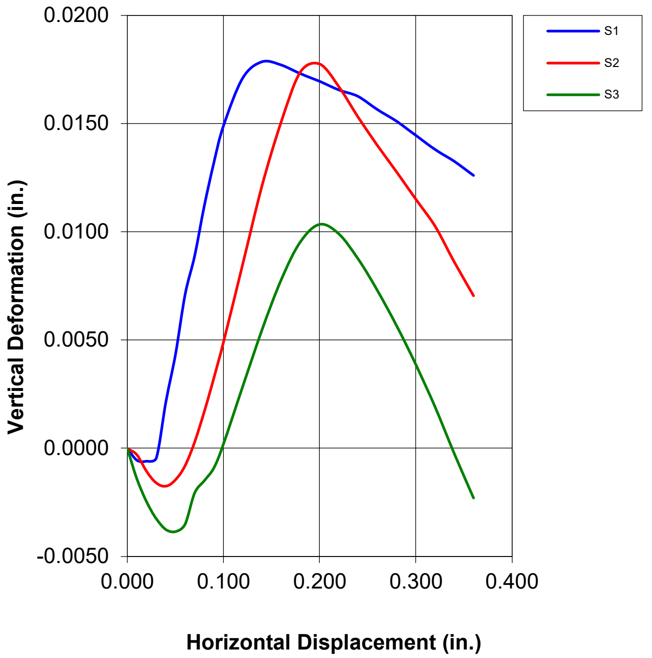
PROJECT NO.:	20180876
ENTRY BY:	A. Wohletz
CHECKED BY:	S. Rader
DATE:	3/25/2020

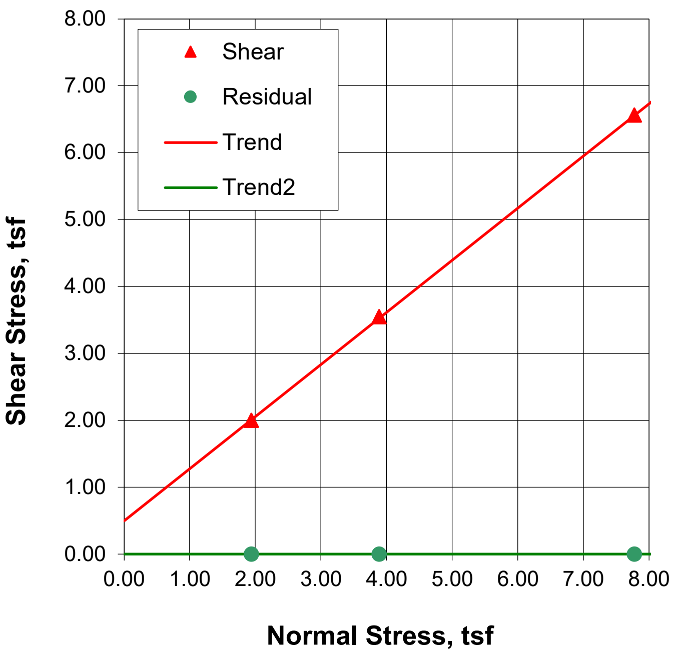
DIRECT SHEAR TEST ASTM D3080

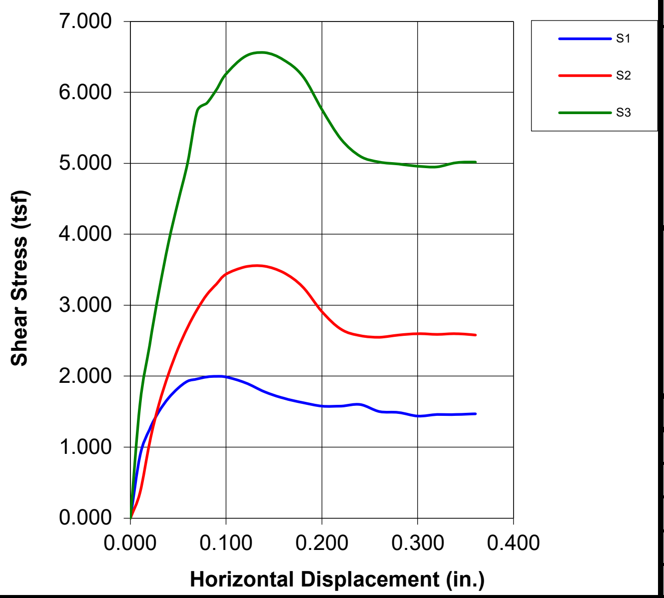
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-39







Specimen Number		1	2	3	
Initial	Water Content, %	17.5	17.8	17.3	
	Dry Density, pcf	111.2	112.4	113.7	
	Void Ratio	0.487	0.471	0.454	
	Saturation, %	95.2	99.9	100.7	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.96	0.96	0.96	
At Test	Water Content, %	20.5	20.9	19.2	
	Dry Density, pcf	109.7	111.0	113.1	
	Void Ratio	0.487	0.471	0.454	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.96	0.96	0.96	

Maximum Shear Stress, tsf	2.00	3.55	6.56	
Residual Shear Stress, tsf	na	na	na	
Hoizontal Displacment, in.	0.090	0.140	0.140	
Normal Stress, tsf	1.94	3.89	7.78	
Strain Rate, in./min.	0.005	0.005	0.005	

LL:	nm	PL:	nm	PI:	nm	G_S:	2.65	Assumed

	c, tsf	ϕ, deg.	Tan ϕ
Failure	0.5	37.9	0.78
Residual	na	na	na

Test Conditions: Undisturbed / Inundated

Specimen 1: Gray Poorly Graded Sand

Specimen 2: Gray Poorly Graded Sand

Specimen 3: Gray Poorly Graded Sand

Boring:	R-20-002
Sample:	S32
Depth, ft:	136-136.5
Test Date:	4/3/2020

Remarks: nm = not measured, na = not applicable

The determination of strength envelopes and the development of relationships to aid in interpreting and evaluating test results are beyond the scope of this test method. The user of this report retains the sole responsibility to evaluate and approve any interpreted values from the testing.

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9969 Horn Rd., Sacramento, CA 95827

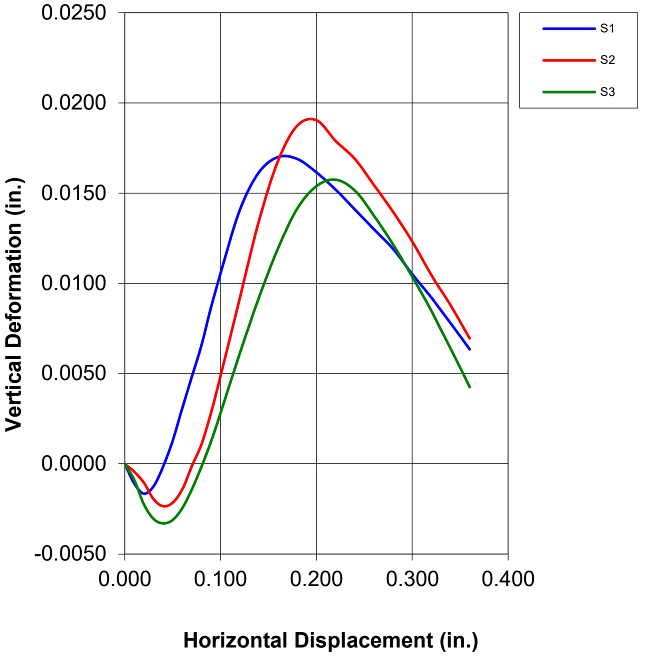
PROJECT NO.:	20180876
ENTRY BY:	A. Wohletz
CHECKED BY:	S. Rader
DATE:	4/6/2020

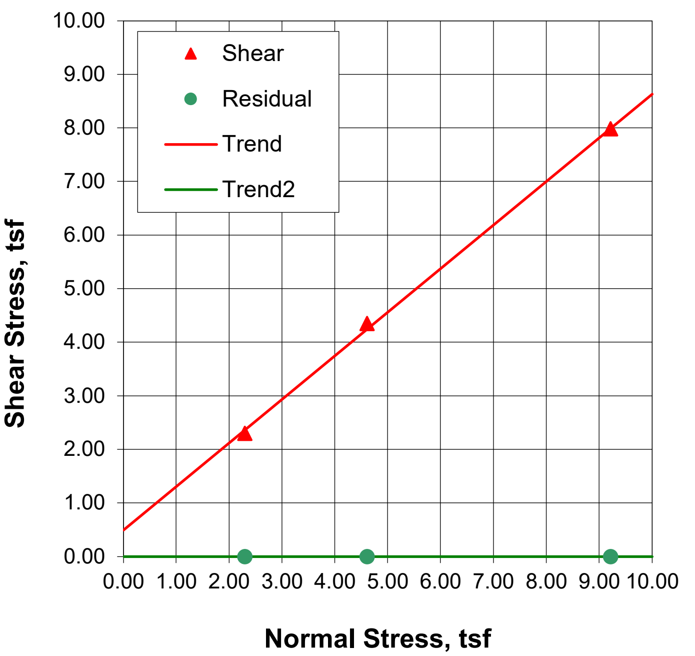
DIRECT SHEAR TEST ASTM D3080

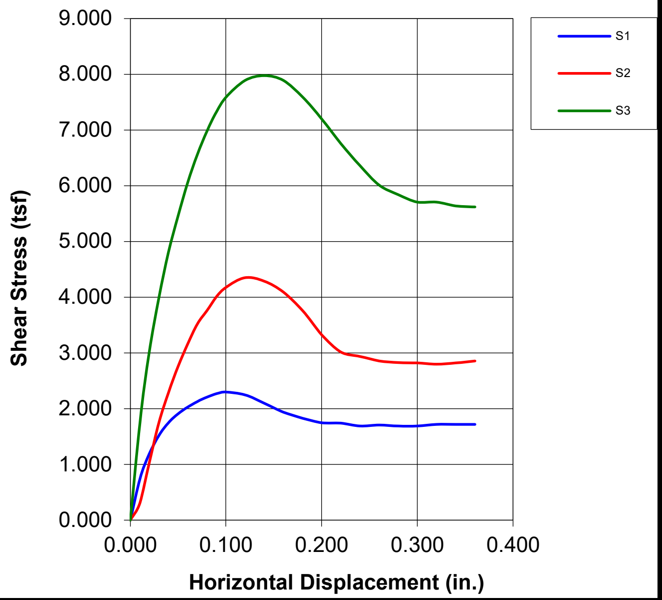
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-40







Specimen Number		1	2	3	
Initial	Water Content, %	15.9	16.2	17.3	
	Dry Density, pcf	109.1	112.3	111.6	
	Void Ratio	0.516	0.473	0.482	
	Saturation, %	81.6	90.7	95.3	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.96	0.96	0.96	
At Test	Water Content, %	20.4	20.4	19.9	
	Dry Density, pcf	110.8	116.9	116.0	
	Void Ratio	0.476	0.401	0.411	
	Diameter, in	2.42	2.42	2.42	
	Height, in	0.93	0.91	0.91	
Maximum Shear Stress, tsf		2.30	4.35	7.98	
Residual Shear Stress, tsf		na	na	na	
Hoizontal Displacment, in.		0.100	0.120	0.140	
Normal Stress, tsf		2.30	4.61	9.22	
Strain Rate, in./min.		0.005	0.005	0.005	

LL:	NM	PL:	NM	PI:	NM	G_S:	2.65	Assumed

	c, tsf	ϕ, deg.	Tan ϕ
Failure	0.5	39.1	0.81
Residual	na	na	na

Test Conditions: Undisturbed / Inundated

Specimen 1: Gray Poorly Graded Sand

Specimen 2: Gray Poorly Graded Sand

Specimen 3: Gray Poorly Graded Sand

Boring:	R-20-002
Sample:	S36
Depth, ft:	161
Test Date:	03/26/2020

Remarks: nm = not measured, na = not applicable

The determination of strength envelopes and the development of relationships to aid in interpreting and evaluating test results are beyond the scope of this test method. The user of this report retains the sole responsibility to evaluate and approve any interpreted values from the testing.

KLEINFELDER
Bright People. Right Solutions.

9969 Horn Rd., Sacramento, CA 95827

PROJECT NO.:	20180876
ENTRY BY:	A. Wohletz
CHECKED BY:	S. Rader
DATE:	03/27/2020

DIRECT SHEAR TEST ASTM D3080

Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-41

APPENDIX D

LOG OF TEST BORINGS (LOTBs)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

PLAN

SCALE: 1"=30'

HOLE ID	STATION AND OFFSET
CPT-20-003	158+93 / 27.79 Rt
R-20-002	159+06 / 23.60 Rt
CPT-20-002	162+90 / 22.51 Rt
R-20-001	166+83 / 23.70 Lt
CPT-20-001	167+03 / 20.16 Lt

ORIGINAL SCALE: 0 1 2 3 4 INCHES

REVISION	DESCRIPTION	APPROVED	DATE

SCALE: AS SHOWN
ACAD FILE NO.
PROJECT NO.

DESIGNED: D. Fahrney
DRAWN: D. Fahrney
CHECKED: J. Bonfiglio

ENGINEER OF WORK
R.C.E. NO.
DATE

KLEINFELDER
Bright people, Right Solutions.
550 WEST C STREET, SUITE 1200 SAN DIEGO, CA. 92101 (619) 831-4600

APPROVED
PUBLIC WORKS DIRECTOR
DATE

CITY OF DEL MAR
CAMINO DEL MAR BRIDGE OVER SAN DIEGUITO RIVER
REPLACEMENT ALTERNATIVE 1 – 6-SPAN OPTION
LOG OF TEST BORINGS

SHEET 1 OF 8 SHEETS

FOR PLAN VIEW AND ADDITIONAL NOTES, SEE "LOG OF TEST BORINGS" SHEET 1 OF 8

ASPHALT CONCRETE; (5")

POORLY GRADED SAND with SILT (SP-SM); yellowish red (5YR 5/6); moist; mostly medium to fine SAND; little fines; non-plastic (ARTIFICIAL FILL (af)).

- yellow (10YR 7/6) and dark yellowish brown (10YR 4/4); coarse to fine SAND

- medium dense; strong brown (7.5YR 4/6); medium to fine SAND

POORLY GRADED SAND with SILT (SP-SM); medium dense; light brownish gray (2.5Y 6/1); moist; mostly fine SAND; little fines; non-plastic (RECENT ALLUVIAL DEPOSITS (Qa)).

- gray (2.5Y 5/1); medium to fine SAND; increase in moisture content

- wet

POORLY GRADED SAND (SP); medium dense; gray (2.5Y 5/1); wet; medium to fine SAND; non-plastic; micaceous

POORLY GRADED SAND with SILT (SP-SM); medium dense; dark gray (2.5Y 4/1); wet; medium to fine SAND; little fines; non-plastic

- loose; few coarse subrounded GRAVEL, 2 in. max. dia.

SILTY SAND (SM); medium dense; dark gray (2.5Y 4/1); wet; mostly medium to fine SAND; little fines

LEAN CLAY (CL); very soft; dark gray (2.5Y 4/1); wet; few fine SAND; mostly fines; medium plasticity

-PP=0.0 tsf

SILTY SAND (SM); loose; dark gray (2.5Y 4/1); wet; mostly medium to fine SAND; little fines; non-plastic; trace shell fragments

FAT CLAY (CH); very soft; dark gray (2.5Y 4/1); wet; few medium SAND; mostly fines; medium to high plasticity; trace roots and shell fragments; PP=0.0 tsf

SILTY SAND with GRAVEL (SM); wet; (inferred from drilling action)

- subrounded gravel (3") inside sampler

SILTY SAND (SM); dense; dark gray (2.5Y 4/1); wet; mostly coarse to fine SAND; little fines; non-plastic; trace shell fragments (YOUNG ALLUVIAL DEPOSITS (Qya)).

SILTY SAND (SM); dense; dark gray (2.5Y 4/1); wet; mostly medium to fine SAND; little fines; non-plastic

- very dense; micaceous

- loose; PP=0.0 tsf

SANDY LEAN CLAY (CL); very soft; dark gray (2.5Y 4/1); wet; some fine SAND; mostly fines; low to medium plasticity

POORLY GRADED SAND with SILT (SP-SM); very dense; dark gray (2.5Y 4/1); wet; mostly medium to fine SAND; little fines; non-plastic; trace shell fragments

- dense

MATCHLINE
(See "Log of Test Borings 3 of 8")

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

PROFILE

HOR. 1"=30'
VERT. 1"=5'

REVISION	DESCRIPTION	APPROVED	DATE

SCALE: AS SHOWN
ACAD FILE NO.
PROJECT NO.

DESIGNED: D. Fahrney
DRAWN: D. Fahrney
CHECKED: J. Bonfiglio
ENGINEER OF WORK
R.C.E. NO.
DATE

KLEINFELDER
Bright people. Right Solutions.
550 WEST C STREET, SUITE 1200 SAN DIEGO, CA. 92101 (619) 831-4600

APPROVED
PUBLIC WORKS DIRECTOR
DATE

CITY OF DEL MAR
CAMINO DEL MAR BRIDGE OVER SAN DIEGUITO RIVER
REPLACEMENT ALTERNATIVE 1 – 6-SPAN OPTION
LOG OF TEST BORINGS

SHEET 2 OF 8 SHEETS

ORIGINAL SCALE: 0 1 INCHES 2 3 4

FOR PLAN VIEW AND ADDITIONAL NOTES, SEE "LOG OF TEST BORINGS" SHEET 1 OF 8

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

PROFILE

HOR. 1"=30'
VERT. 1"=5'

ORIGINAL SCALE: 0 1 INCHES 2 3 4

REVISION	DESCRIPTION	APPROVED	DATE

SCALE: AS SHOWN
ACAD FILE NO.
PROJECT NO.

DESIGNED: D. Fahrney
DRAWN: D. Fahrney
CHECKED: J. Bonfiglio
ENGINEER OF WORK
R.C.E. NO.
DATE

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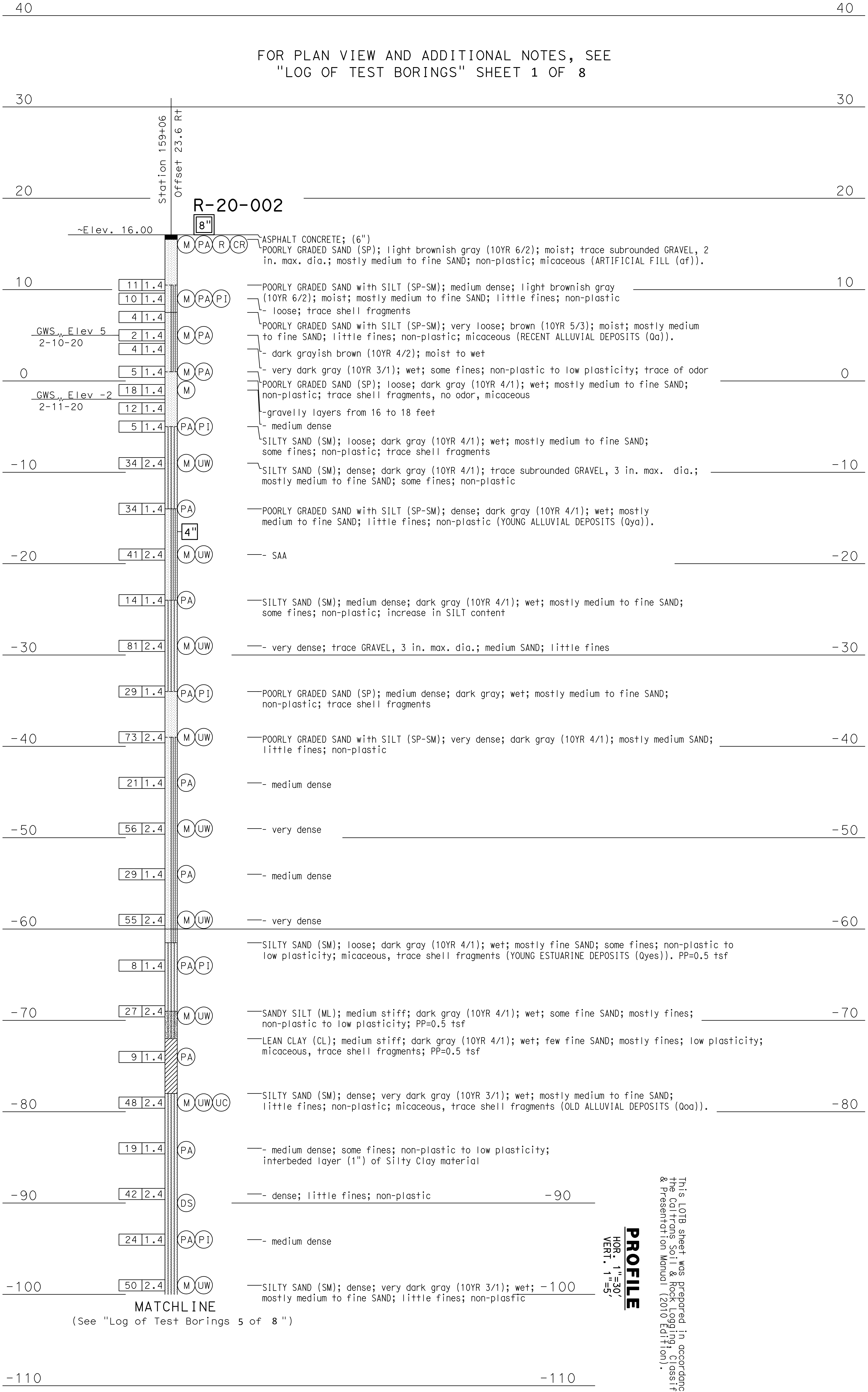
APPROVED
PUBLIC WORKS DIRECTOR
DATE

CITY OF DEL MAR

CAMINO DEL MAR BRIDGE OVER SAN DIEGUITO RIVER
REPLACEMENT ALTERNATIVE 1 – 6-SPAN OPTION
LOG OF TEST BORINGS

SHEET 3 OF 8 SHEETS

FOR PLAN VIEW AND ADDITIONAL NOTES, SEE "LOG OF TEST BORINGS" SHEET 1 OF 8



ASPHALT CONCRETE; (6")

POORLY GRADED SAND (SP); light brownish gray (10YR 6/2); moist; trace subrounded GRAVEL, 2 in. max. dia.; mostly medium to fine SAND; non-plastic; micaceous (ARTIFICIAL FILL (af)).

POORLY GRADED SAND with SILT (SP-SM); medium dense; light brownish gray (10YR 6/2); moist; mostly medium to fine SAND; little fines; non-plastic

- loose; trace shell fragments

POORLY GRADED SAND with SILT (SP-SM); very loose; brown (10YR 5/3); moist; mostly medium to fine SAND; little fines; non-plastic; micaceous (RECENT ALLUVIAL DEPOSITS (Qa)).

- dark grayish brown (10YR 4/2); moist to wet

- very dark gray (10YR 3/1); wet; some fines; non-plastic to low plasticity; trace of odor

POORLY GRADED SAND (SP); loose; dark gray (10YR 4/1); wet; mostly medium to fine SAND; non-plastic; trace shell fragments, no odor, micaceous

-gravelly layers from 16 to 18 feet

- medium dense

SILTY SAND (SM); loose; dark gray (10YR 4/1); wet; mostly medium to fine SAND; some fines; non-plastic; trace shell fragments

SILTY SAND (SM); dense; dark gray (10YR 4/1); trace subrounded GRAVEL, 3 in. max. dia.; mostly medium to fine SAND; some fines; non-plastic

POORLY GRADED SAND with SILT (SP-SM); dense; dark gray (10YR 4/1); wet; mostly medium to fine SAND; little fines; non-plastic (YOUNG ALLUVIAL DEPOSITS (Qya)).

- SAA

SILTY SAND (SM); medium dense; dark gray (10YR 4/1); wet; mostly medium to fine SAND; some fines; non-plastic; increase in SILT content

- very dense; trace GRAVEL, 3 in. max. dia.; medium SAND; little fines

POORLY GRADED SAND (SP); medium dense; dark gray; wet; mostly medium to fine SAND; non-plastic; trace shell fragments

POORLY GRADED SAND with SILT (SP-SM); very dense; dark gray (10YR 4/1); mostly medium SAND; little fines; non-plastic

- medium dense

- very dense

- medium dense

- very dense

SILTY SAND (SM); loose; dark gray (10YR 4/1); wet; mostly fine SAND; some fines; non-plastic to low plasticity; micaceous, trace shell fragments (YOUNG ESTUARINE DEPOSITS (Qyes)). PP=0.5 tsf

SANDY SILT (ML); medium stiff; dark gray (10YR 4/1); wet; some fine SAND; mostly fines; non-plastic to low plasticity; PP=0.5 tsf

LEAN CLAY (CL); medium stiff; dark gray (10YR 4/1); wet; few fine SAND; mostly fines; low plasticity; micaceous, trace shell fragments; PP=0.5 tsf

SILTY SAND (SM); dense; very dark gray (10YR 3/1); wet; mostly medium to fine SAND; little fines; non-plastic; micaceous, trace shell fragments (OLD ALLUVIAL DEPOSITS (Qoa)).

- medium dense; some fines; non-plastic to low plasticity; interbeded layer (1") of Silty Clay material

- dense; little fines; non-plastic

- medium dense

SILTY SAND (SM); dense; very dark gray (10YR 3/1); wet; mostly medium to fine SAND; little fines; non-plasfic

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

PROFILE

HOR. 1"=30'
VERT. 1"=5'

REVISION	DESCRIPTION	APPROVED	DATE

SCALE: AS SHOWN
ACAD FILE NO.
PROJECT NO.

DESIGNED: D. Fahrney
DRAWN: D. Fahrney
CHECKED: J. Bonfiglio
ENGINEER OF WORK
R.C.E. NO.
DATE

KLEINFELDER
Bright people. Right Solutions.
550 WEST C STREET, SUITE 1200 SAN DIEGO, CA. 92101 (619) 831-4600

APPROVED
PUBLIC WORKS DIRECTOR
DATE

CITY OF DEL MAR
CAMINO DEL MAR BRIDGE OVER SAN DIEGUITO RIVER
REPLACEMENT ALTERNATIVE 1 – 6-SPAN OPTION
LOG OF TEST BORINGS

SHEET 4 OF 8 SHEETS

ORIGINAL SCALE: 0 1 INCHES 2 3 4

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

PROFILE

HOR. 1"=30'
VERT. 1"=5'

FOR PLAN VIEW AND ADDITIONAL NOTES, SEE "LOG OF TEST BORINGS" SHEET 1 OF 8

- medium dense; coarse to medium SAND

- very dense

- medium dense

- very dense

- loose; some medium to fine SAND; some fines; non-plastic to low plasticity; micaceous, increase in SILT content; PP=0.5 tsf

SILTY SAND (SM); very dense; very dark gray (10YR 3/1); wet; mostly medium to fine SAND; little fines; non-plastic

POORLY GRADED SAND with SILT (SP-SM); dense; very dark gray; wet; mostly medium to fine SAND; little fines; non-plastic

- very dense; coarse to medium SAND

SILTY SAND (SM); medium dense; very dark gray; wet; some fines; non-plastic to low plasticity

SILTY SAND (SM); medium dense; very dark gray (10YR 3/1); wet; mostly coarse to medium SAND; some fines; non-plastic to low plasticity

SILTY CLAY with SAND (CL-ML); stiff; very dark gray (10YR 3/1); wet; little SAND; mostly fines; medium plasticity; PP=1.0 tsf

SILTY SAND (SM); very dense; very dark gray (10YR 3/1); wet; mostly medium to fine SAND; little fines; non-plastic

- dense

- very dense; olive gray (5Y 5/2); trace subrounded GRAVEL, 1 in. max. dia.; coarse to medium SAND; little fines; iron oxide staining

SILTY SAND (SM); very dense; olive gray (5Y 5/2); trace subrounded GRAVEL, 1 in. max. dia.; mostly coarse to medium SAND; little fines

02-13-20
Terminated at Elev ~-192.00
Hammer Energy Ratio (ERi) = 94%

REVISION	DESCRIPTION	APPROVED	DATE

SCALE: AS SHOWN
ACAD FILE NO.
PROJECT NO.

DESIGNED: D. Fahrney
DRAWN: D. Fahrney
CHECKED: J. Bonfiglio

ENGINEER OF WORK R.C.E. NO. DATE

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550 WEST C STREET, SUITE 1200 SAN DIEGO, CA. 92101 (619) 831-4600

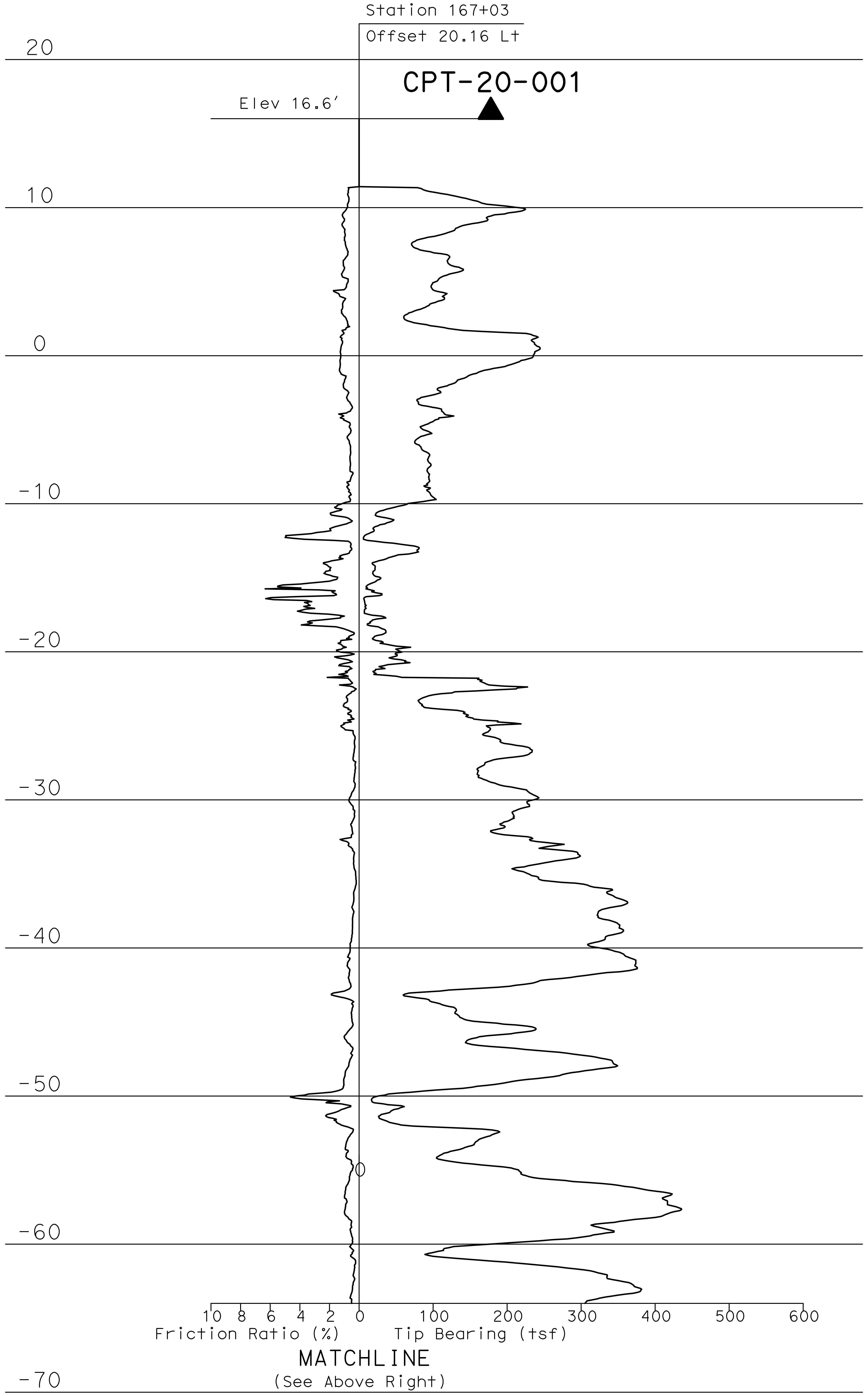
APPROVED

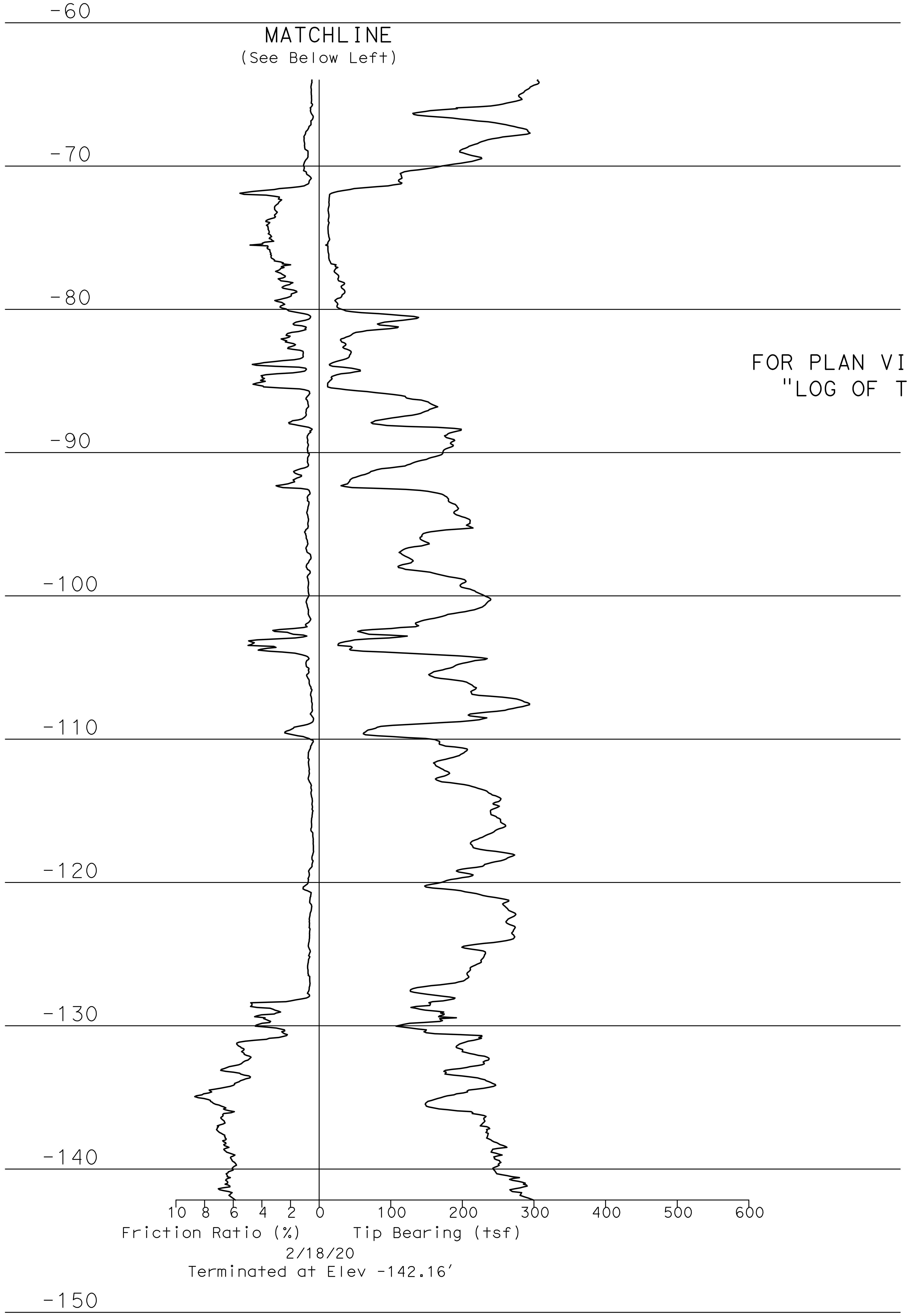
PUBLIC WORKS DIRECTOR DATE

CITY OF DEL MAR
CAMINO DEL MAR BRIDGE OVER SAN DIEGUITO RIVER
REPLACEMENT ALTERNATIVE 1 – 6-SPAN OPTION
LOG OF TEST BORINGS

SHEET 5 OF 8 SHEETS

ORIGINAL SCALE: 0 1 2 3 4 INCHES





2/18/20
Terminated at Elev -142.16'

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

FOR PLAN VIEW AND ADDITIONAL NOTES, SEE "LOG OF TEST BORINGS" SHEET 1 OF 8

REVISION	DESCRIPTION	APPROVED	DATE

SCALE: AS SHOWN
ACAD FILE NO.
PROJECT NO.

DESIGNED: D. Fahrney
DRAWN: D. Fahrney
CHECKED: J. Bonfiglio

ENGINEER OF WORK
R.C.E. NO.
DATE

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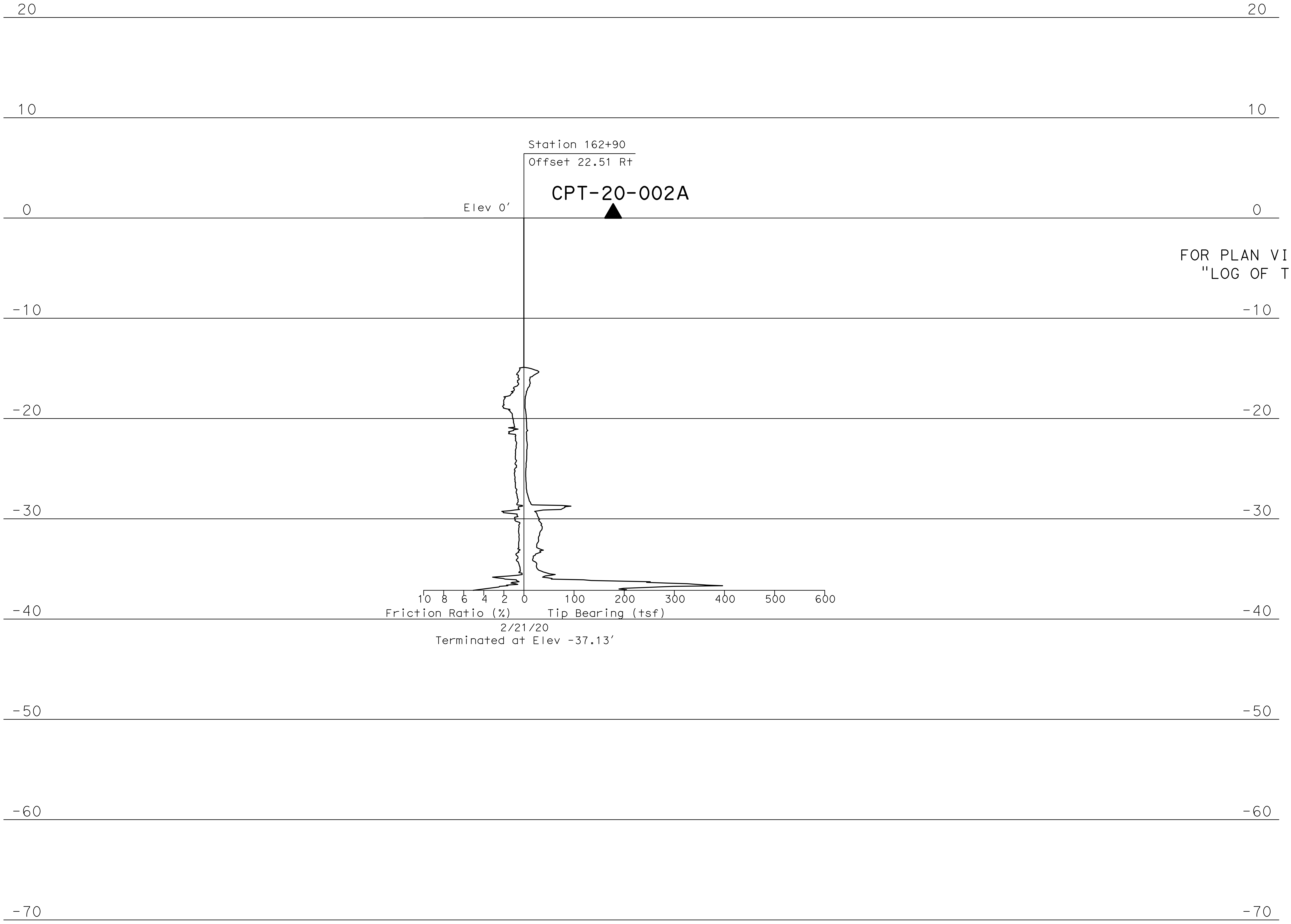
APPROVED
PUBLIC WORKS DIRECTOR
DATE

CITY OF DEL MAR
CAMINO DEL MAR BRIDGE OVER SAN DIEGUITO RIVER
REPLACEMENT ALTERNATIVE 1 – 6-SPAN OPTION
LOG OF TEST BORINGS

SHEET
6
OF 8 SHEETS

ORIGINAL SCALE: 0 1 INCHES 2 3 4



This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

FOR PLAN VIEW AND ADDITIONAL NOTES, SEE "LOG OF TEST BORINGS" SHEET 1 OF 8

REVISION	DESCRIPTION	APPROVED	DATE

SCALE: AS SHOWN
ACAD FILE NO.
PROJECT NO.

DESIGNED: D. Fahrney
DRAWN: D. Fahrney
CHECKED: J. Bonfiglio
ENGINEER OF WORK
R.C.E. NO.
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550 WEST C STREET, SUITE 1200 SAN DIEGO, CA. 92101 (619) 831-4600

APPROVED
PUBLIC WORKS DIRECTOR
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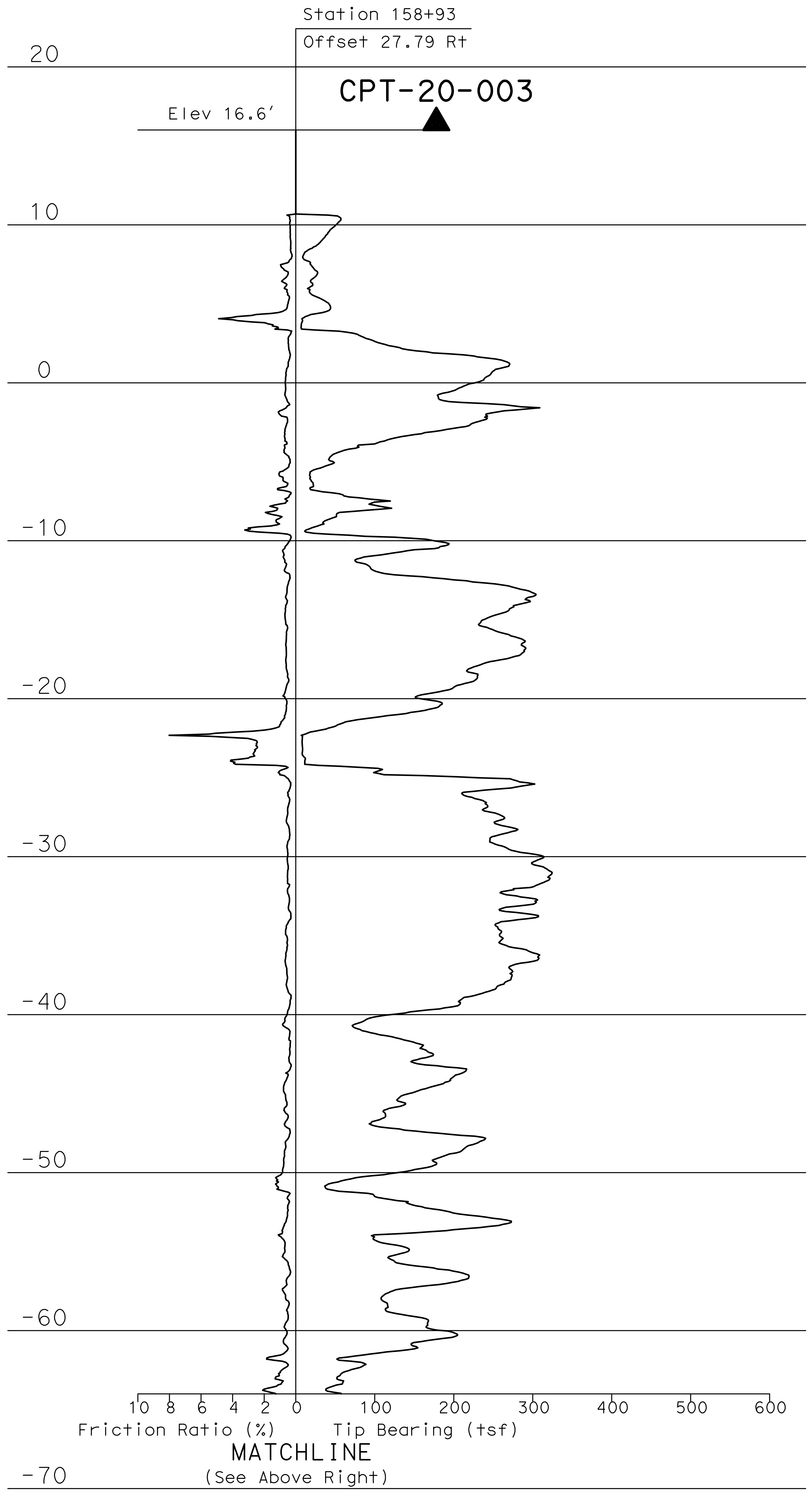
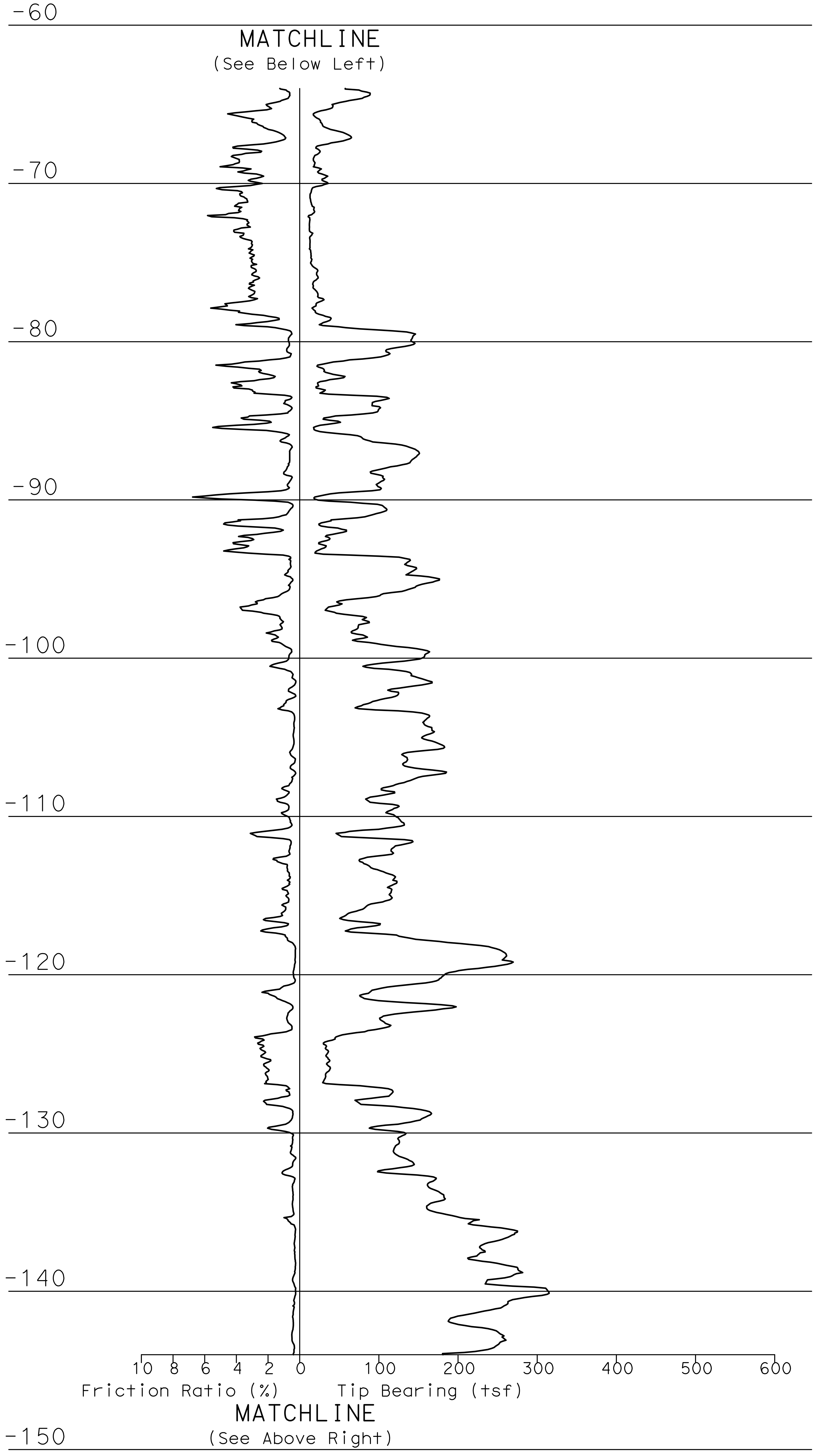
CITY OF DEL MAR
CAMINO DEL MAR BRIDGE OVER SAN DIEGUITO RIVER
REPLACEMENT ALTERNATIVE 1 – 6-SPAN OPTION
LOG OF TEST BORINGS

SHEET 7 OF 8 SHEETS

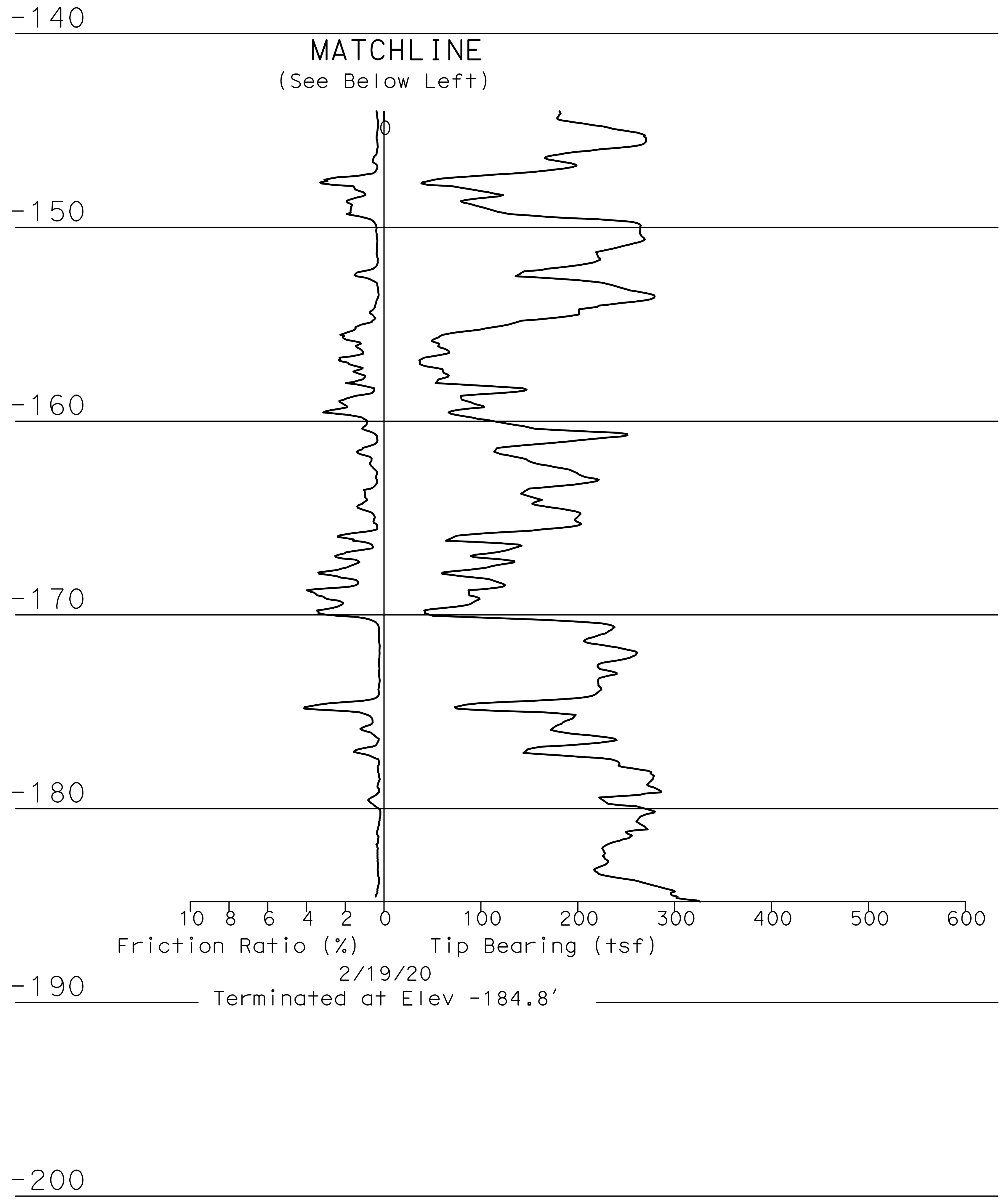
ORIGINAL SCALE: INCHES 0 1 2 3 4





This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

FOR PLAN VIEW AND ADDITIONAL NOTES, SEE "LOG OF TEST BORINGS" SHEET 1 OF 8



2/19/20
Terminated at Elev -184.8'

REVISION	DESCRIPTION	APPROVED	DATE

SCALE: AS SHOWN
ACAD FILE NO.
PROJECT NO.

DESIGNED: D. Fahrney
DRAWN: D. Fahrney
CHECKED: J. Bonfiglio
ENGINEER OF WORK
R.C.E. NO.
DATE

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APPROVED
PUBLIC WORKS DIRECTOR
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CITY OF DEL MAR
CAMINO DEL MAR BRIDGE OVER SAN DIEGUITO RIVER
REPLACEMENT ALTERNATIVE 1 – 6-SPAN OPTION
LOG OF TEST BORINGS

SHEET 8 OF 8 SHEETS

ORIGINAL SCALE: 0 1 2 3 4 INCHES

APPENDIX E

PREVIOUS RELEVANT GEOTECHNICAL INFORMATION BY OTHERS

APPENDIX E
PREVIOUS RELEVANT GEOTECHNICAL INFORMATION BY OTHERS

E.1 California Department of Public Works 1960 LOTBs

E.2 Ninyo & Moore 2018 Exploration Logs

E.3 Ninyo & Moore 2018 Laboratory Test Results

E.1 California
Department of
Public Works 1960
LOTBs

GEOLOGIC LEGEND

SOIL TYPES

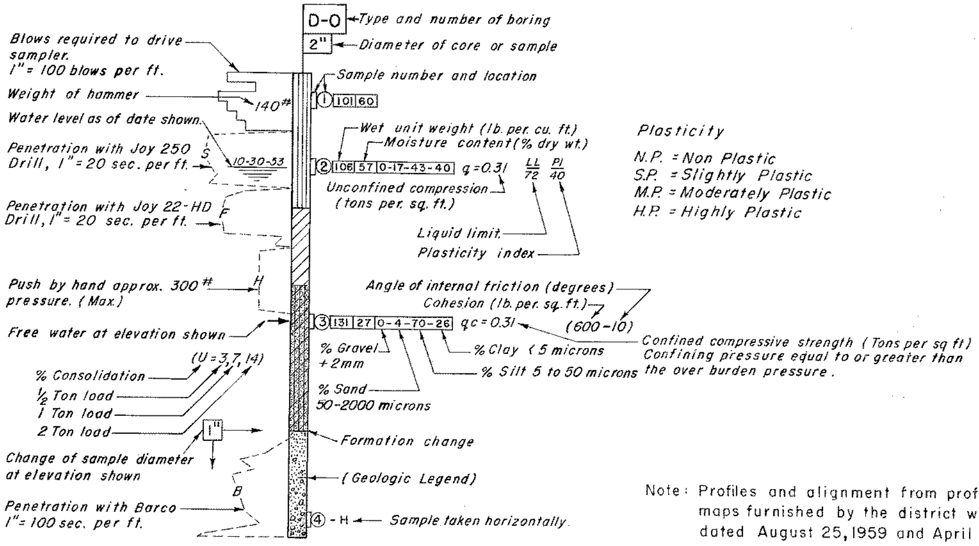
- ▨ Gravel
- ▨ Sand
- ▨ Silt
- ▨ Clay
- ▨ Claypan
- ▨ Hardpan
- ▨ Sandy Gravel
- ▨ Gravelly Sand
- ▨ Loam
- ▨ Sandy Loam
- ▨ Silty Sand
- ▨ Sandy Silt
- ▨ Sandy Clay
- ▨ Clayey Sand
- ▨ Silty Clay
- ▨ Clayey Silt
- ▨ Peat
- ▨ Topsoil
- ▨ Organic Silt
- ▨ Organic Clay
- ▨ Shells
- ▨ Fill

ROCK TYPES

- ▨ Sandstone
- ▨ Shale
- ▨ Slate
- ▨ Schist
- ▨ Volcanics
- ▨ Unclassified Intrusives
- ▨ Ultra-basic Intrusives (Serpentine)
- ▨ Sandstone
- ▨ Shale
- ▨ Unclass.-Intrus.

} Decomposed Weathered or Fractured

BORING LEGEND
CROSS-SECTION & PROFILE SHEETS



Plasticity

N.P. = Non Plastic
S.P. = Slightly Plastic
M.P. = Moderately Plastic
H.P. = Highly Plastic

Note: Profiles and alignment from profile and contour maps furnished by the district with letters dated August 25, 1959 and April 8, 1960.

NOTE - All borings are numbered consecutively regardless of type of boring

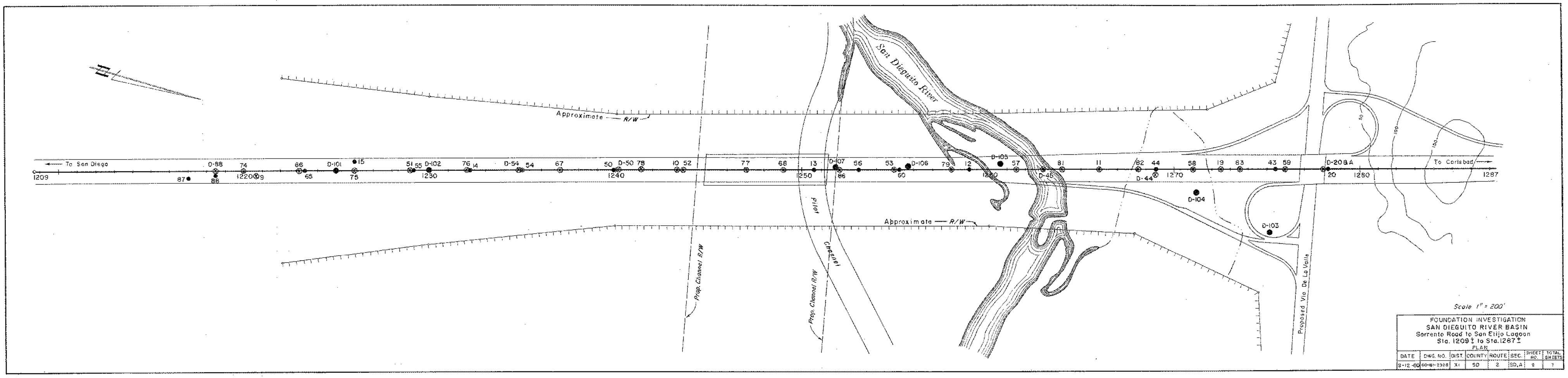
BORING LEGEND - PLAN SHEETS

● A-O Auger (Continuous helix)
● B-O Bucket Boring
● C-O Core Boring
● C-O(E) Electric log of Boring
● D-O Drive Boring made by Hdqtrs. Lab.
● D-O(P) Probe Boring
● -O1" Boring placed by Dist.
● R-O Rotary Boring
● J-O Jetted Boring
× V-O Vane borer test
■ SP-O Settlement Platform
▲ Z-O Piezometer
⊗ 2" Boring placed by Dist.

STATE OF CALIFORNIA DIVISION OF HIGHWAYS
MATERIALS & RESEARCH DEPARTMENT

FOUNDATION INVESTIGATION
Road XI - SD - 2 - SD,A
Sorrento Road to San Elijo Lagoon
San Dieguito River Basin
LEGEND SHEET

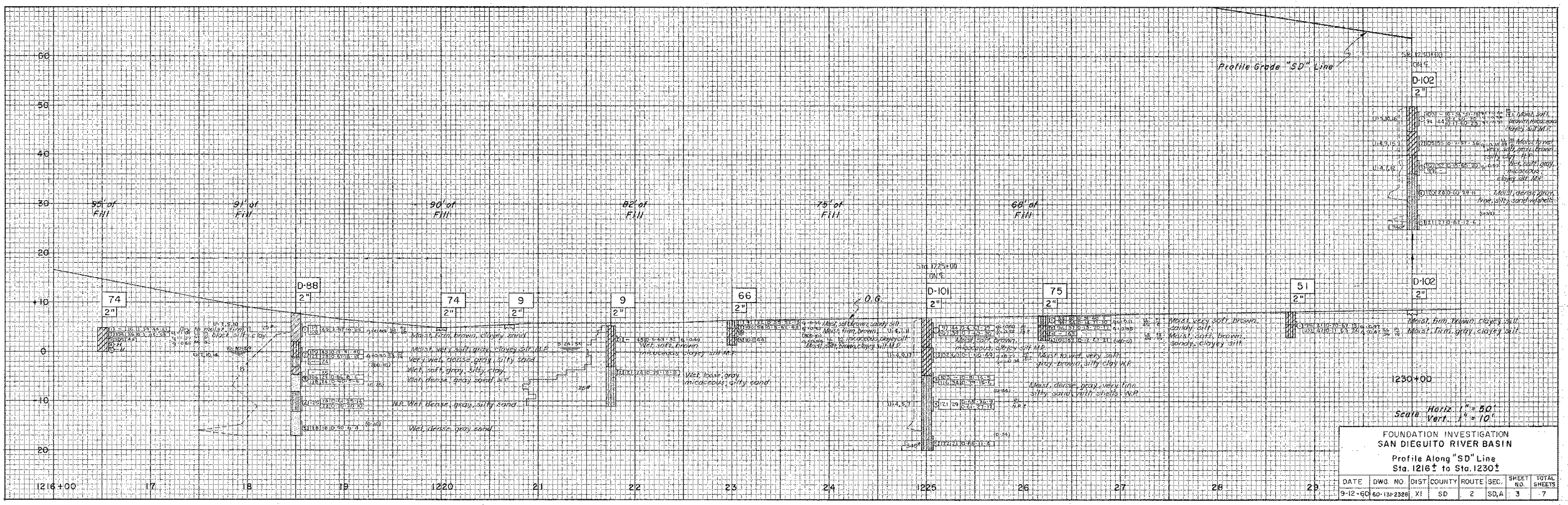
DWG. NO. 60-130-2328
DRAWN BY R.P.
CHECKED BY W.F.K.
SUBMITTED BY: A.W. Root
SUPV. MATERIALS & RESEARCH ENG'R.
APPROVED BY: F.N. Hveem
MATERIALS & RESEARCH ENG'R.
DATE 9-12-60
SHEET NO. 1 OF 7 SHEETS



Scale 1" = 200'

FOUNDATION INVESTIGATION
SAN DIEGUITO RIVER BASIN
Sorrento Road to San Elijo Lagoon
Sta. 1209± to Sta. 1287±
PLAN

DATE	DWG. NO.	DIST.	COUNTY	ROUTE	SEC.	SHEET NO.	TOTAL SHEETS
9-12-60	60-161-2328	XI	SD	2	SD,A	2	7

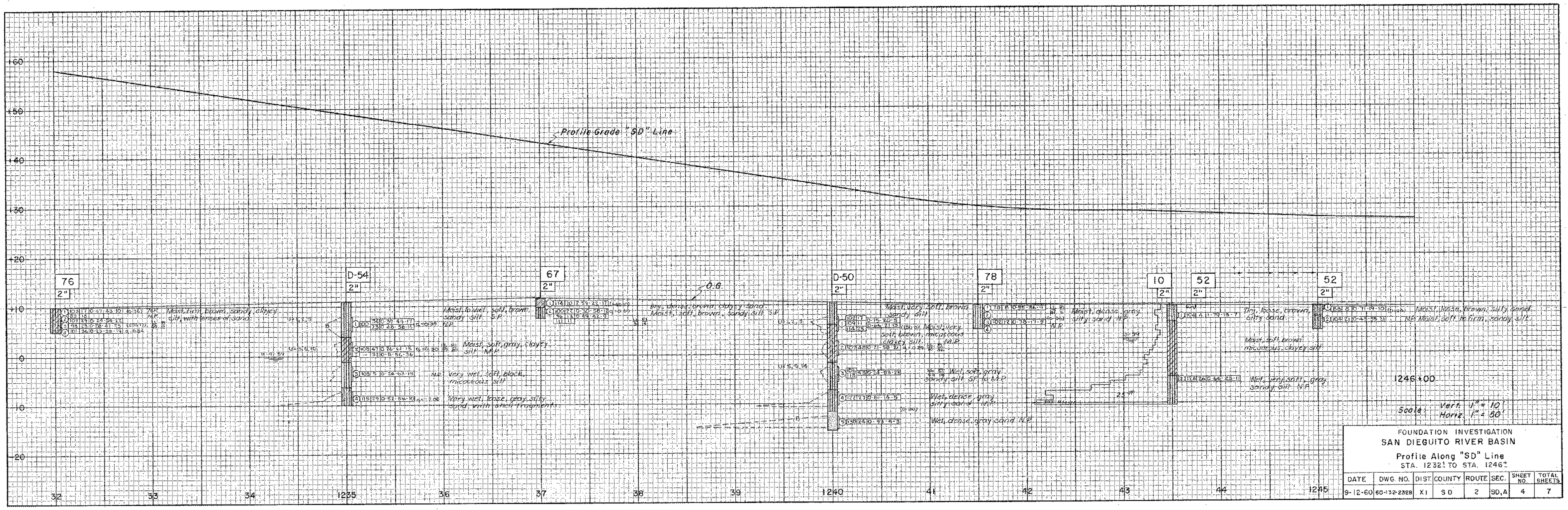


Scale Horiz. 1" = 50'
Vert. 1" = 10'

FOUNDATION INVESTIGATION
SAN DIEGUITO RIVER BASIN

Profile Along "SD" Line
Sta. 1216± to Sta. 1230±

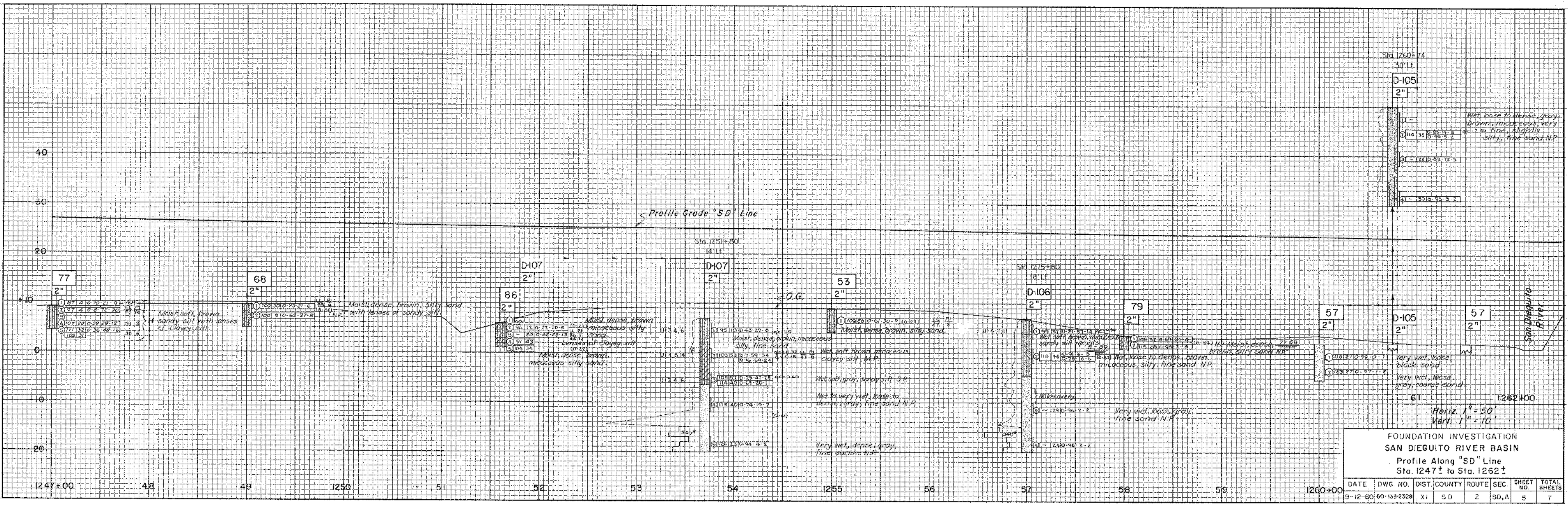
DATE	DWG. NO.	DIST.	COUNTY	ROUTE	SEC.	SHEET NO.	TOTAL SHEETS
9-12-60	60-131-2328	XI	SD	2	SD,A	3	7



FOUNDATION INVESTIGATION
SAN DIEGUITO RIVER BASIN

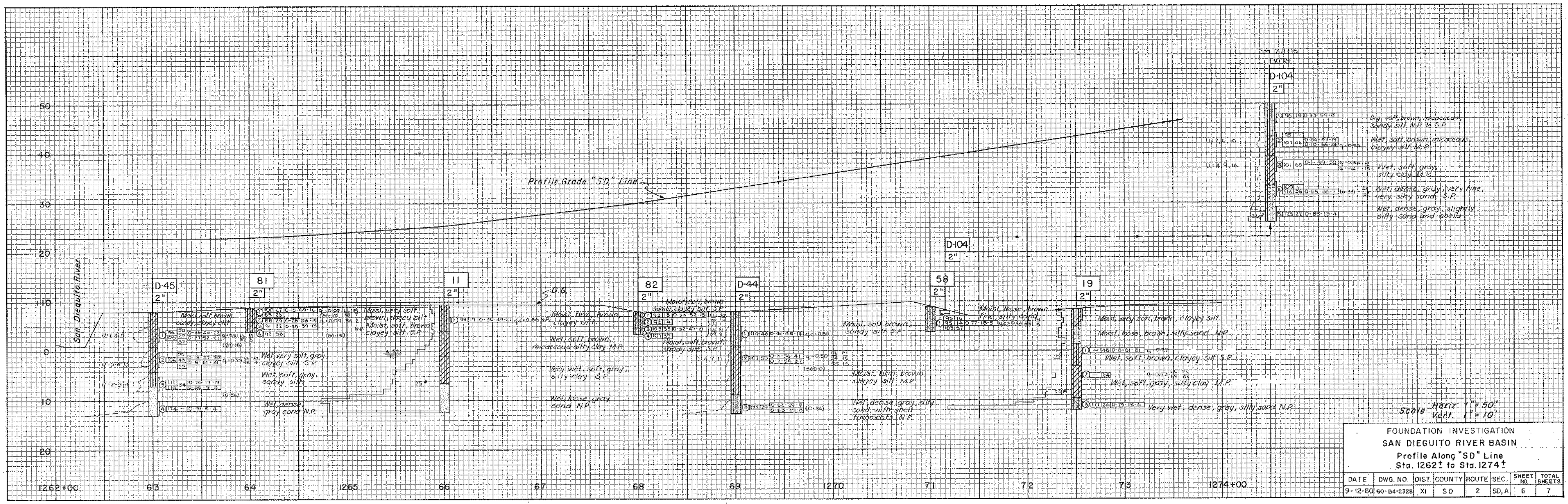
Profile Along "SD" Line
STA. 1232± TO STA. 1246±

DATE	DWG. NO.	DIST.	COUNTY	ROUTE	SEC.	SHEET NO.	TOTAL SHEETS
9-12-60	60-132-2328	XI	SD	2	SD,A	4	7



FOUNDATION INVESTIGATION
SAN DIEGUITO RIVER BASIN
Profile Along "SD" Line
Sta. 1247± to Sta. 1262±

DATE	DWG. NO.	DIST.	COUNTY	ROUTE	SEC.	SHEET NO.	TOTAL SHEETS
9-12-60	60-133-2328	XI	S D	2	SD,A	5	7



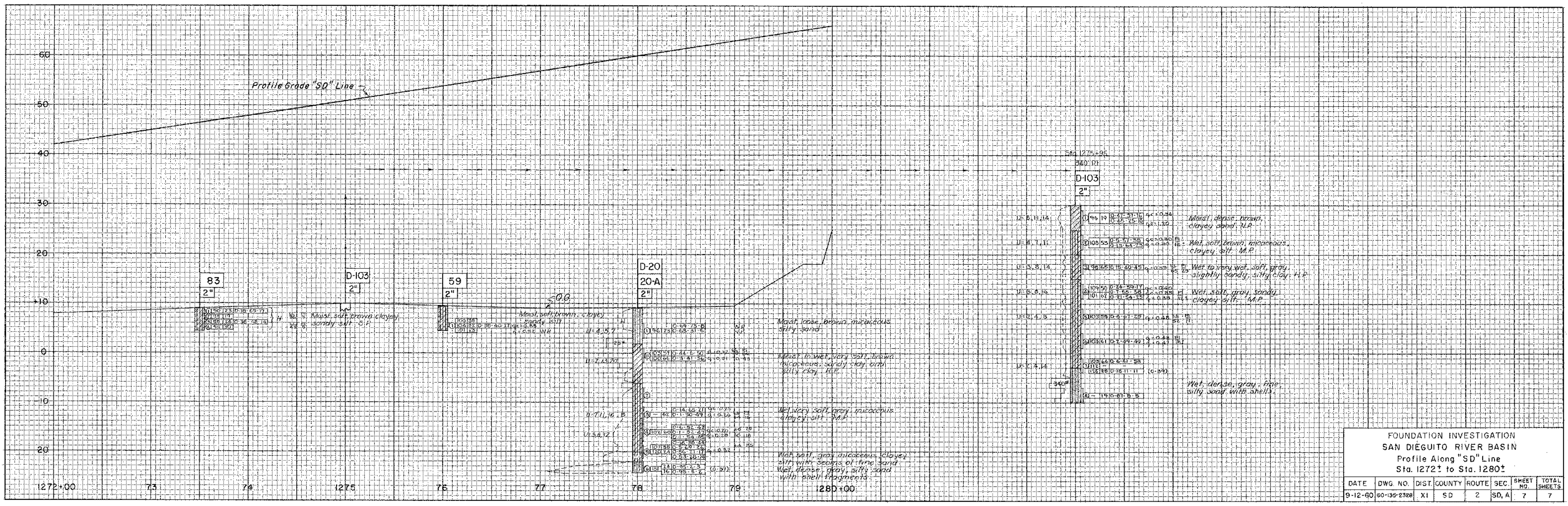
FOUNDATION INVESTIGATION

SAN DIEGUITO RIVER BASIN

Profile Along "SD" Line

Sta. 1262± to Sta. 1274±

DATE	DWG. NO.	DIST.	COUNTY	ROUTE	SEC.	SHEET NO.	TOTAL SHEETS
9-12-60	60-134-2328	XI	SD	2	SD, A	6	7



FOUNDATION INVESTIGATION
SAN DIÉGUITO RIVER BASIN
Profile Along "SD" Line
Sta. 1272± to Sta. 1280±

DATE	DWG. NO.	DIST.	COUNTY	ROUTE	SEC.	SHEET NO.	TOTAL SHEETS
9-12-60	60-135-2328	XI	SD	2	SD, A	7	7

STRATIOGRAPHIC·SECTION

SITE·PROPOSED·BRIDGE·OVER
SAN·DIEGUITO·RIVER
VII - S·D - 2 - A

FEB. 1931
SCALES Hor. 1"=50ft.
Vert 1"=10ft.

484

LEGEND

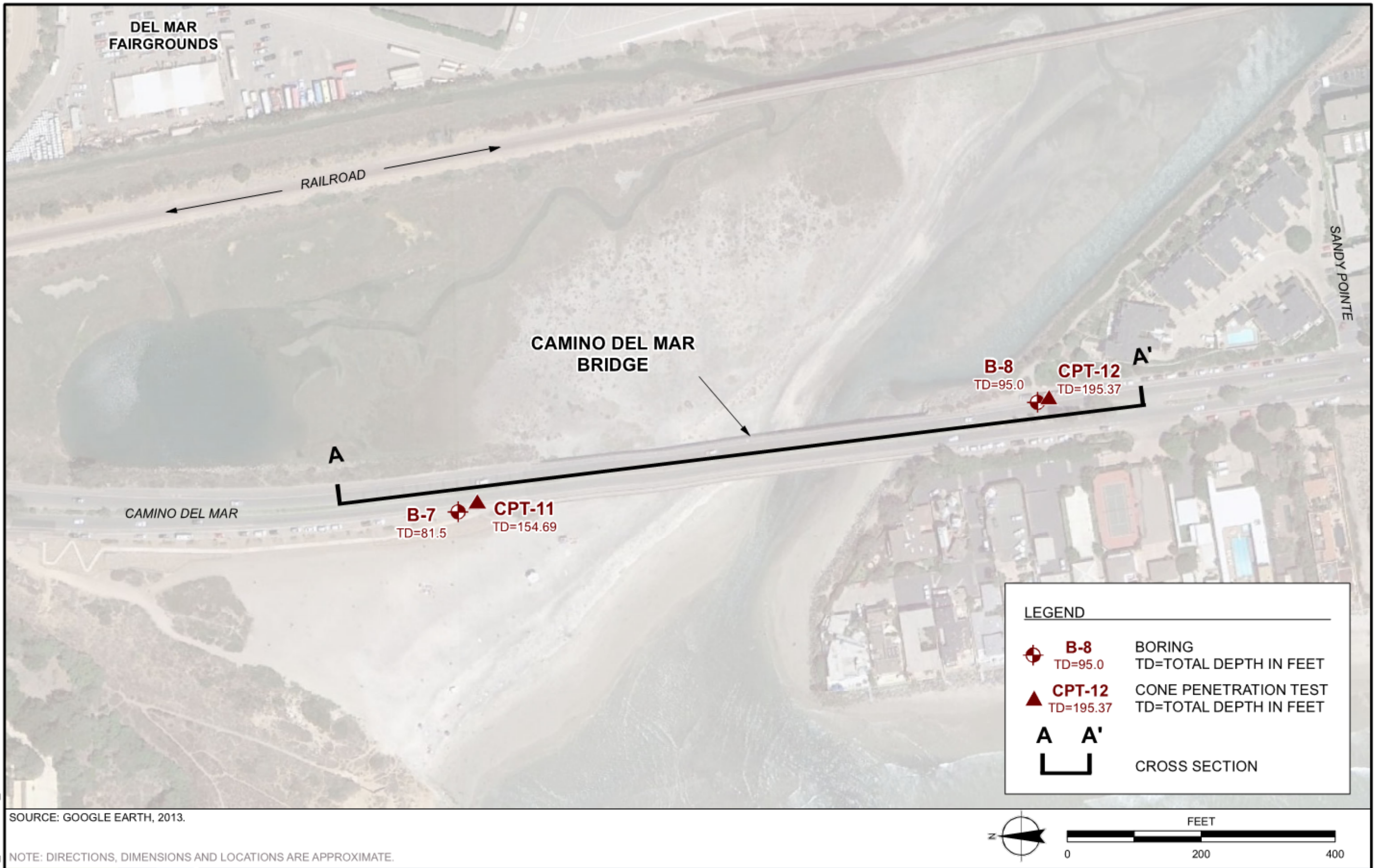
▨ CLAY

☐ SAND

░ GRAVEL

⁚ SHELLS

E.2 Ninyo & Moore
2018 Exploration
Logs



SOURCE: GOOGLE EARTH, 2013.

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

2_108449001_SSE.mxd 7/31/2018 AOB

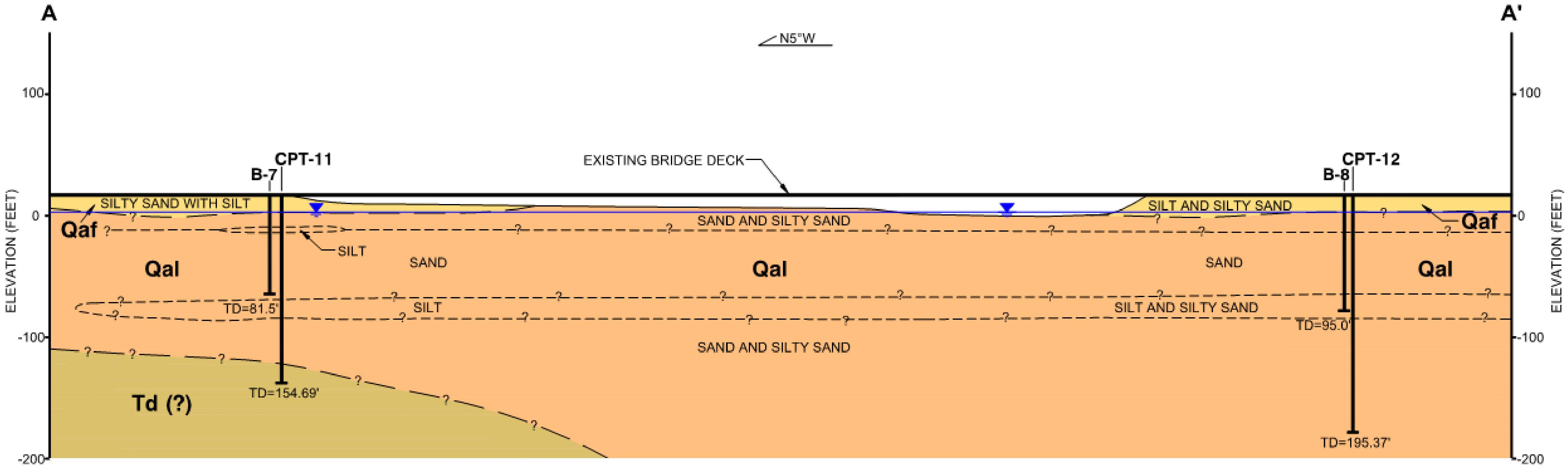
FIGURE 2

SUBSURFACE EXPLORATIONS

CAMINO DEL MAR BRIDGE REPLACEMENT
DEL MAR, CALIFORNIA

108449001 | 7/18

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



LEGEND

B-8 TD=95.0'	BORING TD=TOTAL DEPTH IN FEET
CPT-12 TD=195.37'	CPT BORING TD=TOTAL DEPTH IN FEET
Qaf	FILL
Qal	ALLUVIUM
Td	DELMAR FORMATION
— ? —	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
▼	GROUNDWATER ELEVATION

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

4 108449001 CS A-A'.DWG

FIGURE 4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GEOLOGIC CROSS SECTION A-A'

CAMINO DEL MAR BRIDGE REPLACEMENT
DEL MAR, CALIFORNIA

108449001 I 7/18

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
11	SD	–	--	4	5

Soumitra Guha --/--/--
REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

SOUMITRA GUHA No. GE 2632 Exp. 06/30/19

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--

NINYO & MOORE
5710 Ruffin Road
San Diego, CA 92123
Phone (858) 576-1000

BENCH MARK

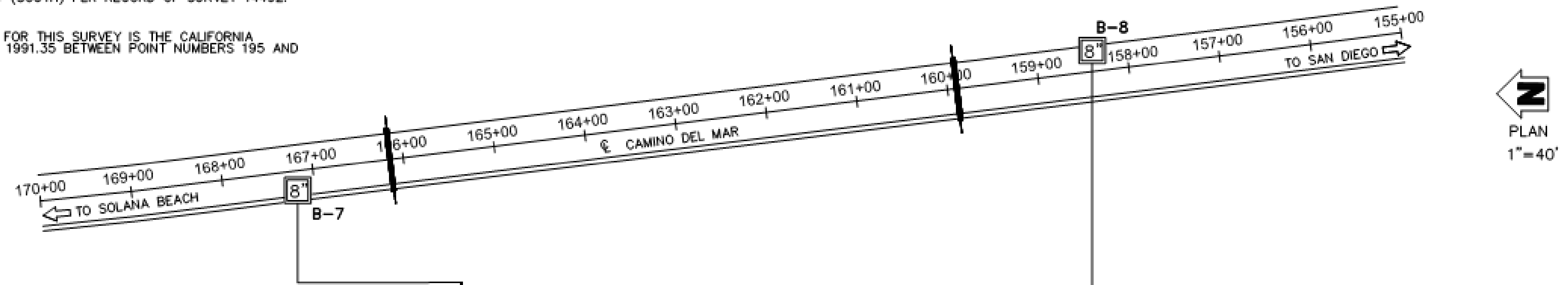
VERTICAL BENCHMARK: 2" CO SD BRASS DISK IN MON WELL, PC JIMMY DURANTE BOULEVARD 600 FEET NORTHEAST OF JIMMY DURANTE WAY (SOUTH) PER RECORD OF SURVEY 14492.

DATUM: NAVD88 ELEVATION: 13.32'

BASIS OF BEARINGS: THE BASIS OF BEARINGS FOR THIS SURVEY IS THE CALIFORNIA COORDINATE SYSTEM NAD 83, ZONE 6, EPOCH 1991.35 BETWEEN POINT NUMBERS 195 AND 459 AS SHOWN ON ROS 14492.

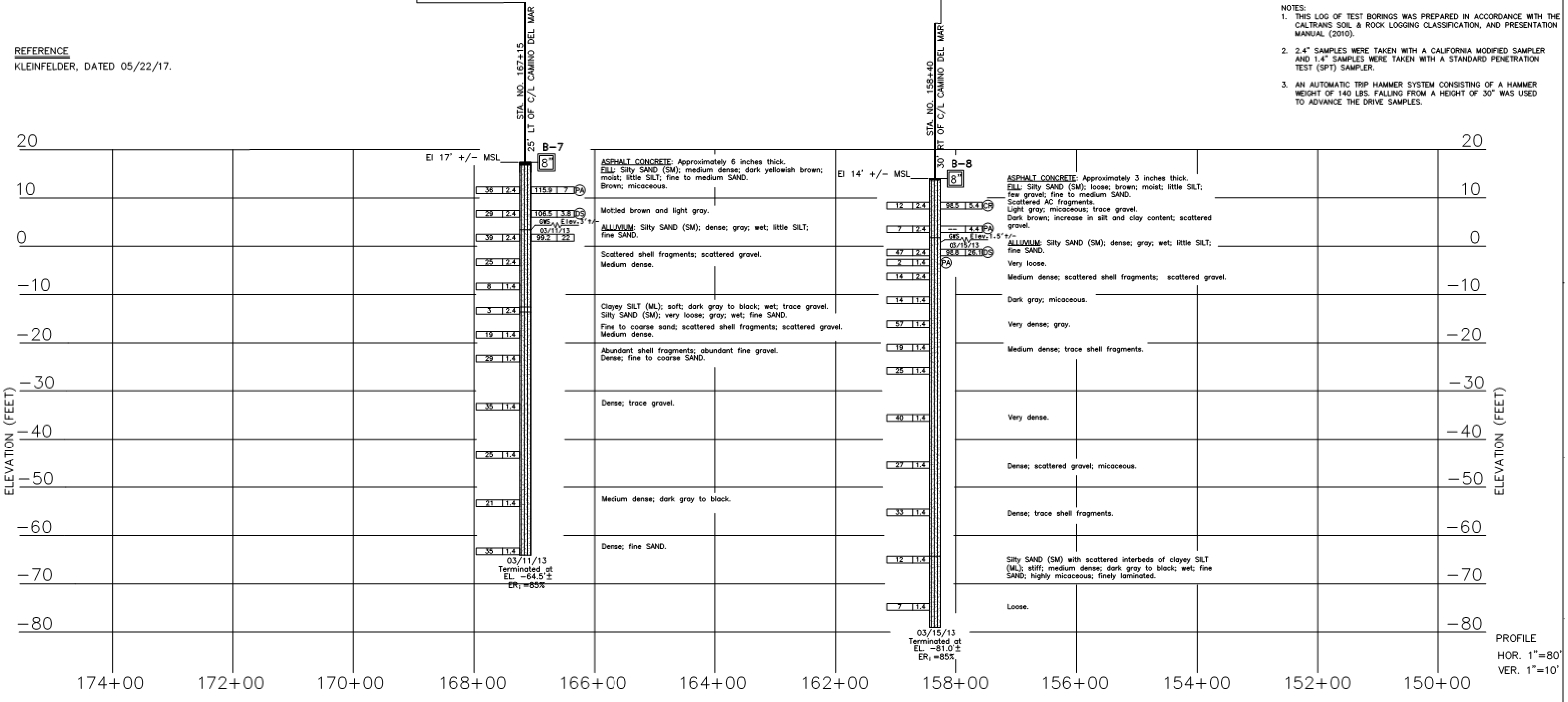
REFERENCE

KLEINFELDER, DATED 05/22/17.



NOTES:

1. THIS LOG OF TEST BORINGS WAS PREPARED IN ACCORDANCE WITH THE CALTRANS SOIL & ROCK LOGGING CLASSIFICATION, AND PRESENTATION MANUAL (2010).
2. 2.4" SAMPLES WERE TAKEN WITH A CALIFORNIA MODIFIED SAMPLER AND 1.4" SAMPLES WERE TAKEN WITH A STANDARD PENETRATION TEST (SPT) SAMPLER.
3. AN AUTOMATIC TRIP HAMMER SYSTEM CONSISTING OF A HAMMER WEIGHT OF 140 LBS. FALLING FROM A HEIGHT OF 30" WAS USED TO ADVANCE THE DRIVE SAMPLES.



B-7

ASPHALT CONCRETE: Approximately 6 inches thick.
FILL: Silty SAND (SM); medium dense; dark yellowish brown; moist; little SILT; fine to medium SAND.
Brown; micaceous.

Mottled brown and light gray.

ALLUVIUM: Silty SAND (SM); dense; gray; wet; little SILT; fine SAND.

Scattered shell fragments; scattered gravel.
Medium dense.

Clayey SILT (ML); soft; dark gray to black; wet; trace gravel.
Silty SAND (SM); very loose; gray; wet; fine SAND.

Fine to coarse sand; scattered shell fragments; scattered gravel.
Medium dense.

Abundant shell fragments; abundant fine gravel.
Dense; fine to coarse SAND.

Dense; trace gravel.

Medium dense; dark gray to black.

Dense; fine SAND.

B-8

ASPHALT CONCRETE: Approximately 3 inches thick.
FILL: Silty SAND (SM); loose; brown; moist; little SILT; few gravel; fine to medium SAND.
Scattered AC fragments.
Light gray; micaceous; trace gravel.
Dark brown; increase in silt and clay content; scattered gravel.

ALLUVIUM: Silty SAND (SM); dense; gray; wet; little SILT; fine SAND.

Very loose.

Medium dense; scattered shell fragments; scattered gravel.

Dark gray; micaceous.

Very dense; gray.

Medium dense; trace shell fragments.

Very dense.

Dense; scattered gravel; micaceous.

Dense; trace shell fragments.

Silty SAND (SM) with scattered interbeds of clayey SILT (ML); stiff; medium dense; dark gray to black; wet; fine SAND; highly micaceous; finely laminated.

Loose.

-- DESIGN OVERSIGHT	DRAWN BY	ALEXIS BALANE	NINYO & MOORE FIELD INVESTIGATION BY:
SIGN OFF DATE	CHECKED BY	SOUMITRA GUHA	DATE: 03/11/13, 03/13/13, AND 03/15/13

GS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

--
PROJECT ENGINEER

CU: --
EA: --

BRIDGE NO. 57C-0209
POST MILES --

CAMINO DEL MAR BRIDGE REPLACEMENT

LOG OF TEST BORINGS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES				SHEET	OF
1				4	5

BENCH MARK

VERTICAL BENCHMARK: 2" CO SD BRASS DISK IN MON WELL, PC JIMMY DURANTE BOULEVARD 600 FEET NORTHEAST OF JIMMY DURANTE WAY (SOUTH) PER RECORD OF SURVEY 14492.

DATUM: NAVD88 ELEVATION: 13.32'

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
11	SD	–	--	5	5

Soumitra Guha -/--/--
REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

SOUMITRA GUHA
No. GE 2632
Exp. 06/30/19
GEOTECHNICAL

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--

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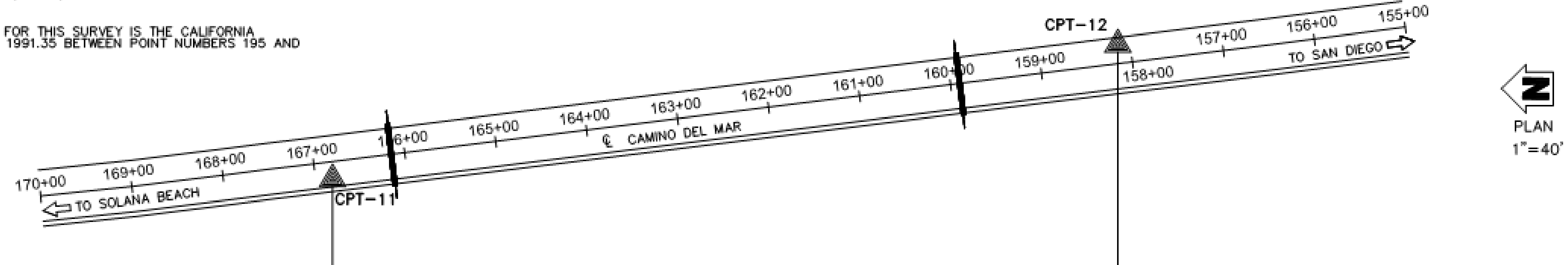
5710 Ruffin Road
San Diego, CA 92123
Phone (858) 576-1000

NOTES:

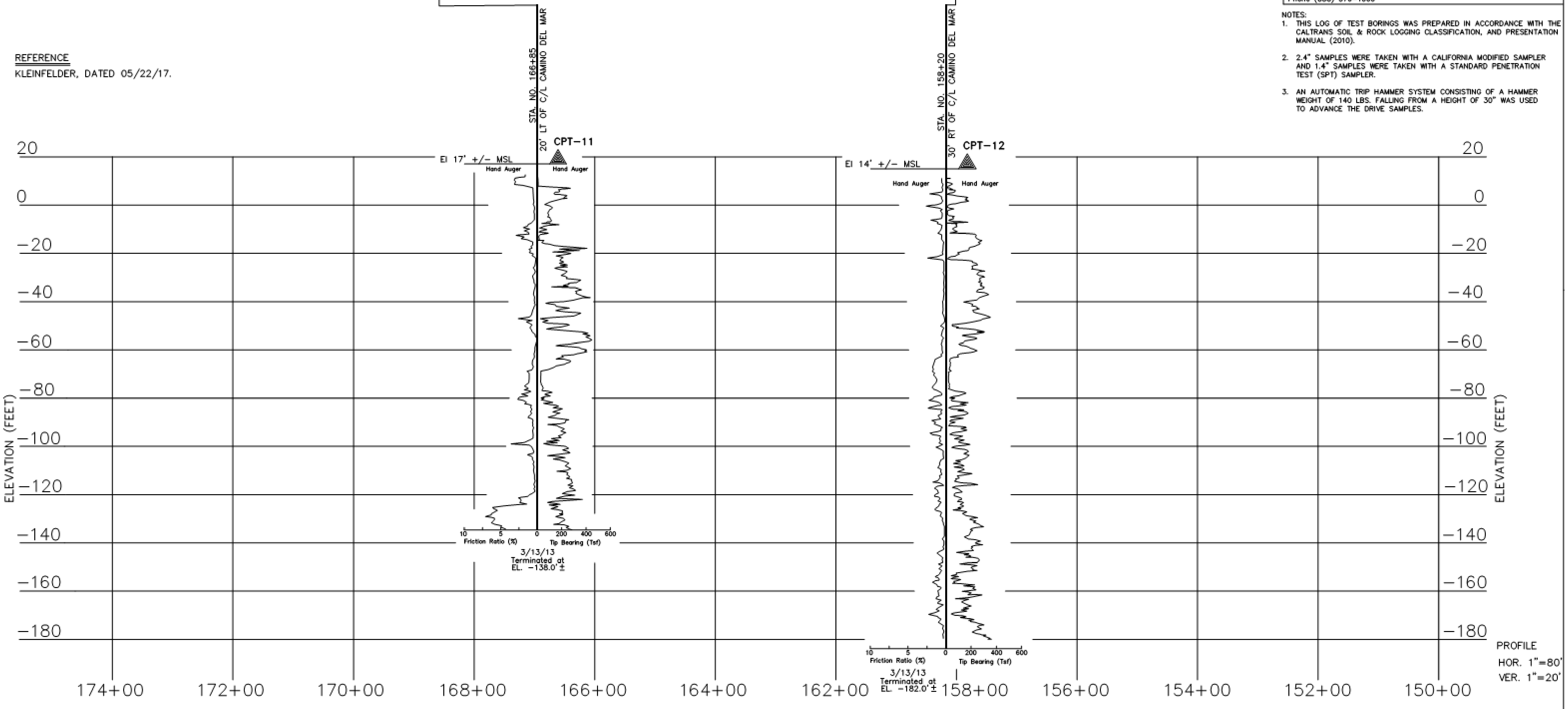
1. THIS LOG OF TEST BORINGS WAS PREPARED IN ACCORDANCE WITH THE CALTRANS SOIL & ROCK LOGGING CLASSIFICATION, AND PRESENTATION MANUAL (2010).
2. 2.4" SAMPLES WERE TAKEN WITH A CALIFORNIA MODIFIED SAMPLER AND 1.4" SAMPLES WERE TAKEN WITH A STANDARD PENETRATION TEST (SPT) SAMPLER.
3. AN AUTOMATIC TRIP HAMMER SYSTEM CONSISTING OF A HAMMER WEIGHT OF 140 LBS. FALLING FROM A HEIGHT OF 30" WAS USED TO ADVANCE THE DRIVE SAMPLES.



PLAN
1"=40'

REFERENCE

KLEINFELDER, DATED 05/22/17.



3/13/13
Terminated at
EL. -138.0' ±

3/13/13
Terminated at
EL. -182.0' ±

PROFILE
HOR. 1"=80'
VER. 1"=20'

-- DESIGN OVERSIGHT SIGN OFF DATE	DRAWN BY	ALEXIS BALANE	NINYO & MOORE FIELD INVESTIGATION BY:	
	CHECKED BY	SOUMITRA GUHA	DATE: 03/11/13, 03/13/13, AND 03/15/13	

GS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

--
PROJECT ENGINEER

BRIDGE NO.
57C-0209
POST MILES
--

CAMINO DEL MAR BRIDGE REPLACEMENT

LOG OF TEST BORINGS

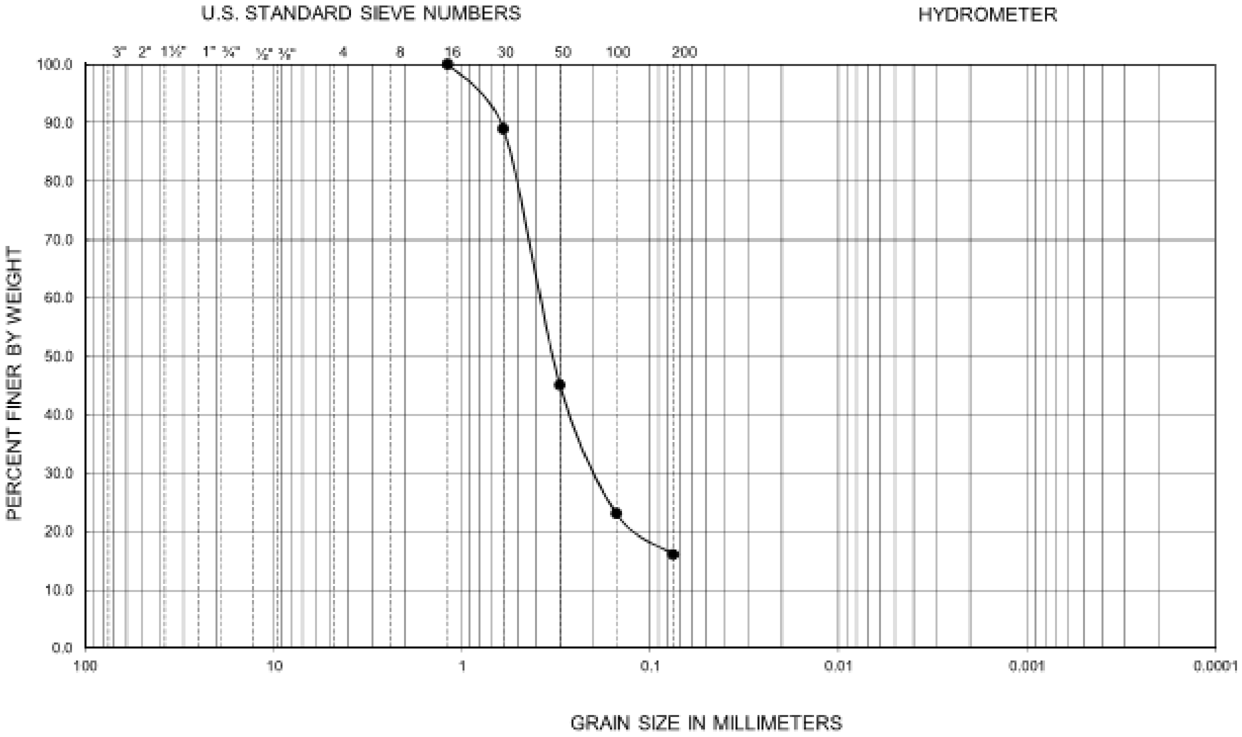
CU: --
EA: --

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES				SHEET	OF
1				5	5

E.3 Ninyo & Moore
2018 Laboratory
Test Results

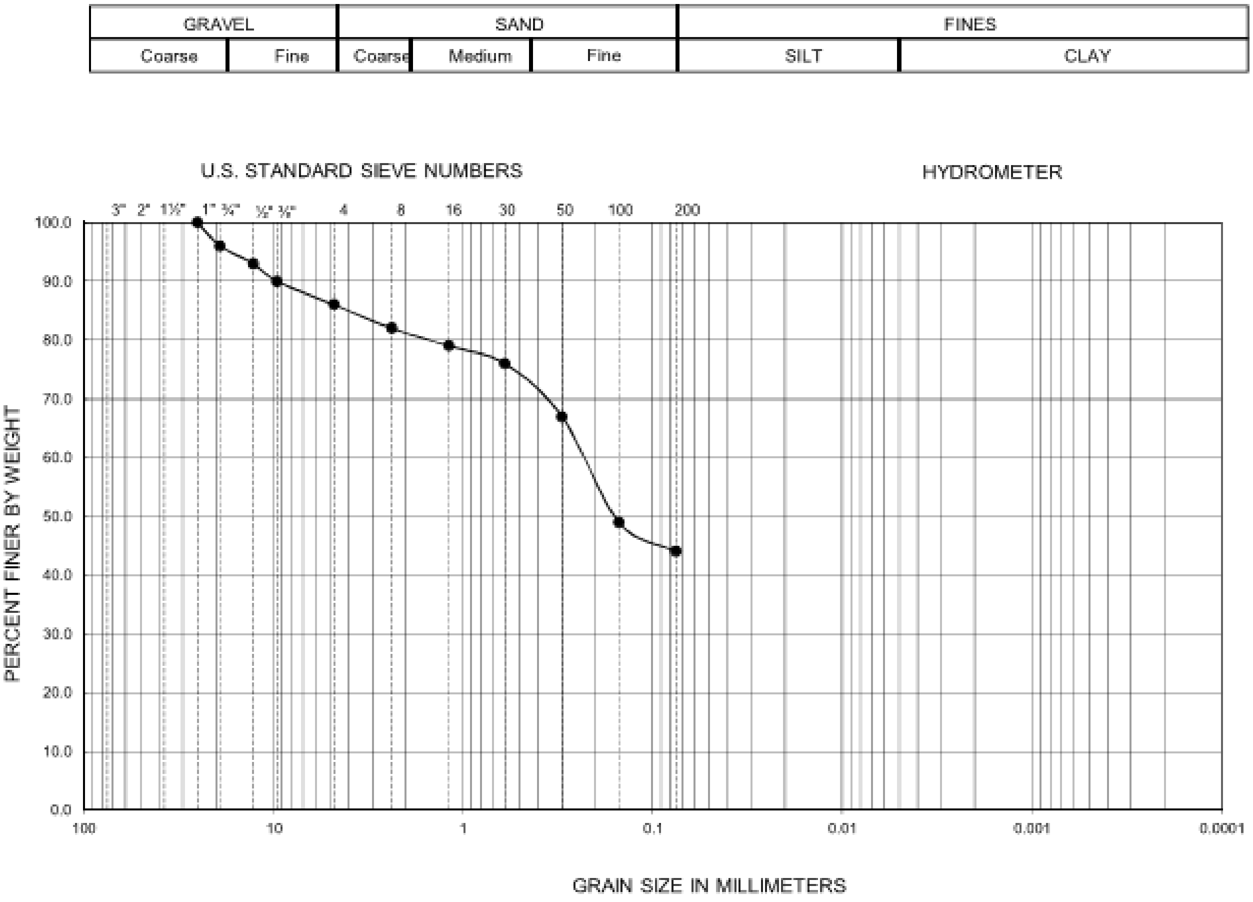
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-7	5.0-6.5	--	--	--	--	--	--	--	--	16	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	CAMINO DEL MAR BRIDGE REPLACEMENT DEL MAR, CALIFORNIA	C-1
108449001	7/18		



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-8	10.0-11.5	--	--	--	--	--	--	--	--	44	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore

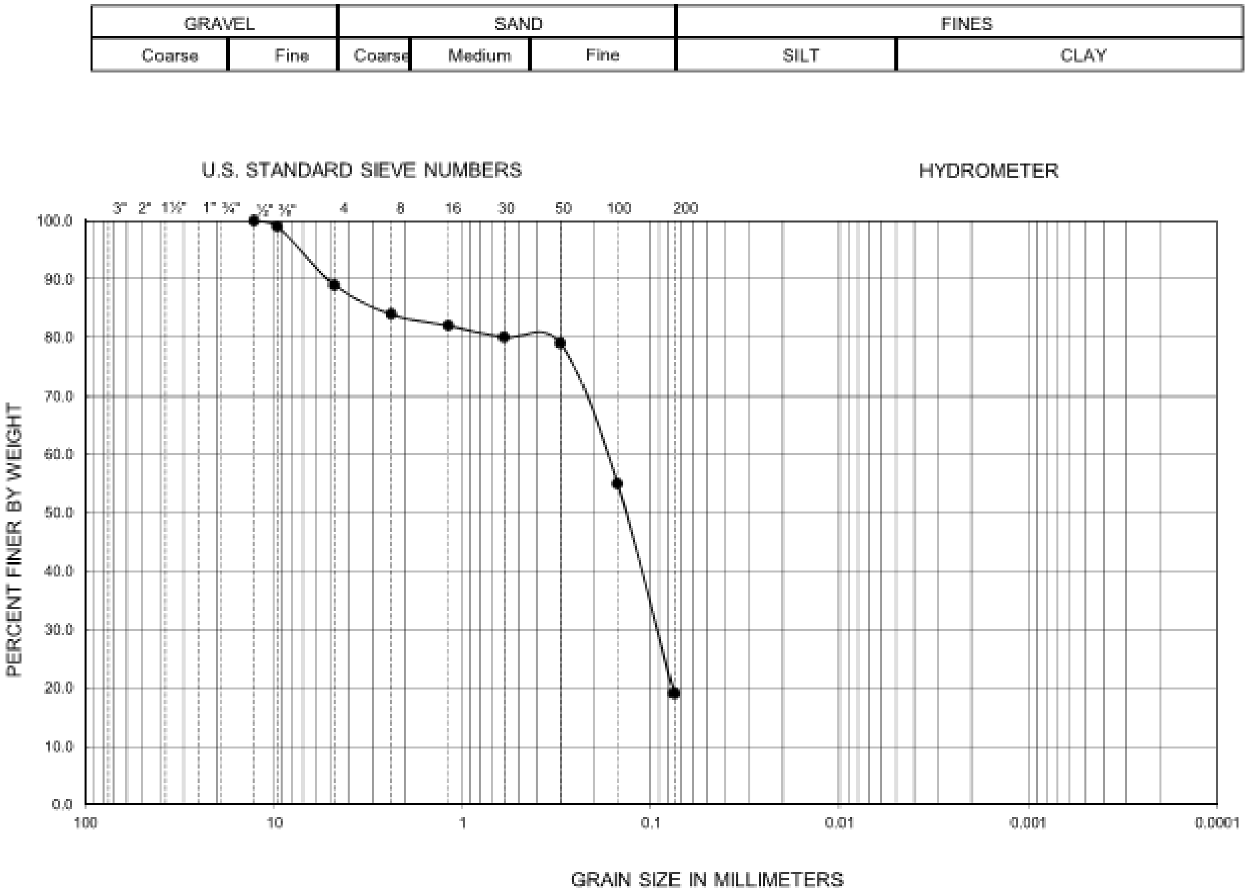
GRADATION TEST RESULTS

PROJECT NO.	DATE
108449001	7/18

CAMINO DEL MAR BRIDGE REPLACEMENT
DEL MAR, CALIFORNIA

FIGURE

C-2



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-8	17.0-18.5	--	--	--	--	--	--	--	--	19	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore

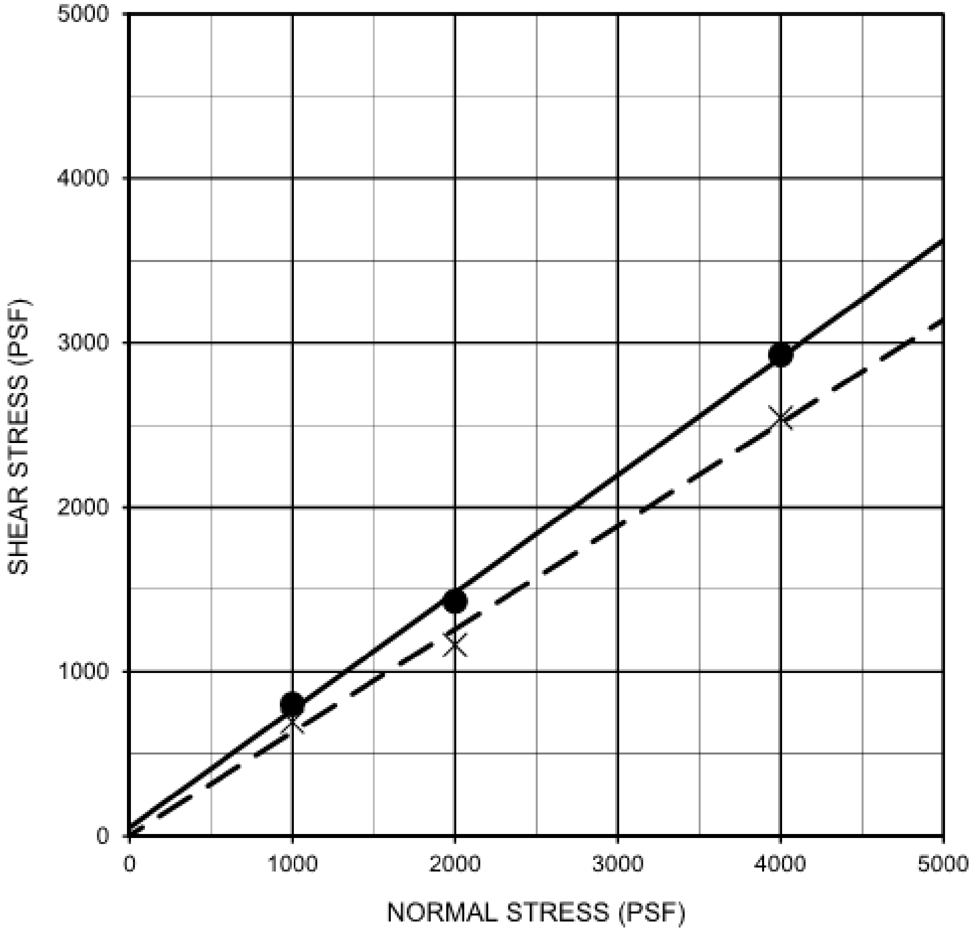
GRADATION TEST RESULTS

FIGURE

C-3

PROJECT NO.	DATE
108449001	7/18

CAMINO DEL MAR BRIDGE REPLACEMENT
DEL MAR, CALIFORNIA



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SAND	—●—	B-7	10.0-11.5	Peak	50	36	SM
Silty SAND	- - x - -	B-7	10.0-11.5	Ultimate	10	32	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore

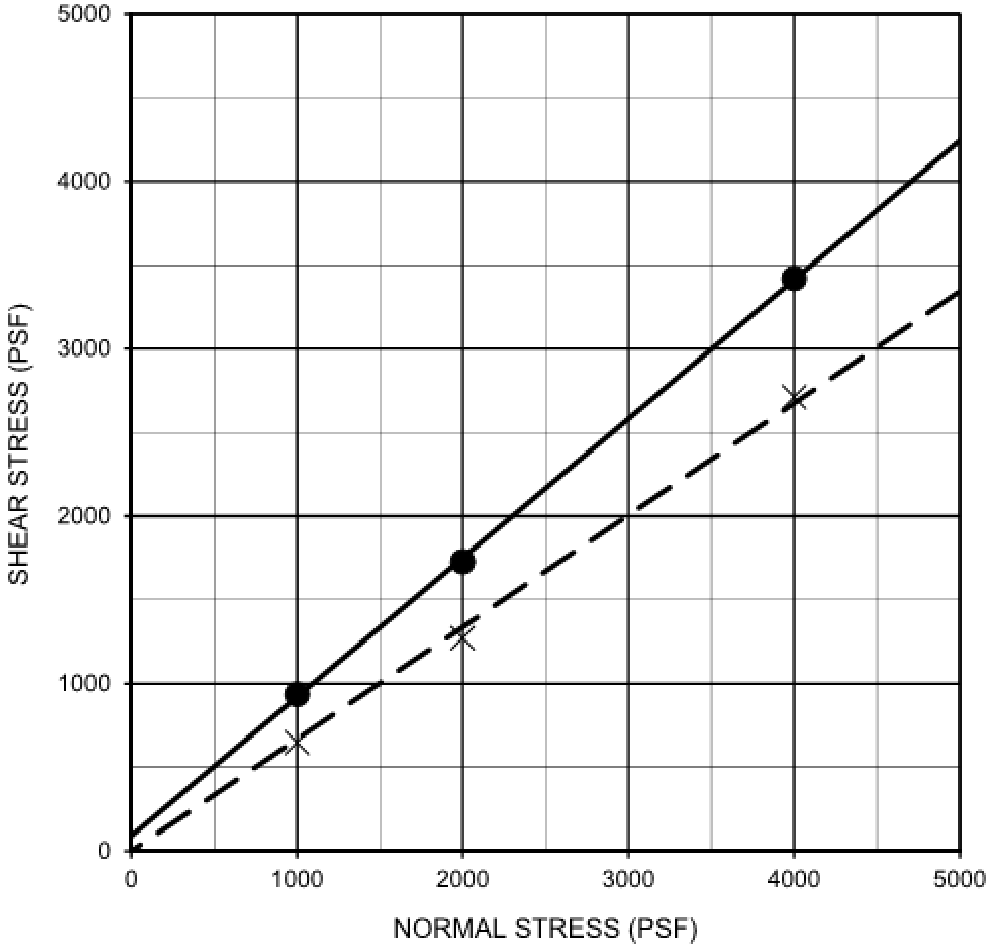
DIRECT SHEAR TEST RESULTS

PROJECT NO.	DATE
108449001	7/18

CAMINO DEL MAR BRIDGE REPLACEMENT
DEL MAR, CALIFORNIA

FIGURE

C-4



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SAND	—●—	B-8	15.0-16.5	Peak	90	40	SM
Silty SAND	– – x – –	B-8	15.0-16.5	Ultimate	0	35	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore

DIRECT SHEAR TEST RESULTS

FIGURE

C-5

PROJECT NO.	DATE
108449001	7/18

CAMINO DEL MAR BRIDGE REPLACEMENT
DEL MAR, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH[1]	RESISTIVITY[1] (Ohm-cm)	SULFATE CONTENT[2]		CHLORIDE CONTENT[3] (ppm)
				(ppm)	(%)	
B-8	5.0-6.5	8.4	10,000	40	0.004	50

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore

CORROSIVITY TEST RESULTS

PROJECT NO.	DATE
108449001	7/18

CAMINO DEL MAR BRIDGE REPLACEMENT
DEL MAR, CALIFORNIA

FIGURE

C-6

APPENDIX F
SITE RESPONSE ANALYSIS

APPENDIX F
SITE RESPONSE ANALYSIS

INTRODUCTION

This appendix presents the results of Kleinfelder's site response analysis for the Camino Del Mar Bridge Replacement project over the San Dieguito River in Del Mar, California. Based on the results of our current subsurface investigation, previous subsurface investigations by others, and preliminary engineering analyses, there is a significant liquefaction hazard at the site. Accordingly, the project site is classified as Soil Profile Type F per the 2019 Caltrans Seismic Design Criteria (SDC) V2.0 (Caltrans, 2019). Therefore, Caltrans SDC requires that a site response analysis be performed.

The purpose of this analysis is to develop a site-specific design acceleration response spectrum in accordance with the requirements of the 2019 Caltrans SDC V2.0 and the American Association of State Highway Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 8th Edition, with California Amendments (Caltrans, 2019). The site-specific design acceleration response spectrum developed from this analysis will be used for the seismic design of the proposed replacement bridge and other ancillary structures at the site.

The site response analysis relies upon data from the field and laboratory investigations completed for the project as presented in Sections 2 and 3 and in Appendices A through E of this report.

Project Understanding

As discussed in Section 1.4 of this report, the proposed project is still in the bridge type selection phase and five bridge options are still currently being considered for replacement of the existing Camino Del Mar Bridge which spans the San Dieguito River channel. These alternatives consist of three 5-span and 6-span cast-in-place box girder bridge options as well as two 6-span precast concrete girder bridge options. Large diameter Cast-In-Drilled-Hole (CIDH) type piles with permanent steel casing are recommended for support of the piers and abutments of the proposed replacement bridge. Ancillary structures proposed for the project include Caltrans Standard cantilever-type retaining walls along each side of the northern and southern bridge approaches. These retaining walls will support new approach fill in order to raise grades for to accommodate the design storm water level.

Based on discussions with the project structural engineer, we understand that the longitudinal and transverse fundamental periods of the proposed bridge alternatives range from approximately 0.5 to 1.4 seconds and 0.7 to 1.3 seconds, respectively, for the various alternatives.

At this time, it is our understanding that ground motion time histories will not be needed for structural design.

Project Location

We have used the approximate coordinates near the center of the bridge as the control point for the seismic hazard analysis. The coordinates of the approximate center of the bridge structure are:

Latitude:	32.9750° N	Longitude:	117.2690° W

Material properties and other parameters used were selected to be representative of the response of the site as a whole to ground motions based on the preliminary field explorations performed at the project site.

Approach

This site response analysis was performed in general accordance with the requirements of the 2019 Caltrans SDC V2.0 and the AASHTO LRFD Bridge Design Specifications (BDS), 8th Edition, with California Amendments. The scope of this analysis includes the following:

- Review of subsurface conditions impacting the seismic hazards at the site including geology and subsurface stratigraphy and seismic hazards at the site;
- Development of a horizontal response spectrum at the base of the soil column which serves as the target spectrum in selection of ground motions to be used for the site response analysis. The target spectrum was developed for the 975-year return period ground motion level using an appropriate V_{S30} value in accordance with Caltrans SDC;
- Deaggregation analyses of the hazard to estimate the controlling seismic source(s) associated with the period ranges of interest for the target spectrum;
- Selection and modification of seven acceleration time histories per AASHTO LRFD BDS based on the target spectral shape, earthquake magnitude, distance, and frequency content from historical earthquake records;
- Spectral matching of the selected time histories to the developed target spectrum;

- Development of soil properties to be used in the site response analysis;
- Site response analysis using appropriate equivalent linear and nonlinear models in accordance with Caltrans guidelines and the AASHTO LRFD BDS; and
- Development of the site-specific design acceleration response spectrum in accordance with the requirements of Caltrans guidelines and the AASHTO LRFD BDS.

The scope of this analysis is subject to the limitations provided in Section 6 of the main report.

SUBSURFACE CHARACTERIZATION

Subsurface characterization was developed to support the site response analysis and is based on the results of the current and previous subsurface investigations as discussed in Section 3 of the main report.

Subsurface Geology and Stratigraphy

The project site is generally underlain by an upper layer of Recent Alluvial Deposits (Qa) overlying successive strata of Young Alluvial Deposits (Qya), Young Estuarine Deposits (Qyes), Old Alluvial Deposits (Qoa), and the Del Mar Formation (Td). Further details regarding the characteristics and conditions of each of these geologic units are provided in Section 3 of the main report.

Based on the results of the geotechnical investigations performed at the site, a generalized best estimate profile of material properties was developed for use in the site response analysis and is presented below in Table F-1. These material properties were developed based on in-situ testing which included performing a Seismic Cone Penetrometer Test (SCPT), Cone Penetrometer Testing (CPTs), exploratory borings, and laboratory testing as well as our experience with similar materials in the project vicinity.

Table F-1

Material Properties for Site Response Analysis

Layer No.	Geologic Unit	Dominant Soil Type	Layer Thickness (ft)	Unit Weight (pcf)	Friction Angle (deg)	At-Rest Earth Pressure, Ko	Plasticity Index, PI
1	Qa	Sand (Loose)[1]	12	120	28	0.53	0
2		Clay (Soft)	7	110	18	0.69	40
3		Sand (Loose)[1]	16	120	28	0.53	0

Table F-1 (Continued)
Material Properties for Site Response Analysis

Layer No.	Geologic Unit	Dominant Soil Type	Layer Thickness (ft)	Unit Weight (pcf)	Friction Angle (deg)	At-Rest Earth Pressure, Ko	Plasticity Index, PI
4	Qya	Sand (Med. Dense)[1]	30	125	32	0.47	1
5	Qyes	Clay (Stiff)	16	115	22	0.63	30
6	Qoa	Sand (Med. Dense to Dense)	55	125	34	0.44	1
7	Qoa/Td	Gravelly Sand (Very Dense) and Claystone / Sandstone (Very Dense / Very Stiff)	Half Space	135	-	-	-

Notes:
[1] Potentially liquefiable layers based on results of field investigation and liquefaction triggering analyses as presented in Section 4.1.2 of the main report.
[2] Material parameters and layering selected to represent best estimate for seismic site response and may not be appropriate for other geotechnical evaluations.

Site Class

Due to the potential of extensive liquefaction in the recent and young alluvial deposits at the site as discussed in Section 4.1.2 of this report, the site is classified as a Soil Profile Type F site and site response analysis is required per the SDC.

However, for the purpose of comparing the design spectrum with general response spectrum per AASHTO, site class was evaluated in accordance with the requirements of the Caltrans SDC V2.0 and the AASHTO LRFD BDS, 8th Edition, with California Amendments (Caltrans, 2019). The average shear wave velocity in the upper 100 feet (e.g. V_{S30}) was evaluated using data from the SCPT performed at the CPT-20-003 location. The results of the SCPT are provided on Figure F-1 and further details are provided in Appendix B of this report.

Using the SCPT data, the average shear wave velocity in the upper 100 feet was estimated to be of 711 ft/s (216 m/s), which is consistent with a Soil Profile Type D site classification per Caltrans SDC.

DEVELOPMENT OF BASE GROUND MOTIONS

Development of base ground motions include developing target response spectrum at the base of the soil column and then selecting and developing spectrally matched time histories to be used for performing site response analysis. Details of the target spectrum and time history development are discussed in the subsequent sections.

Target Spectrum Development

The target acceleration response spectrum at the base of the soil column was obtained from the Caltrans ARS Online V3.0.1 tool. The Caltrans ARS Online tool provides the probabilistic design response spectrum based on the United States Geological Survey (USGS) 2014 National Seismic Hazard Maps for a 975-year return period (Petersen et al., 2014). Inputs for the ARS Online tool include the site's coordinates, in which we used the site's coordinates for the approximate center of the bridge, as well as the V_{S30} value. For the target spectrum, a V_{S30} value consistent with soil conditions at the base of the soil column was used. In general, where bedrock is shallow, base of the soil column is located at the bedrock. However, for this site, bedrock is relatively deep, therefore, we have selected our base at a certain depth beyond which the shear wave velocity is quite consistent and reflective of competent materials. Based on this, for our site response analysis, the base of the soil column is located at a depth of approximately 136 feet from the ground surface within the river channel, or at an approximate elevation of -134 ft NAVD88. Based on shear wave velocity values obtained at that elevation in the SCPT performed at the site, a V_{S30} value of 1,000 ft/s (315 m/s) was used for development of the target spectrum.

The target response spectrum for a 975-year return period, using a V_{S30} value of 1,000 ft/s, obtained from the Caltrans ARS Online tool is provided in Table F-2 and Figure F-2. This target spectrum was adjusted for near fault amplification based on the proximity of the site to the controlling Rose Canyon fault in accordance with Caltrans SDC requirements.

Table F-2

Caltrans ARS Online Target Response Spectrum

Period	Near Fault Amplification Factor	Probabilistic Spectral Acceleration (g)
0.01 (PGA)	1	0.43
0.1	1	0.75
0.2	1	1.01
0.3	1	1.06
0.5	1	0.92
0.75	1.1	0.78

Table F-2 (Continued)
Caltrans ARS Online Target Response Spectrum

Period	Near Fault Amplification Factor	Probabilistic Spectral Acceleration (g)
1.0	1.2	0.66
2.0	1.2	0.32
3.0	1.2	0.2
4.0	1.2	0.14
5.0	1.2	0.1

Time History Selection and Spectral Matching

Using the target response spectrum provided in Figure F-2 and Table F-2, a suite of seven time histories were selected from the PEER Strong Ground Motion Database (PEER, 2014) and spectrally matched for use in the site response analysis in accordance with AASHTO and Caltrans. The time histories were selected based on several criteria including near-fault pulse motions, scaling factor, site-to-source distance, magnitude, V_{S30}, arias intensity, duration, style of faulting, shape of response spectrum, etc. These time histories were selected and modified for use in site response analysis only and may not be appropriate for other applications.

Due to the site's close proximity to the Rose Canyon fault, both pulse and non-pulse motions were considered during selection of time histories as required by AASHTO guidelines. Based on the methodology presented in Hayden et al. (2014), the distance from the site to the Rose Canyon fault, and the epsilon value of the spectral acceleration at a period of 1 second, we estimated that the proportion of pulse motions to be selected for the site response analysis is three to four pulse motions out of seven, with the remainder being non-pulse motions.

Consideration was also given to the controlling earthquake sources over various period ranges considering the results of the USGS deaggregation of the probabilistic seismic hazard. Based on the deaggregation results, the shorter period (higher frequency) range of the target spectrum is controlled primarily by events associated with the near (less than 15 km away) to mid-field range such as the nearby Rose Canyon fault at approximately 2.2 miles (3.6 km) west of the site as well as the Oceanside fault and Coronado Bank fault at approximately 11 miles (17.7 km) and 16.5 miles (26.5 km) west of the site, respectively. Longer period ranges were also controlled by these near to mid-field events but also had contributions from farther events such as those associated with the Elsinore fault at 29.5 miles (47.4 km) east of the site and the San Jacinto fault at 54 miles (87 km) east of the site. The style of faulting associated with these controlling sources include strike-slip and reverse/oblique faulting. Based on these results, we evaluated a suite of ground motions considering primarily near to mid-field events for strike-slip and reverse/oblique sources in order to understand the range of responses likely to occur.

Other selection parameters included magnitude and V_{S30}, in which time histories relatively close to the probabilistic mean magnitude of 6.65 and V_{S30} value of 1,000 ft/s for the target spectrum were selected. Considerations for arias intensity and duration of the ground motions used the methodologies of Travasarou et al. (2003) and Bommer et al. (2009) for selection of ground motions in relation to these parameters.

Based on these criteria, a suite of seven time histories was selected from the PEER database that had a spectral shape after scaling (scaling factors less than 3) generally in good agreement with the target response spectrum. These selected ground motion time histories and their associated characteristics are provided in Table F-3.

Table F-3

Selected Time Histories from PEER Database

Record No.	Event Name	Year	M_w	Distance, R_{Rup} (km)	V_{S30} (m/s)	Faulting Mechanism	D_{5-95} (sec)	I_A (m/s)	LUF (Hz)	Pulse Period	Scaling Factor
RSN 725	Superstition Hills-02	1987	6.54	11.16	316.64	SS	13.7	2.1	0.1625	-	1.6
RSN 767	Loma Prieta	1989	6.93	12.82	349.85	RO	11.4	2.1	0.125	2.64	1.4
RSN 1045	Northridge-01	1994	6.69	5.48	285.93	R	8.8	1.5	0.125	2.98	1.2
RSN 1119	Kobe, Japan	1995	6.9	0.27	312	SS	4.6	3.9	0.1625	1.81	0.8
RSN 1605	Duzce, Turkey	1999	7.14	6.58	281.86	SS	11.1	2.9	0.1	5.94*	0.9
RSN 3756	Landers	1992	7.28	40.67	368.2	SS	32.9	1	0.05	-	2.9
RSN 6923	Darfield, NZ	2010	7	30.53	255	SS	20.1	1.6	0.2	-	1.6

Notes: Definitions: M_w – Moment Magnitude; R - Reverse fault; RO – Reverse Oblique fault; SS – Strike-slip fault; D_{5-95} – Significant Duration; I_A – Arias Intensity; LUF – Lowest Usable Frequency

*Pulse motion as defined by Shahi and Baker (2014). This time history is not identified as a pulse motion in the PEER database.

The selected ground motions from the PEER database were then modified by performing spectral matching using the RSPMatch program developed by Atik and Abrahamson (2010) as implemented in the computer program EZ-FRISK^{TM} (Risk Engineering, 2018) which generally implements the spectral matching algorithm proposed by Lilhanand and Tseng (1987, 1988) with an updated wavelet adjustment to preserve the non-stationary characteristics of the ground motions. Spectral matching was completed such that the resulting spectrum was generally in good agreement with the target spectrum particularly over the period range of interest. The spectrally matched ground motions were compared with the PEER database original ground motions to ensure that the matching process retained the non-stationary characteristics of the record.

Figures presenting the selected matched time histories used as the “outcrop” ground motions in the site response analysis, along with the original time histories as obtained from the PEER database, are provided on Figures F-3 through F-9. The matched spectra and average of the matched spectra compared to the target spectrum is shown on Figure F-10.

SITE RESPONSE ANALYSIS

Site response analysis was completed for the site in accordance with the 2019 Caltrans SDC V2.0 and the AASHTO LRFD BDS, 8th Edition, with California Amendments. Evaluations were completed using the selected, matched time histories as the outcrop motions in conjunction with one-dimensional total stress nonlinear (without porewater pressure generation) and equivalent linear response history analyses using the computer program DEEPSOIL v7.0 (Hashash et al., 2020). Results of the site response analysis were used to develop the site-specific design acceleration response spectrum for the project. Details of the site response analysis methodology and results are presented in the subsequent sections.

Representative Soil Profile and Analysis Approach

For the site response analysis, the material properties and generalized soil layering discussed previously were adopted with soil parameters assigned as shown in Table F-4. The various soil layers were fit to the appropriate modulus reduction and damping curves as shown in Table F-4. In fitting the modulus reduction and damping curves, the general quadratic / hyperbolic (GQ/H) strength controlled constitutive model of Groholski et al. (2015) was used as this model is able to account for the small strain behavior and shear strength of the soil. The soil layers were subdivided into sub-layers to allow for higher maximum frequencies to pass through the layers. The number and thickness of the sub-layers are also provided in Table F-4. It should be noted that generation of excess pore pressures for the potentially liquefiable soils at the site were not considered in the site response analysis in accordance with guidance provided in communications with Caltrans. In addition, shear strengths in potentially liquefiable materials were not reduced for site response analysis.

Table F-4

GQ/H Model Soil Parameters for Site Response Analysis

Layer No.	Geologic Unit	Dominant Soil Type	Modulus Reduction / Damping[1]	Layer Thickness (ft)	No. of Sub Layers (Thickness)	Maximum Freq. Passing (Hz)	Vs (fps)
1	Qa	Sand (Loose)[1]	Darendeli (2001)	12	6 (2 ft)	81.3	650
2		Clay (Soft)	Darendeli (2001)	7	2 (3.5 ft)	42.9	600
3		Sand (Loose)[1]	Darendeli (2001)	16	8 (2 ft)	81.3	650
4	Qya	Sand (Med. Dense)[1]	Darendeli (2001)	30	10 (3 ft)	62.5	750
5	Qyes	Clay (Stiff)	Darendeli (2001)	16	4 (4 ft)	43.8	700
6	Qoa	Sand (Med. Dense)	Darendeli (2001)	55	11 (5 ft)	42.5	850
7	Qoa/Td	Gravelly Sand (Very Dense) and Claystone / Sandstone (Very Dense / Very Stiff)	Half Space				1,000

Notes:

[1] Potentially liquefiable layers based on results of field investigation and liquefaction triggering analyses as presented in Section 4.1.2 of the main report.

[2] Modulus Reduction and Damping curves used in fitting of model parameters. Shear strengths for fitting routine taken using cohesion and friction angles shown previously.

The GQ/H model uses shear strength which varies with depth to model large-strain behavior of the soil. The shear strength used in the GQ/H model is the judgement-based shear strength developed at 0.1 percent shear strain for a linear elastic material with 80 percent of the maximum shear modulus derived from the shear wave velocity of the soil layer as defined in Hashash et al. (2020). Viscous small strain damping used a frequency independent formulation implemented in DEEPSOIL as recommended by Hashash et al. (2020). The selected ground motions were modeled as “outcrop” motions at the base of the soil profile.

Evaluation and Results

The profile response with depth and the response spectra at the modeled ground surface were obtained from the site response analysis for each of the selected ground motions as shown on Figures F-11 through F-19 and the averages of the non-linear and equivalent linear responses are provided on Figure F-20. In general, the equivalent linear site response analysis resulted in deamplification of the “outcrop” ground motions at the surface at short periods (generally less than periods of approximately 0.4s to 0.6s) and amplification at the surface at longer periods. The non-linear site response analysis also resulted in deamplification at shorter periods with amplification of the “outcrop” ground motions at the ground surface at periods greater than about 0.7s to 0.9s. When comparing the average equivalent linear and non-linear results of the selected ground motions to the target spectrum, deamplification was observed at periods up to approximately 0.4s and 0.9s, respectively, with amplification at periods thereafter (up to 5 seconds for the site response analysis).

The maximum spectral acceleration values of the non-linear and equivalent linear site response results were used to develop an enveloping spectrum in order to evaluate the amplification of the target spectrum expected at the site. As shown on Figure F-21, the average equivalent linear spectrum controls for periods up to approximately 2 seconds and the average non-linear spectrum controls thereafter. This enveloping spectrum was compared to the average of the “outcrop” ground motions to develop amplification factors (i.e. ratio of enveloping spectral accelerations to “outcrop” spectral accelerations). The amplification factors are also provided on Figure F-21.

Using the amplification factors shown in Figure F-21, the recommended design acceleration response spectrum was developed by multiplying the base target spectrum by the amplification factors at each period consistent with the requirements of AASHTO LRFD BDS. This amplified spectrum was then compared with two-thirds of the general procedure spectrum developed in accordance with AASHTO LRFD BDS as the final recommended design response spectrum should not be less than the two-thirds of the general procedure spectrum. The general procedure response spectrum was developed using the values of peak ground acceleration (PGA), the short-period spectral acceleration coefficient (S_s), and the long-period spectral acceleration coefficient (S_1) obtained from the USGS National Seismic Hazard Maps for a 975-year return period as presented in Section 3.10.2.1 of the AASHTO LRFD BDS. These spectral accelerations were site corrected using the Site Class D site factors referenced from Section 3.10.3.2 of the AASHTO LRFD BDS and the site-corrected spectral accelerations were used to develop the general procedure spectrum is accordance with Section 3.10.4.1 of the AASHTO LRFD BDS.

As shown on Figure F-22, the amplified target spectrum controls for all periods in our analysis except for periods between approximately 0.03 and 0.3 seconds in which the two-thirds of the general procedure spectrum controls. Therefore, the final recommended design acceleration response spectrum is an enveloping spectrum of the amplified target spectrum and the two-thirds of the general procedure spectrum. This recommended design acceleration response spectrum and the associated spectral displacement values are provided in Table F-5 and shown on Figure F-23.

Table F-5
Site-Specific Horizontal 5% Damped Recommended Design Spectral Acceleration and Spectral Displacement Values

Period, T (seconds)	Design Acceleration Spectrum, S_a (g)	Design Displacement Spectrum, S_D (in)
0.010	0.379	0.00
0.020	0.394	0.00
0.030	0.409	0.00
0.050	0.482	0.01
0.075	0.574	0.03
0.1	0.665	0.07
0.113	0.714	0.09
0.2	0.714	0.28
0.28	0.714	0.55
0.3	0.766	0.67
0.5	0.964	2.36
0.75	0.888	4.89
1.0	0.957	9.37
2.0	0.502	19.67
3.0	0.282	24.85
4.0	0.172	26.99
5.0	0.118	28.86

LIMITATIONS

The values in this appendix were developed using site response analysis as required by Caltrans SDC V2.0 and supersede any seismic design parameters provided previously. The results are subject to the limitations in Section 6 of this Preliminary Foundation Report and rely upon the results of the field investigation as presented in this report.

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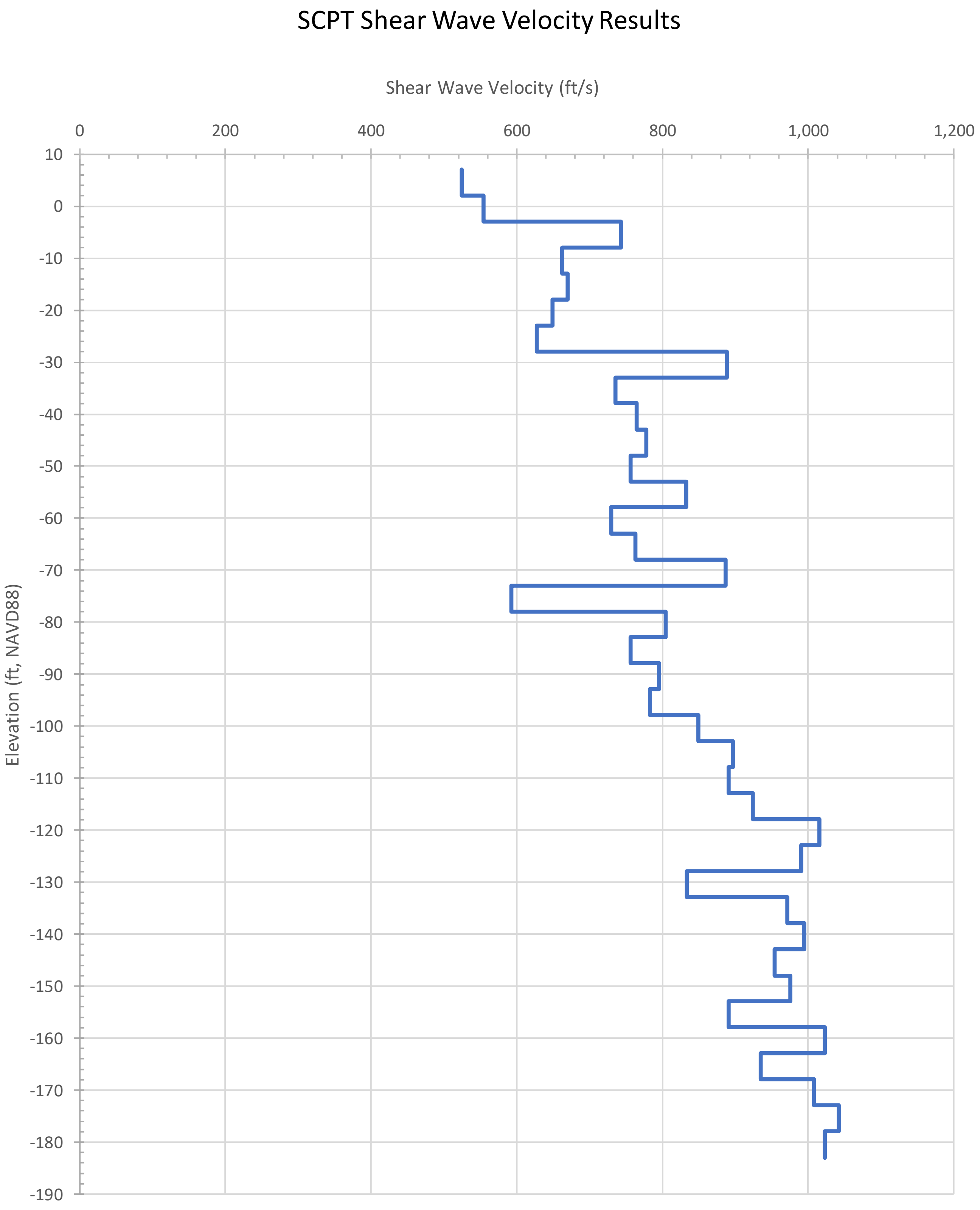

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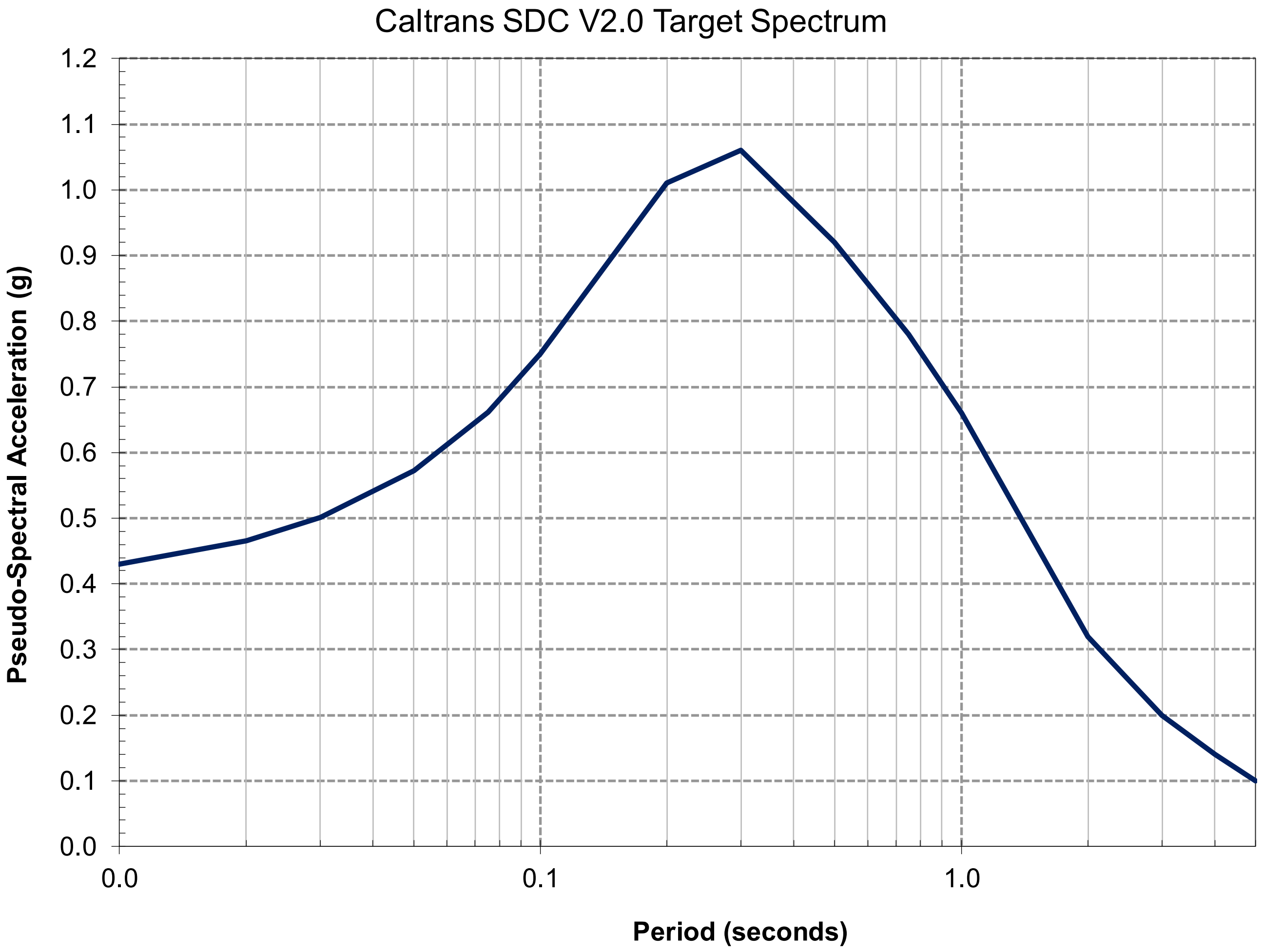
SCPT SHEAR WAVE VELOCITY RESULTS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-1



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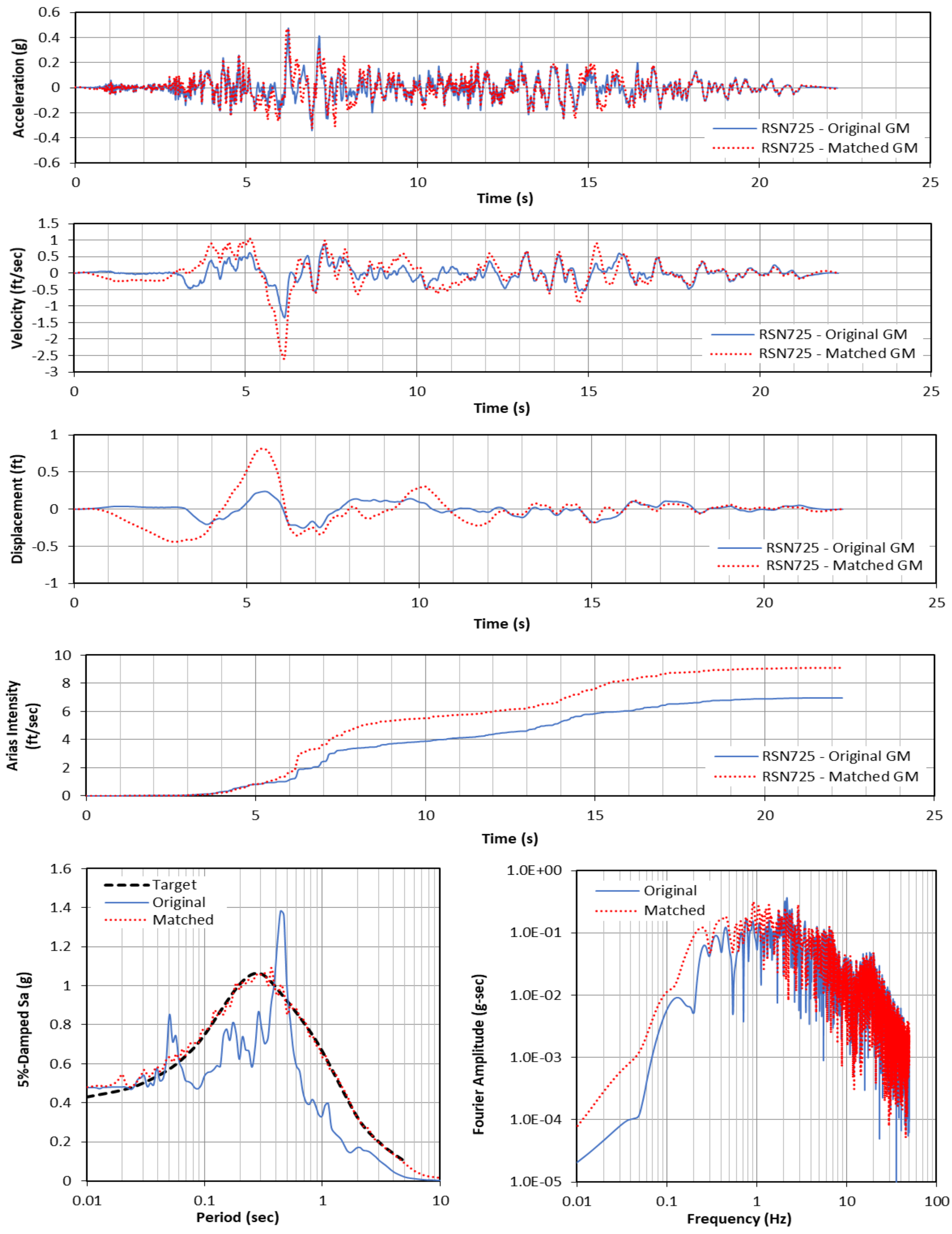
TARGET SPECTRUM FOR SITE RESPONSE ANALYSIS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-2



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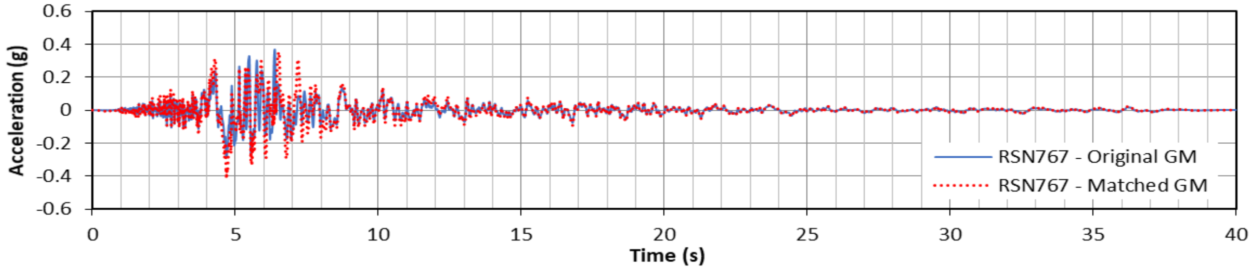
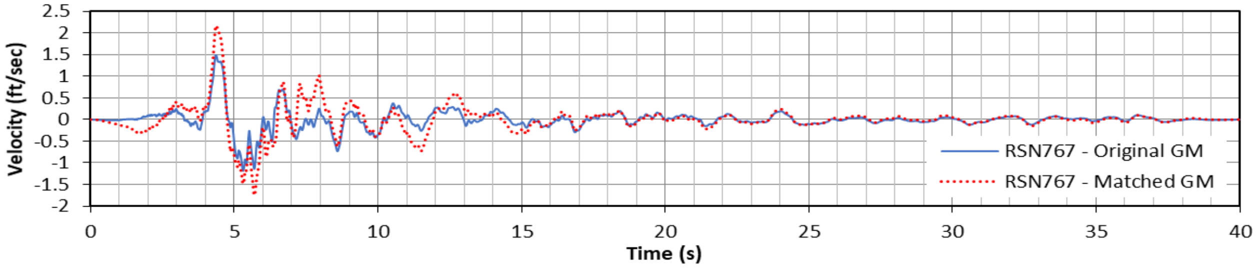
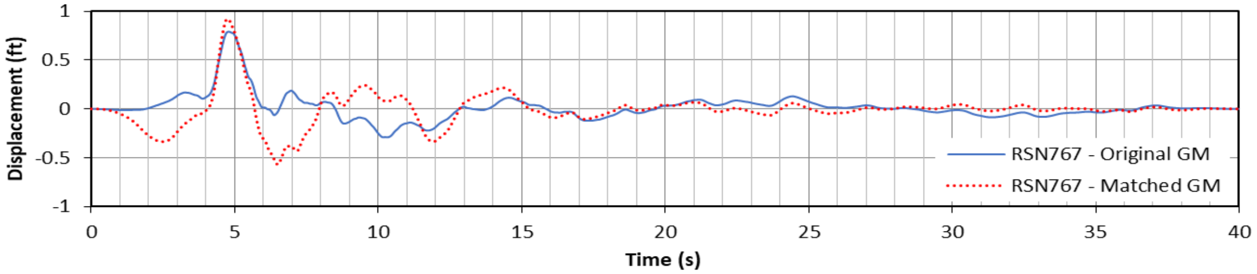
SPECTRAL MATCHING RESULTS
RSN No. 725 (270°)
SUPERSTITION HILLS 1987

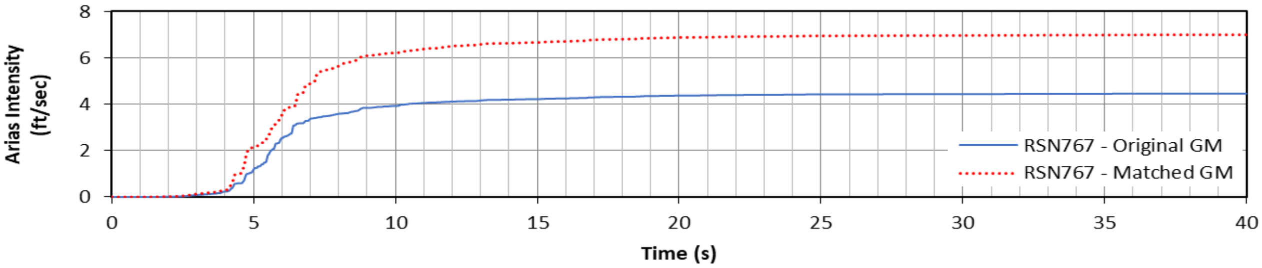
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

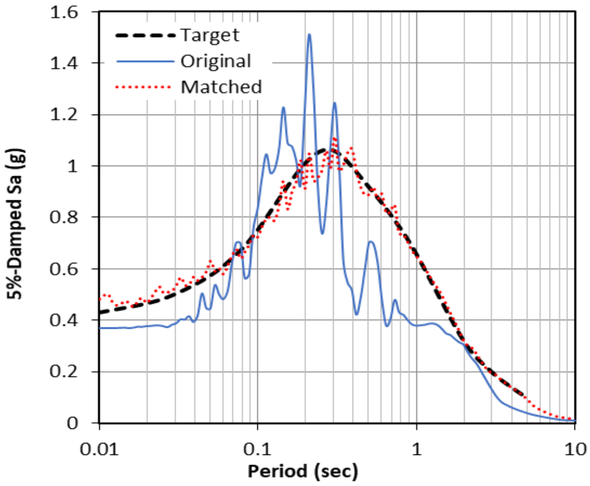
FIGURE
F-3

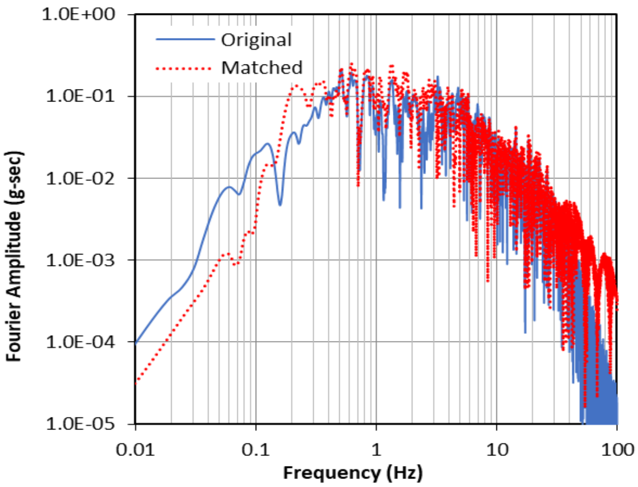












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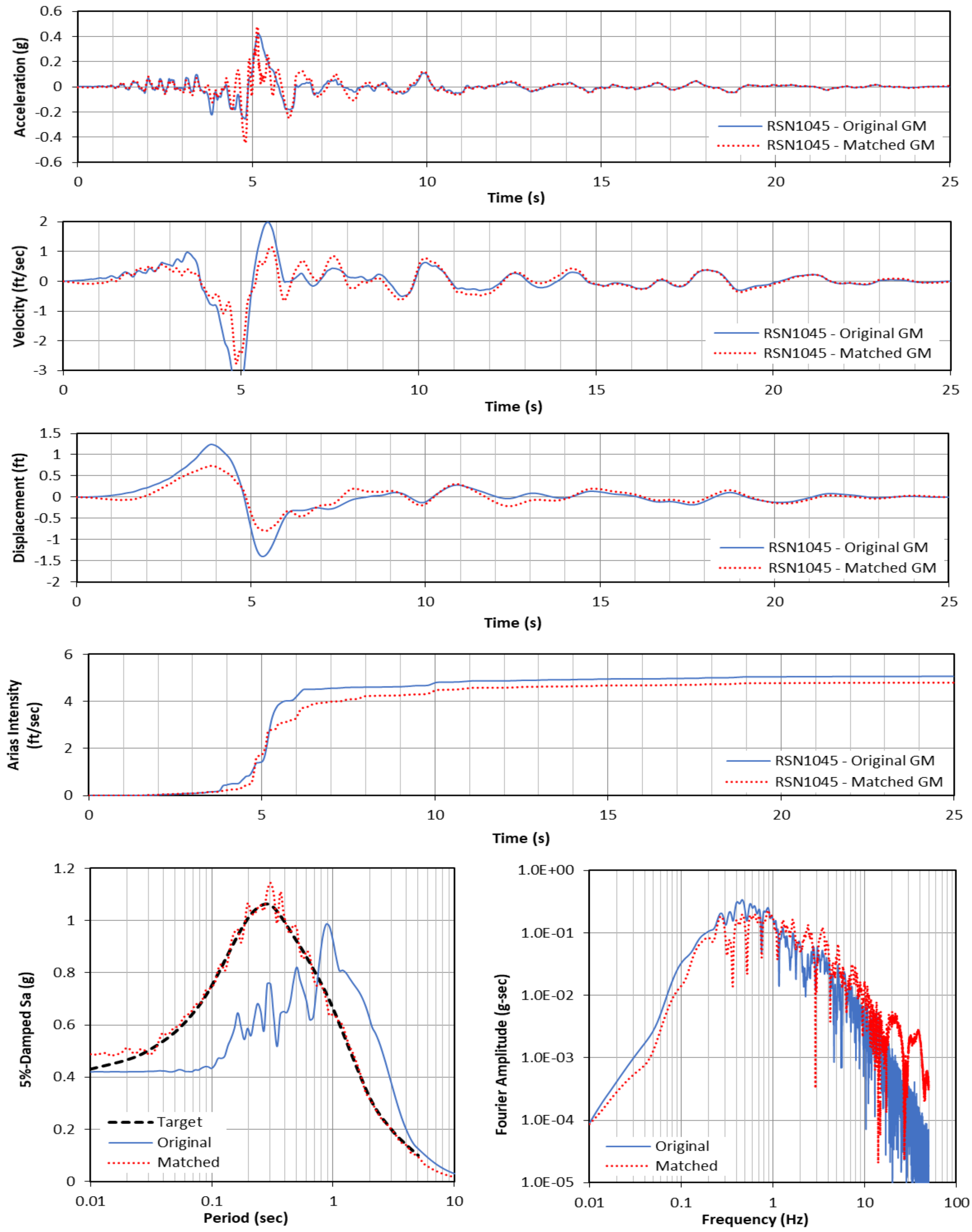
SPECTRAL MATCHING RESULTS
RSN No. 767 (90°)
LOMA PRIETA 1989

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-4



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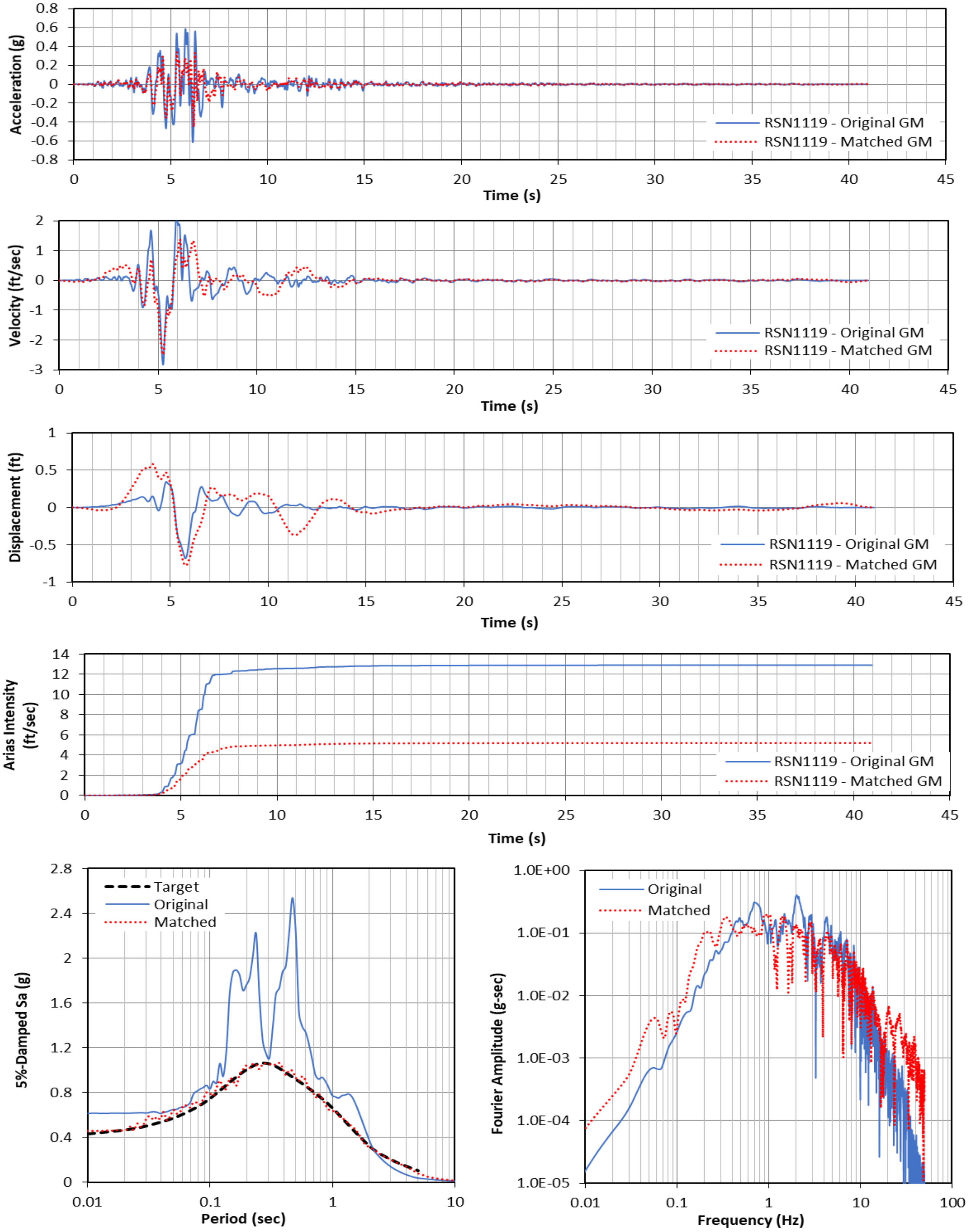
SPECTRAL MATCHING RESULTS
RSN No. 1045 (46º)
NORTHRIDGE 1994

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-5



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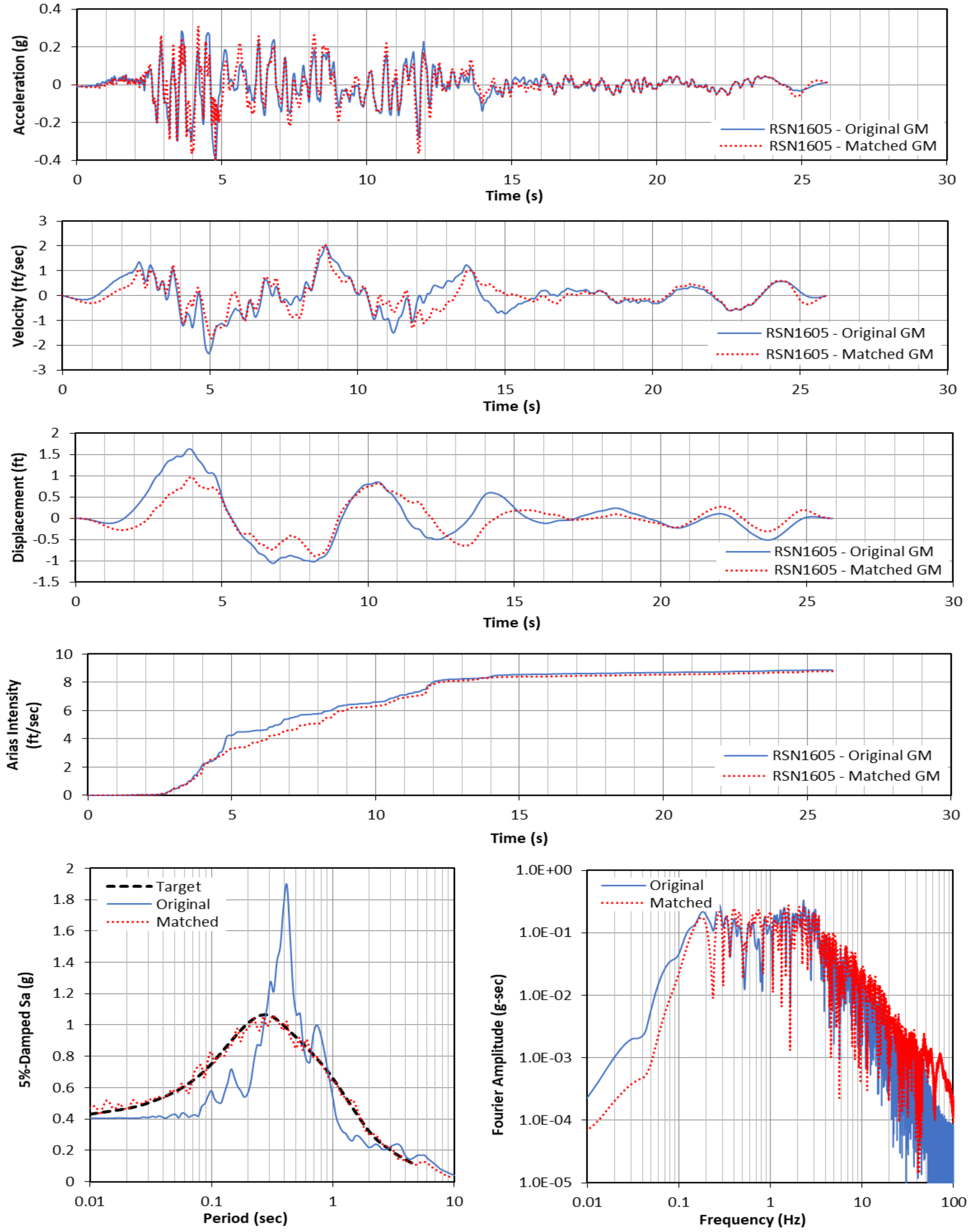
SPECTRAL MATCHING RESULTS
RSN No. 1119 (90º)
KOBE, JAPAN 1995

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-6



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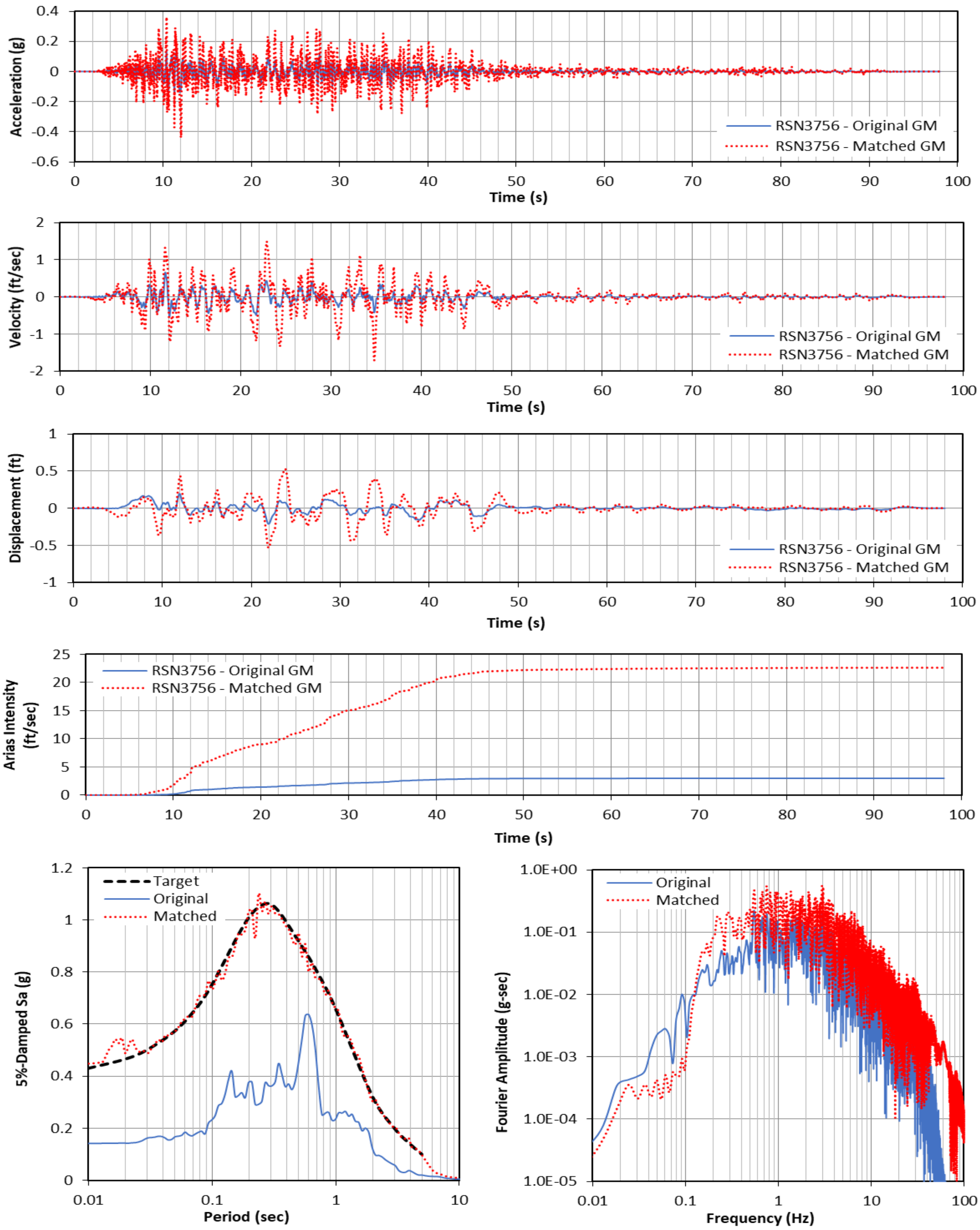
SPECTRAL MATCHING RESULTS
RSN No. 1605 (180°)
DUZCE, TURKEY 1999

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-7



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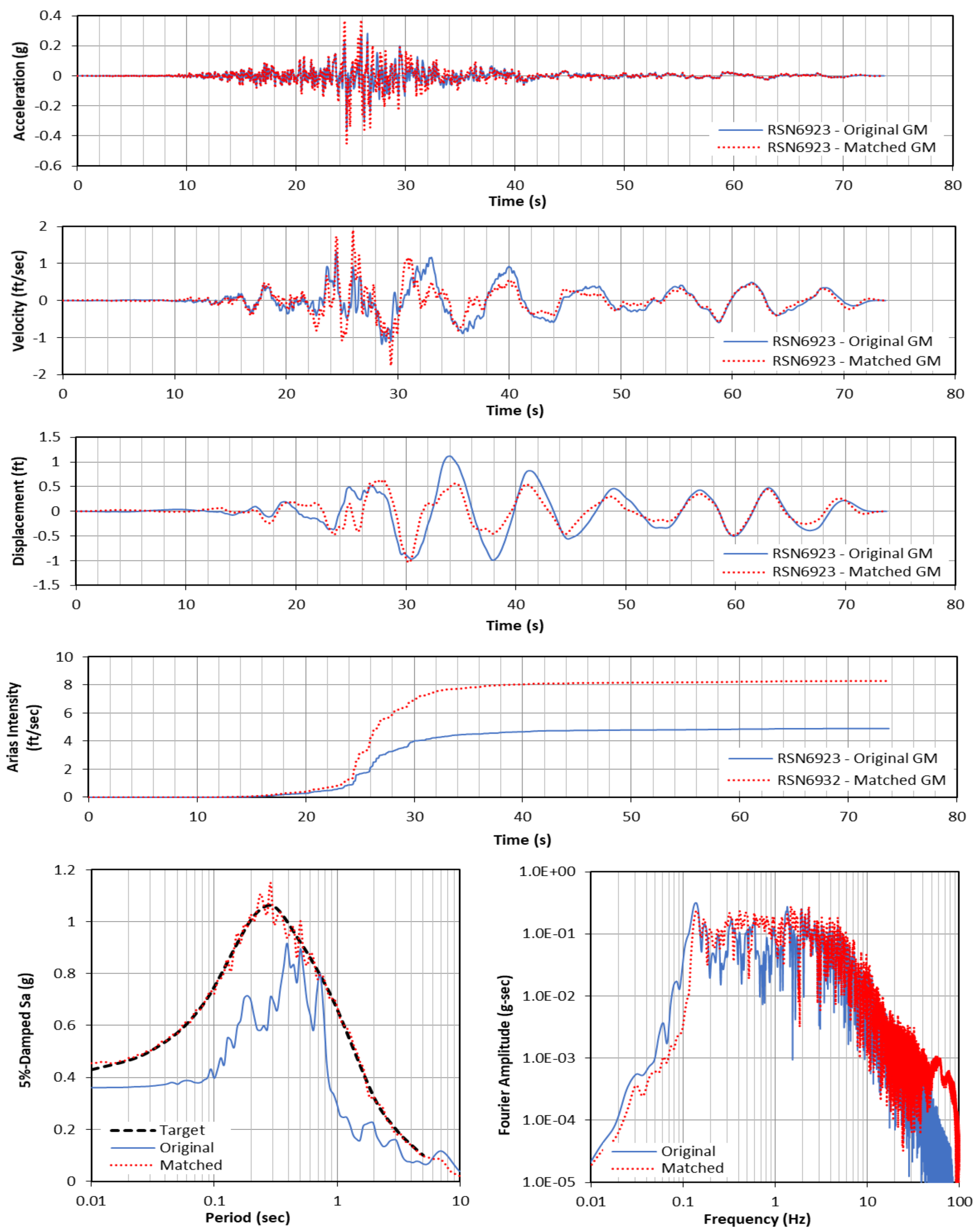
SPECTRAL MATCHING RESULTS
RSN No. 3756 (90º)
LANDERS 1992

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-8



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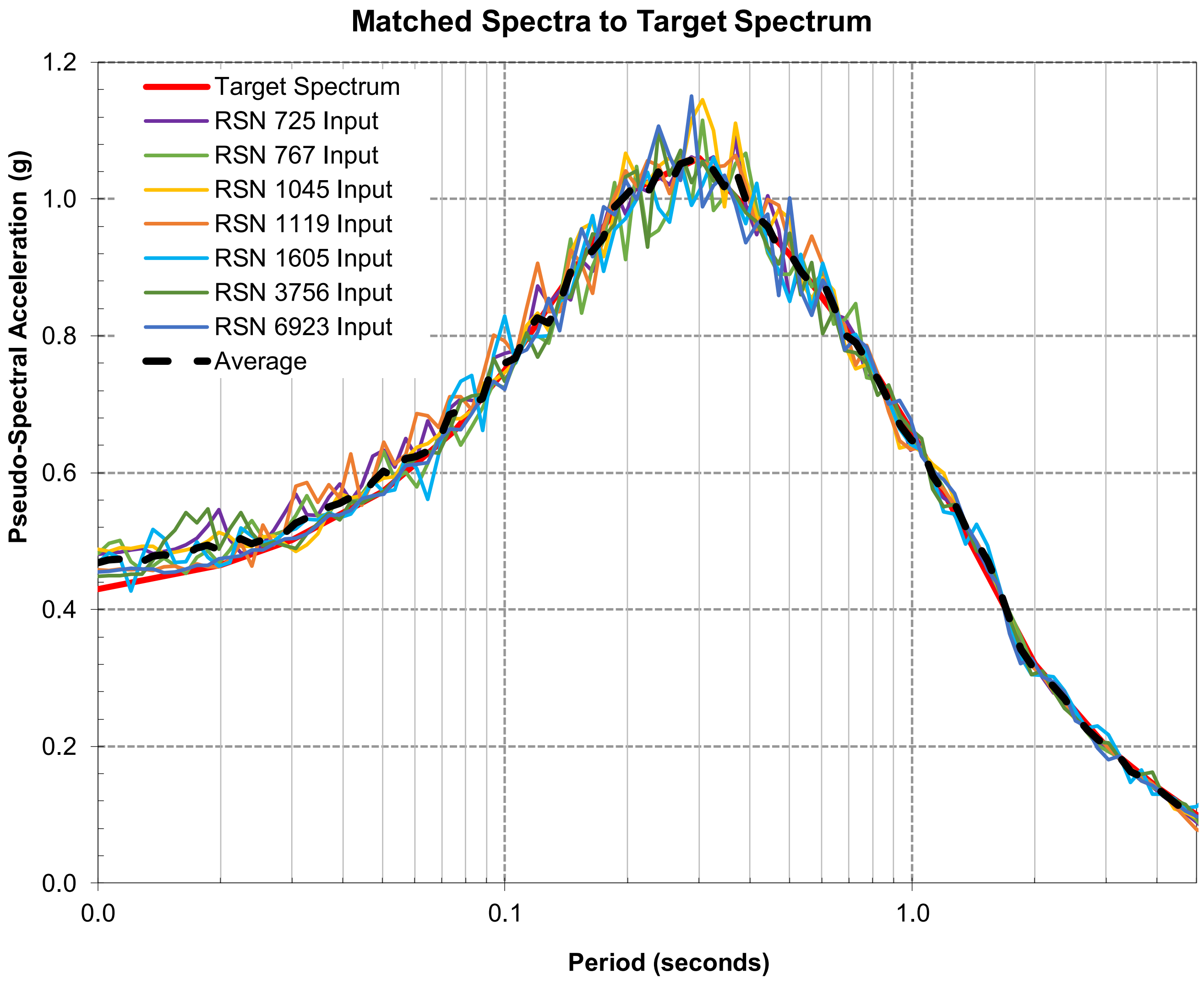
SPECTRAL MATCHING RESULTS
RSN No. 6923 (N15E)
DARFIELD, NEW ZEALAND 2010

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-9



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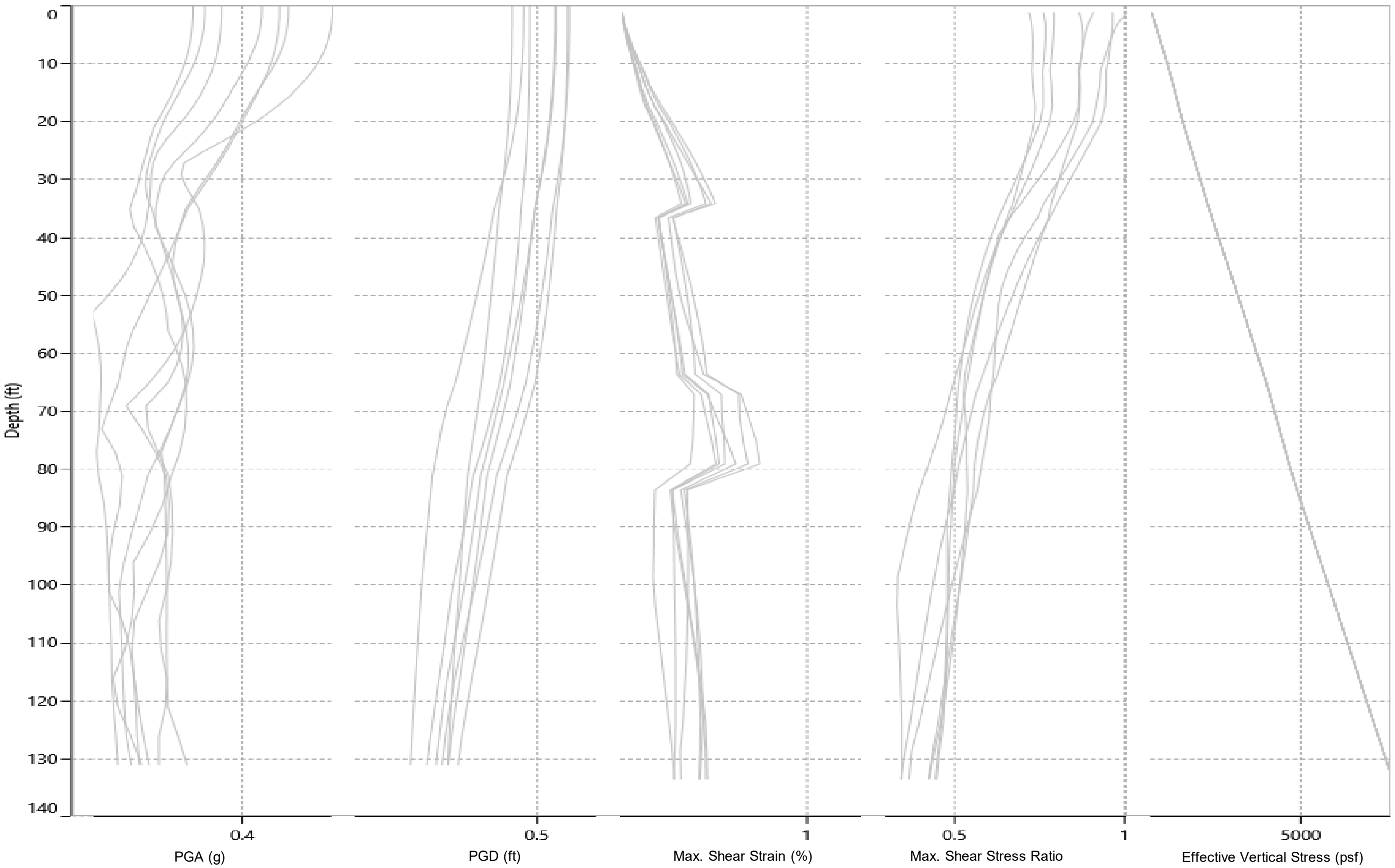
INPUT GROUND MOTIONS RESPONSE SPECTRA VS TARGET SPECTRUM

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-10



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DATE:	06/2020
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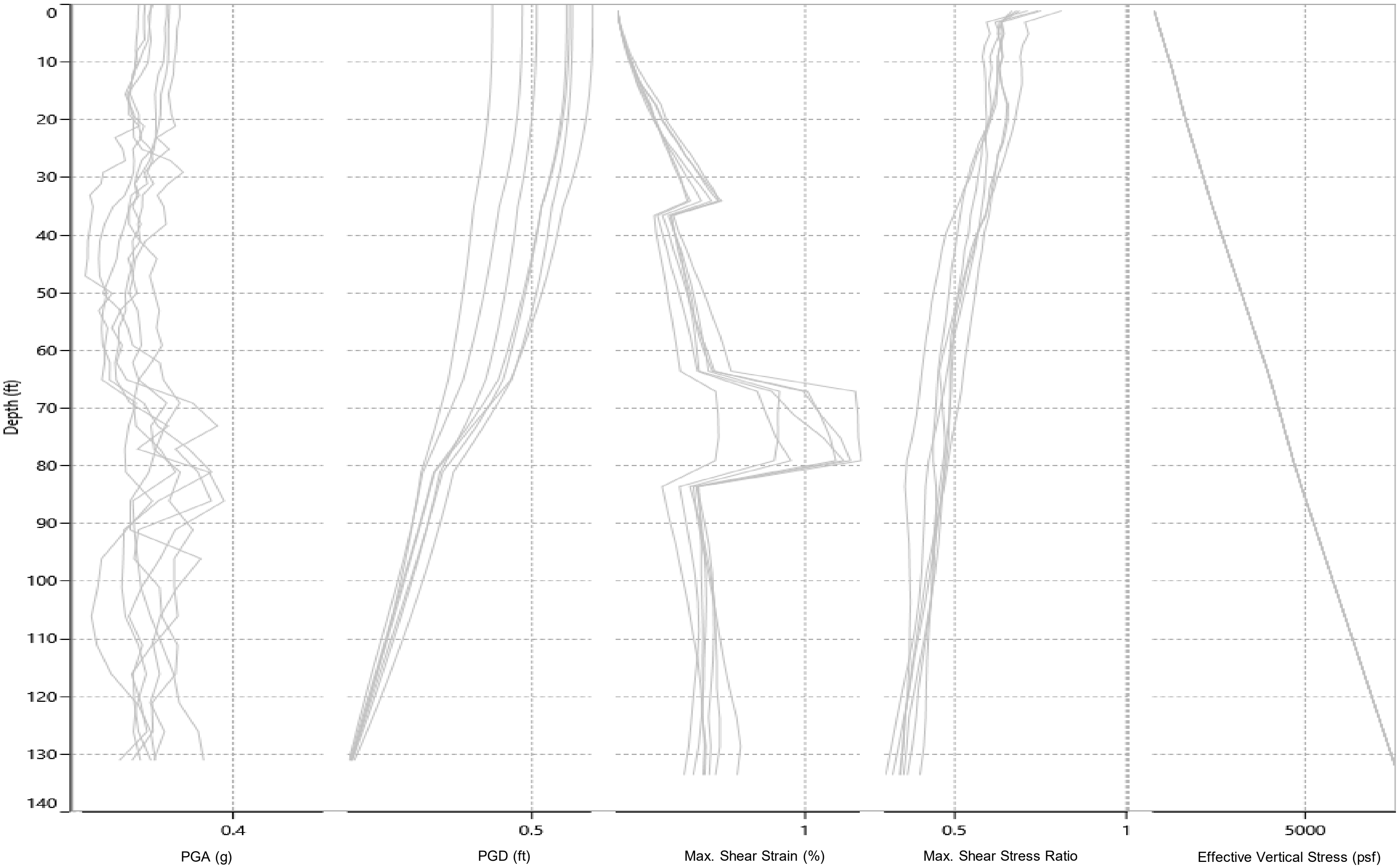
File Name:	Results Prof Plots.ppt

EQUIVALENT LINEAR SITE RESPONSE RESULTS VS DEPTH GM1 TO GM7

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER – PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-11



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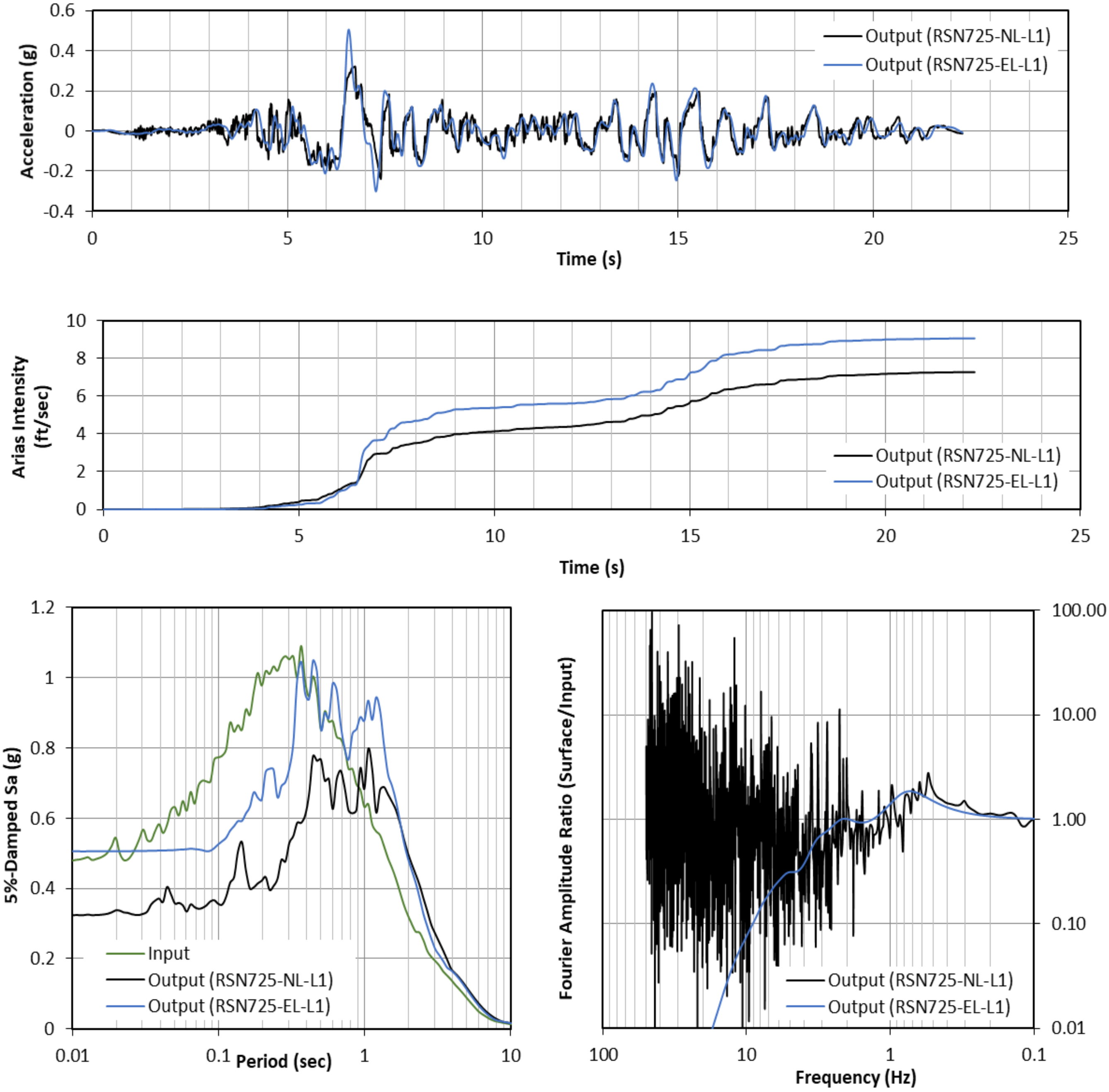
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	Results Prof Plots.ppt

NON-LINEAR SITE RESPONSE RESULTS VS DEPTH GM1 TO GM7

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER – PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-12



Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

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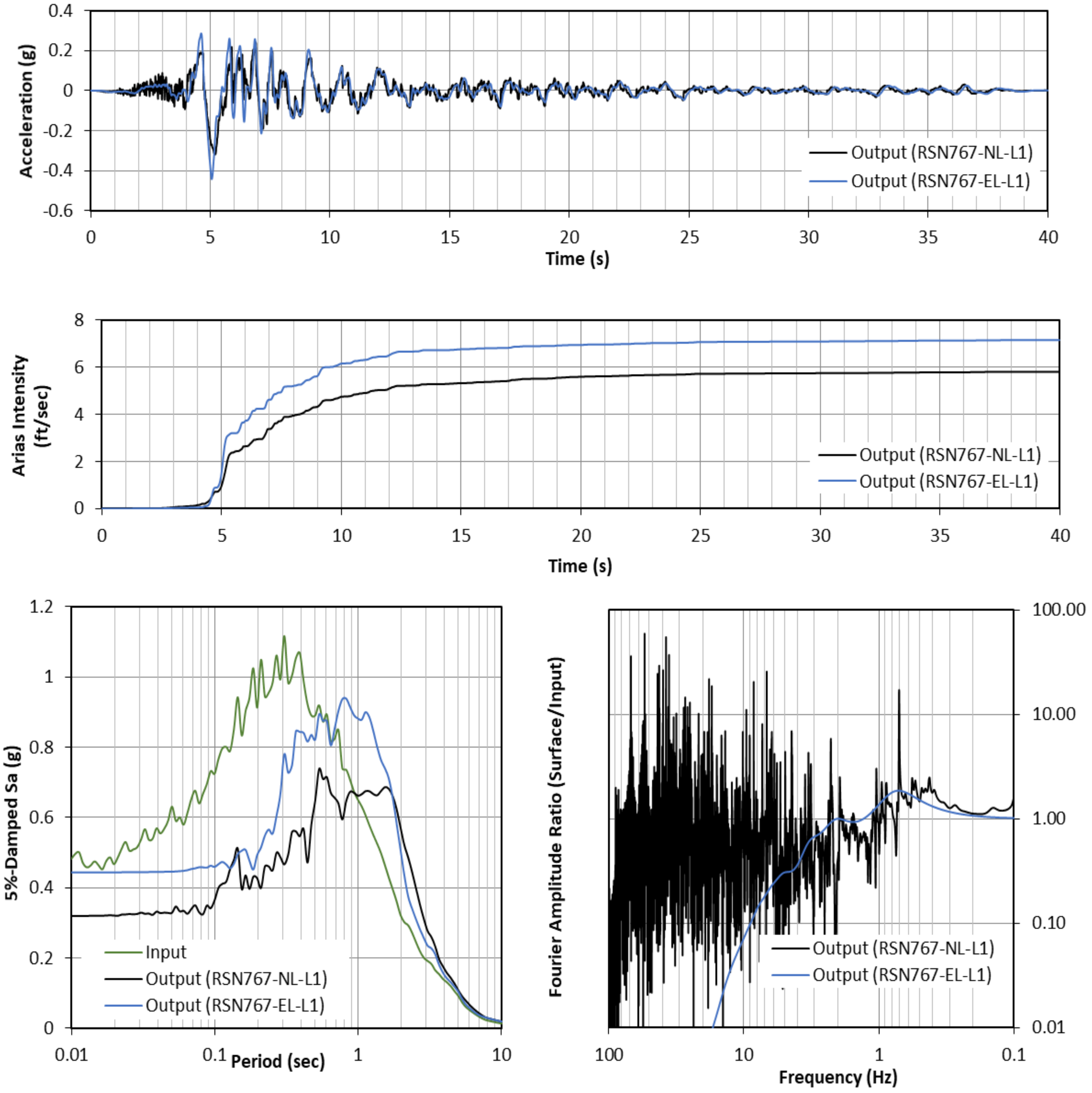
PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM1 Response.ppt

GROUND MOTION RSN725
RESPONSE AT GROUND SURFACE

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-13



Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

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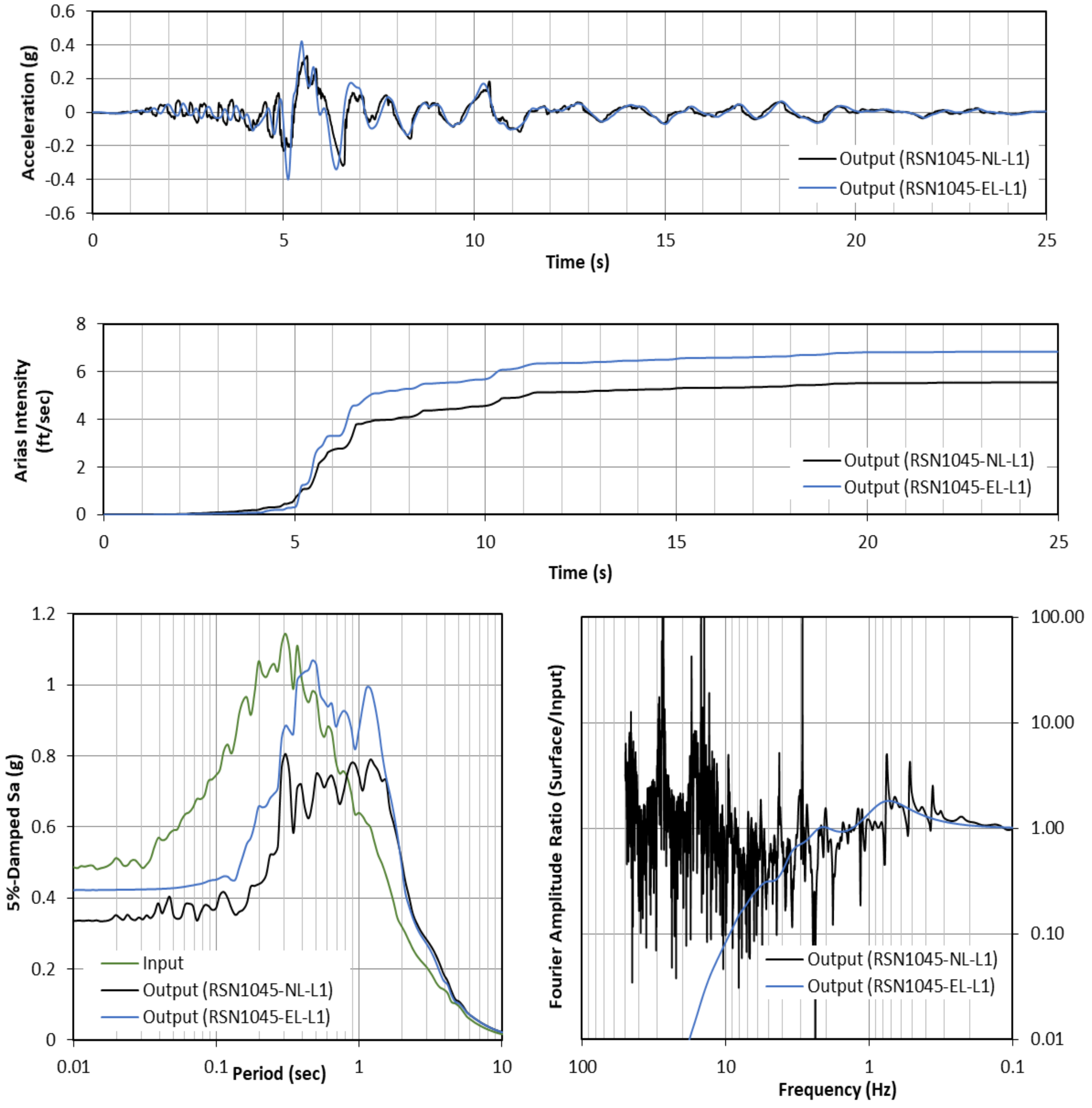
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM2 Response.ppt

GROUND MOTION RSN767
RESPONSE AT GROUND SURFACE

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-14



Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

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DATE: 06/2020
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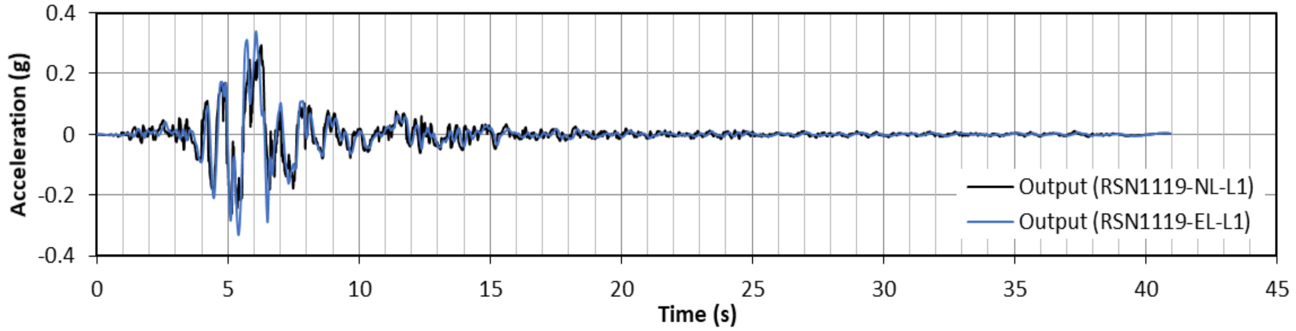
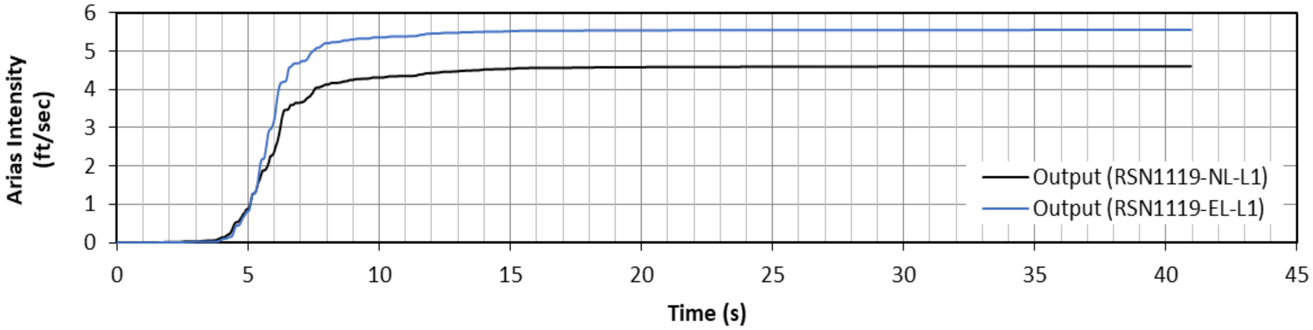
CHECKED BY: ZZ
File Name: GM3 Response.ppt

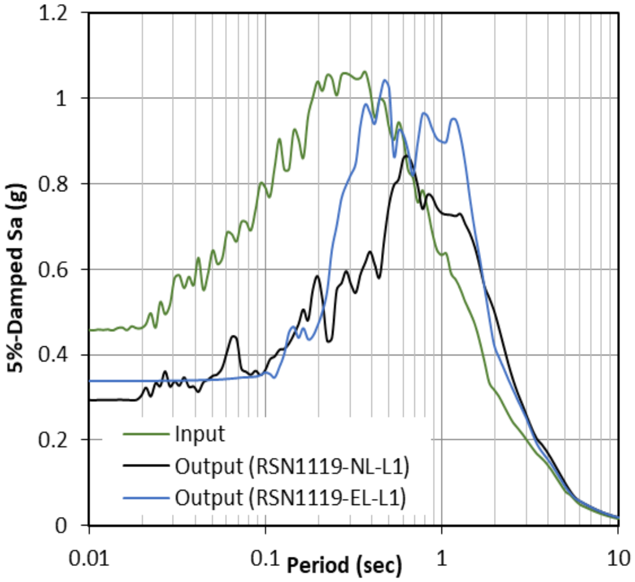
GROUND MOTION RSN1045
RESPONSE AT GROUND SURFACE

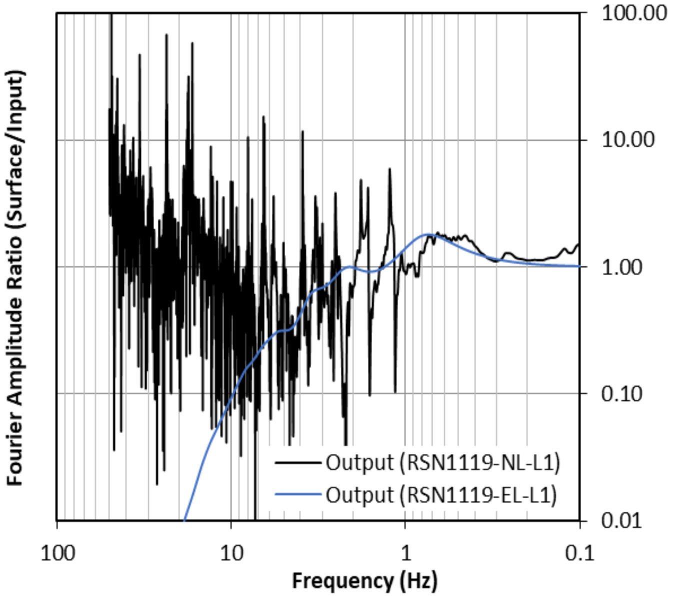
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-15









Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

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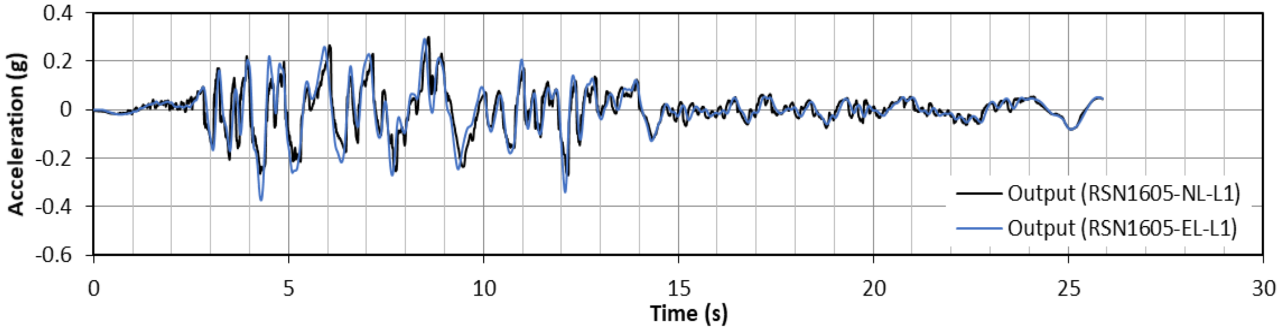
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	GM4 Response.ppt

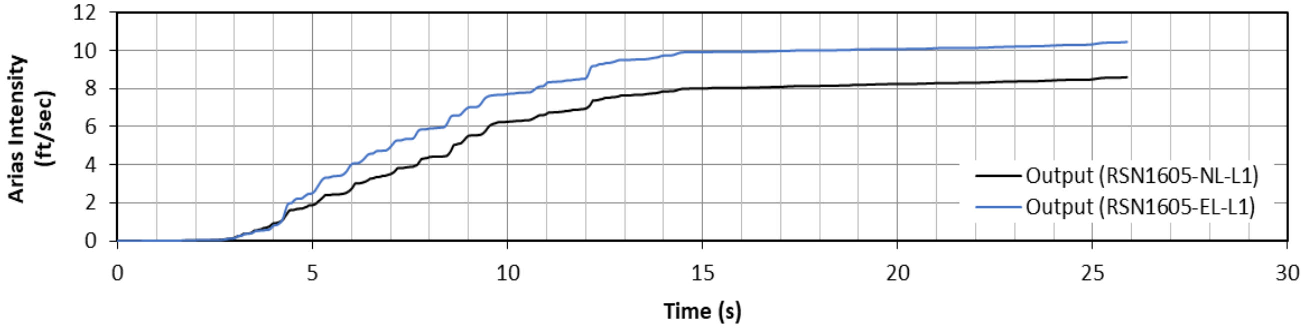
GROUND MOTION RSN1119
RESPONSE AT GROUND SURFACE

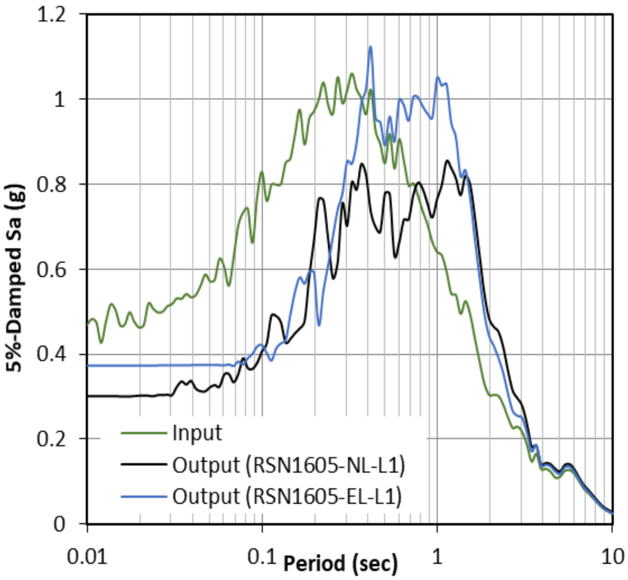
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

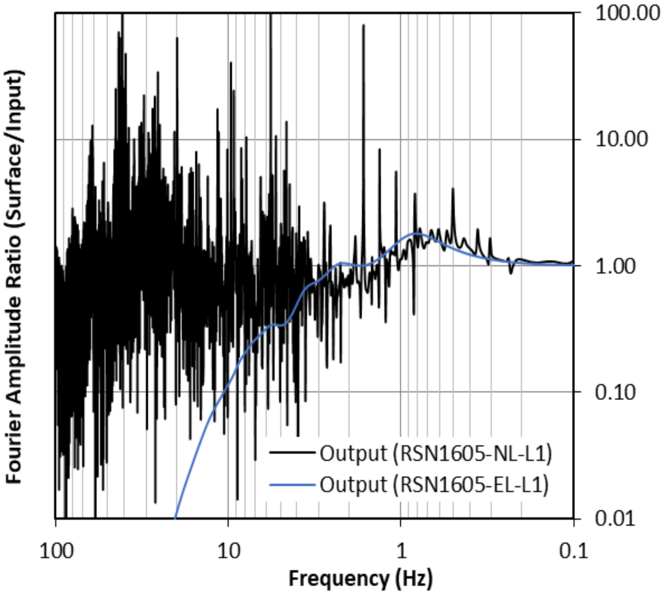
FIGURE

F-16









Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

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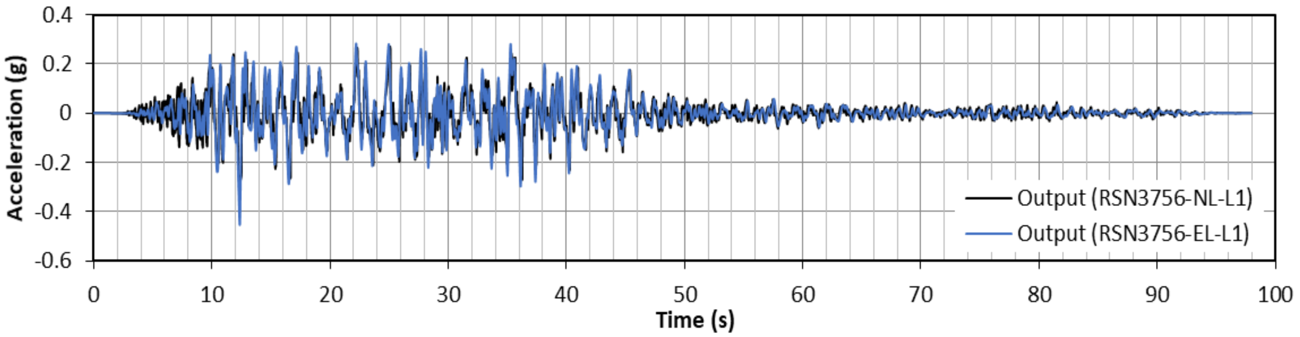
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	GM5 Response.ppt

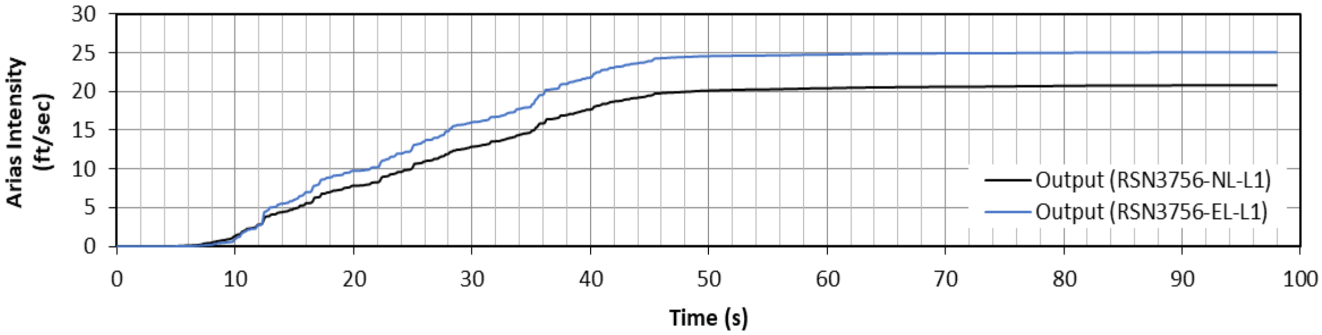
GROUND MOTION RSN1605
RESPONSE AT GROUND SURFACE

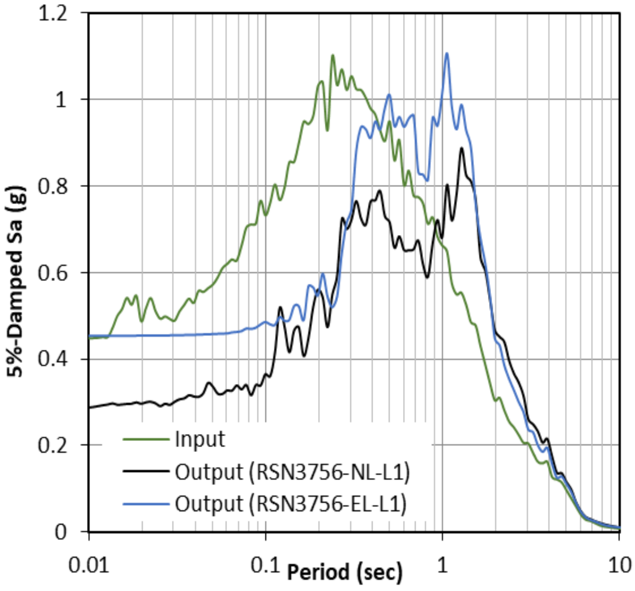
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

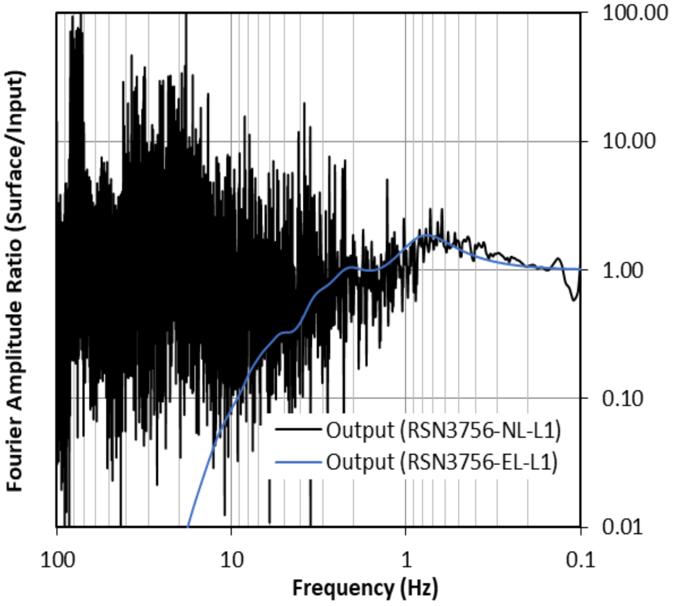
FIGURE

F-17









Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

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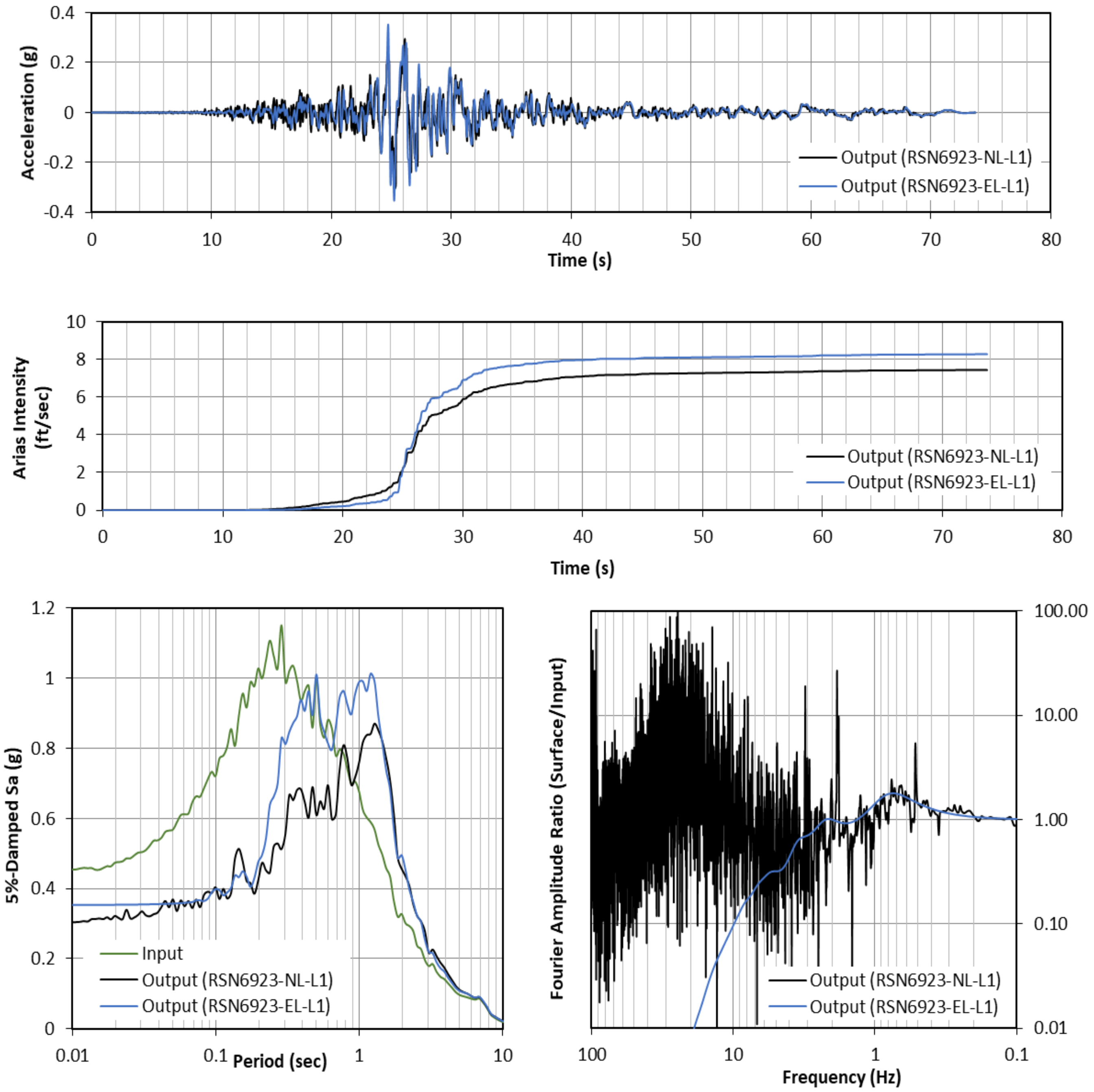
PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM6 Response.ppt

GROUND MOTION RSN3756
RESPONSE AT GROUND SURFACE

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-18



Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

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PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	GM7 Response.ppt

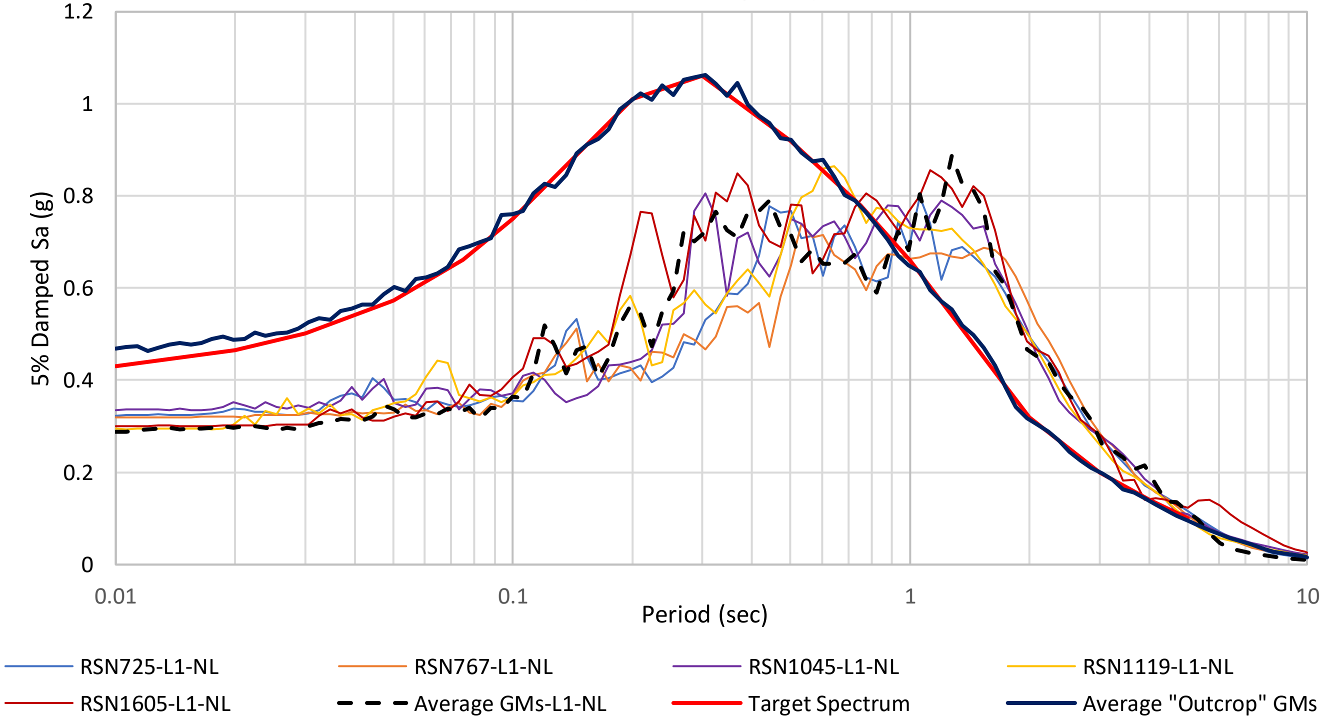
GROUND MOTION RSN6923
RESPONSE AT GROUND SURFACE

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

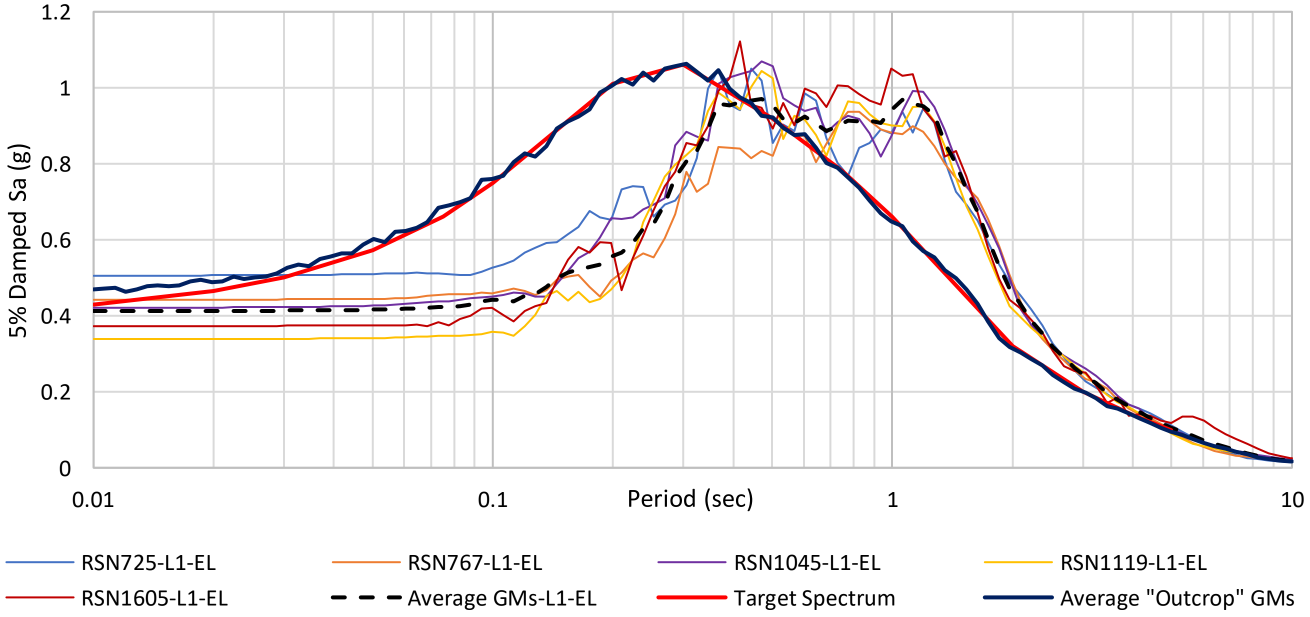
FIGURE

F-19

Non-Linear Site Response Results at Surface vs “Outcrop” and Target Spectra



Equivalent Linear Site Response Results at Surface vs “Outcrop” and Target Spectra



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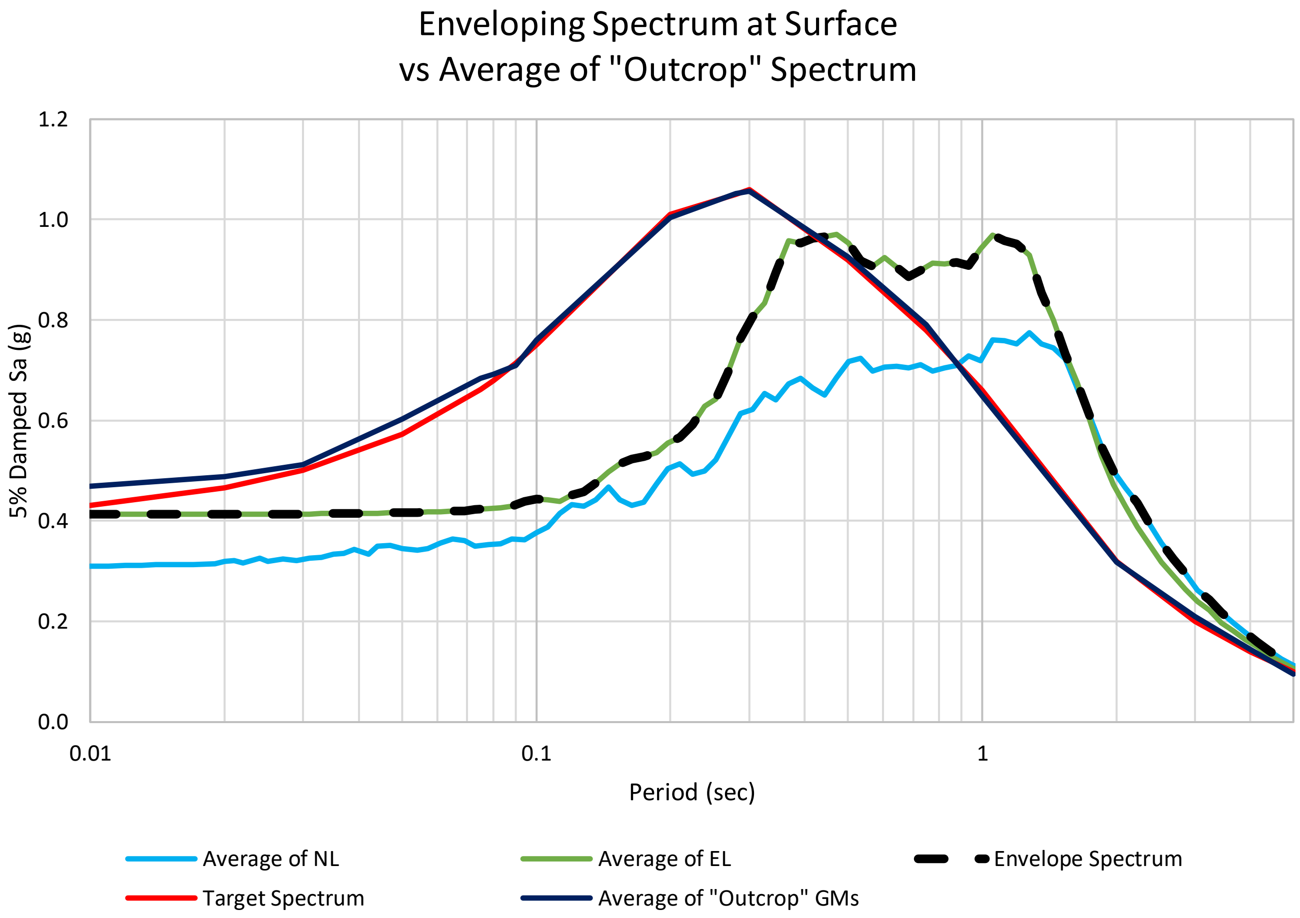
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: SiteResults.ppt

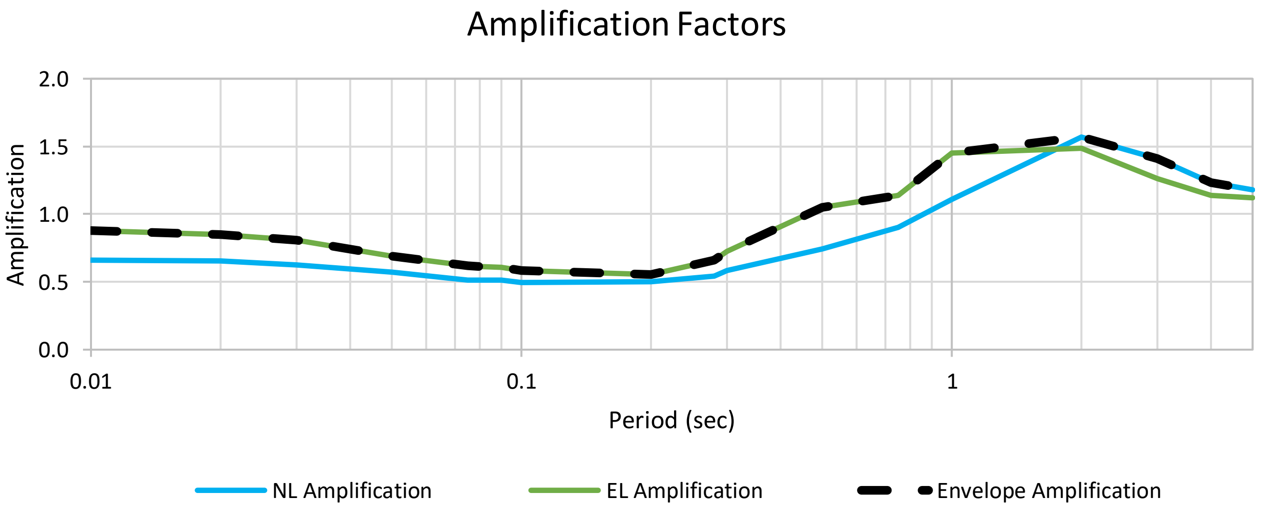
AVERAGE SITE RESPONSE RESULTS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-20



Enveloping Spectrum at Surface vs Average of "Outcrop" Spectrum



Amplification Factors

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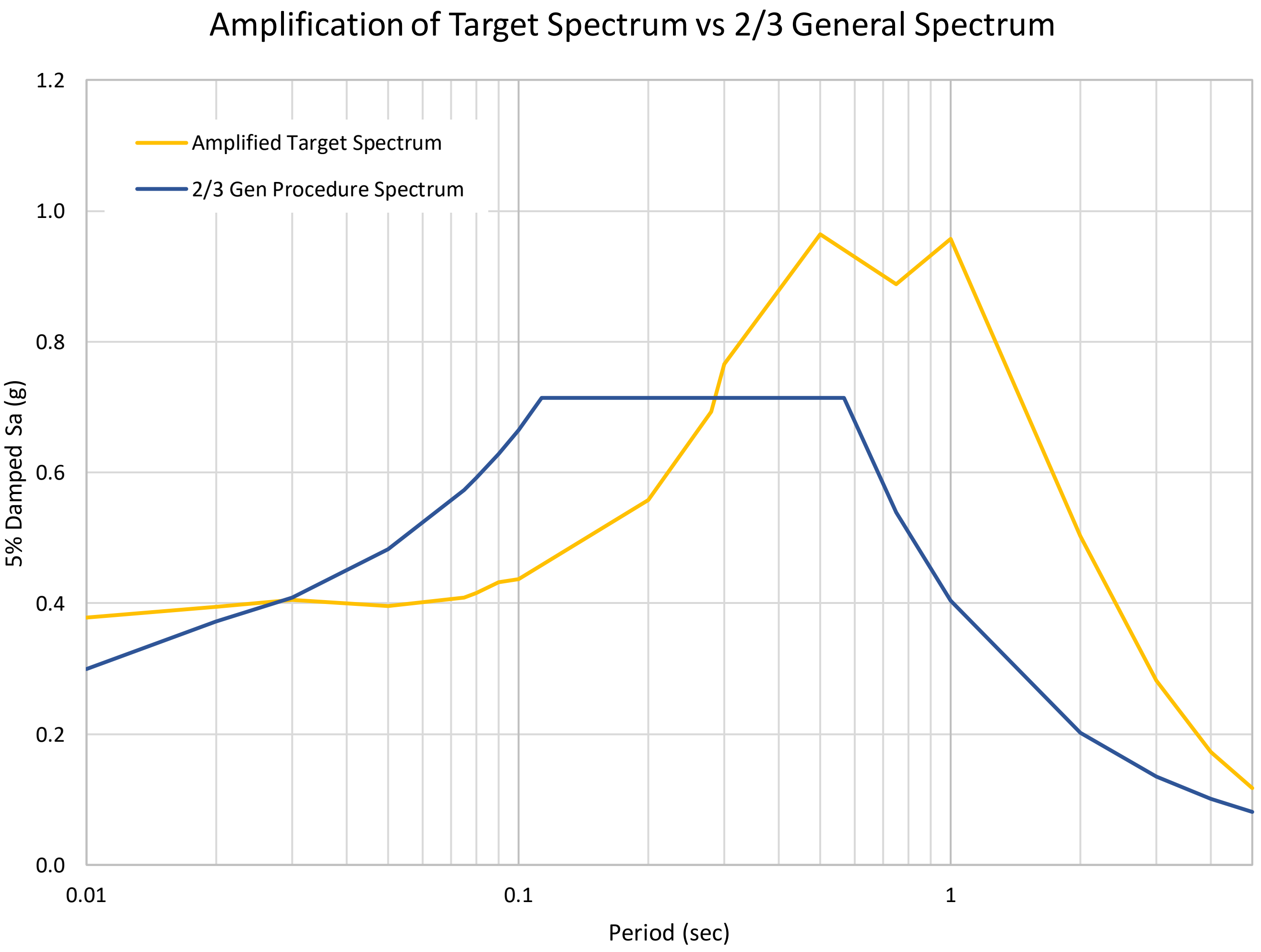
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	Amplification.ppt

ENVELOPE SPECTRUM AND SITE AMPLIFICATION RESULTS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-21



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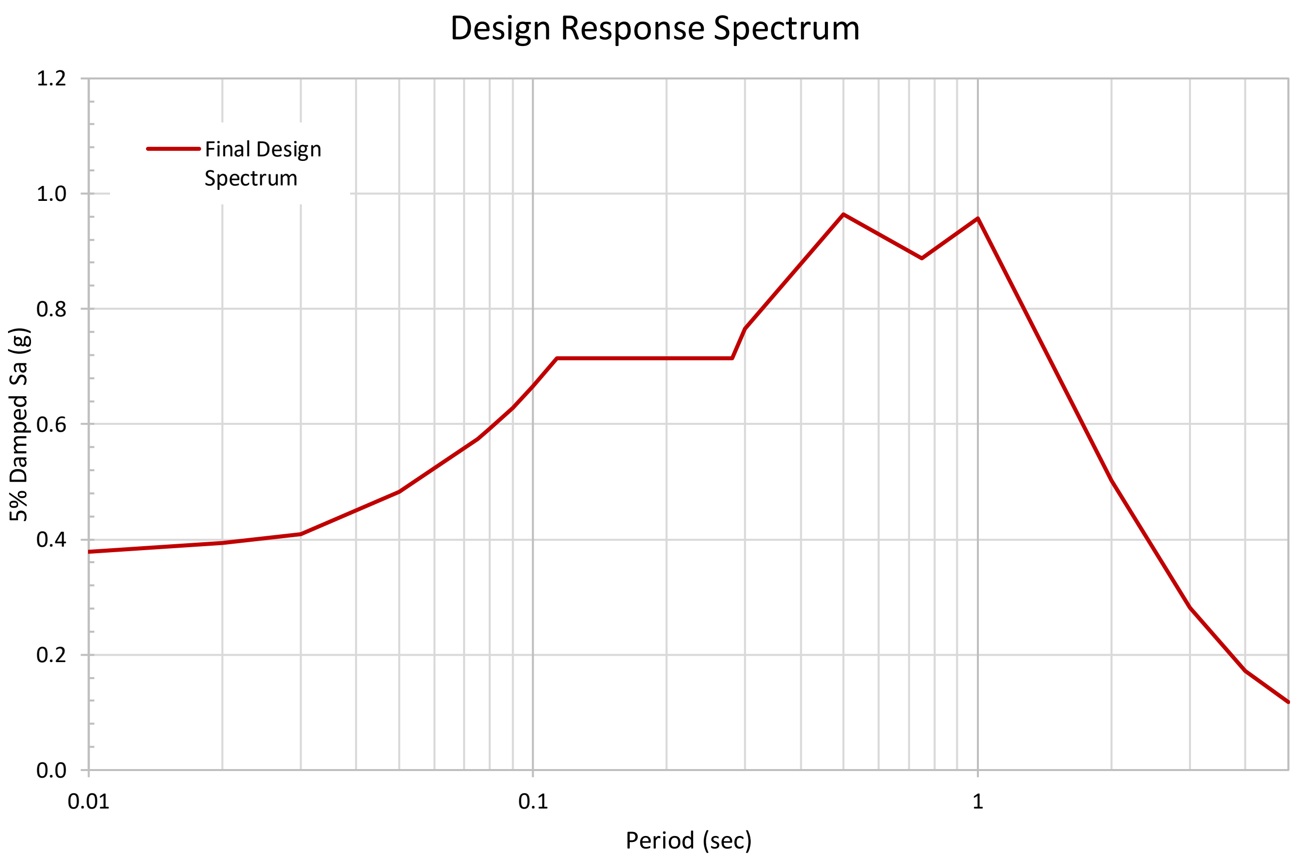
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	Design Spectrum.ppt

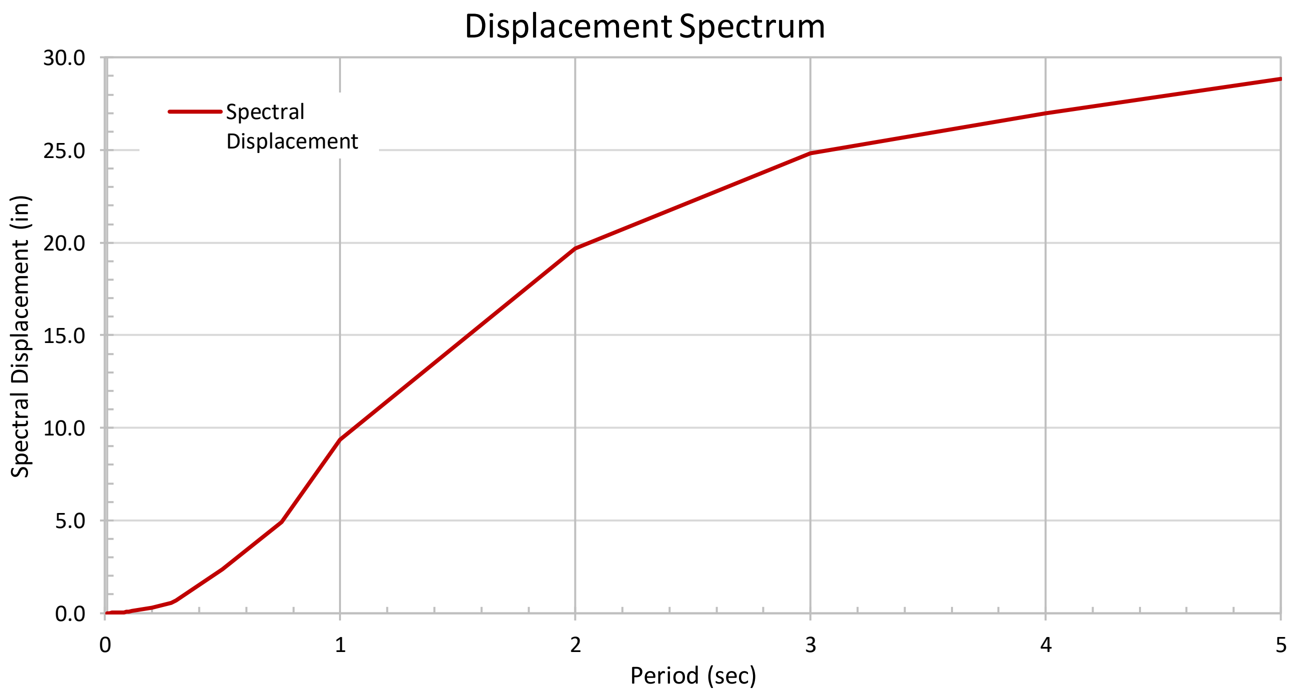
AMPLIFICATION SPECTRUM COMPARISON TO LOWER LIMIT

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-22





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PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	Design Spectrum.ppt

SITE RESPONSE DESIGN SPECTRA

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE

F-23

APPENDIX G

CALCULATIONS

APPENDIX G
CALCULATIONS

G.1 Earth Pressure Calculations

G.2 Liquefaction and Seismic Settlement Calculations

G.3 Liquefaction-Induced Lateral Spreading Analyses

G.4 Slope Stability and Seismic Slope Displacement Analyses

G.5 Axial Pile Capacity Analyses

G.1 EARTH PRESSURE CALCULATIONS

PROJECT: Camino Del Mar Bridge Replacement
PROJECT NO. 20180876.001A DATE: 06/2020
SUBJECT: Earth Pressures for retention walls PERFORMED BY: ST
At-rest, Active, Passive and Seismic REVIEWED BY: JB & KR

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OBJECTIVE: Determine static lateral earth pressures using Coulomb Theory and Mononobe-Okabe seismic earth pressure.

REFERENCE:
AASHTO LRFD Bridge Design Specifications (BDS) with CA Amendments

INPUT SOIL PARAMETERS:

Material Description : Backfill

friction angle, ϕ'_f =	34	(degrees)
cohesion, c =	0	psf
unit weight, γ_s =	120	pcf
soil-wall friction, δ =	24	(degrees)
Buoyant unit weight, γ_B =	57.6	pcf (=γ_s-γ_w)

Wall Geometry:

θ =	90	(degrees)
θ_2 =	0	(degrees)

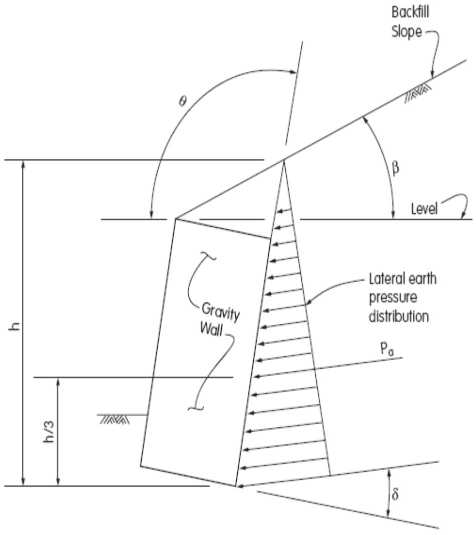


Figure 5.5.5.3-1 Notation for Coulomb Active Lateral Earth Pressure

EQUATIONS FOR EARTH PRESSURE COEFFICIENTS:

At-Rest Earth Pressure Coef.:

$$k_o = (1-\sin\phi'_f)(1+\sin\beta)$$

(BDS Equation 5.5.5.2-1)

Note: use BDS Eqn. 5.5.5.2-2 for OC soil

Active Earth Pressure Coef.:

$$k_a = \frac{\sin^2(\Theta+\phi'_f)}{\Gamma\left[\sin^2\Theta\sin(\Theta-\delta)\right]} \qquad \Gamma = \left[1+\left(\frac{\sin(\phi'_f+\delta)\sin(\phi'_f-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}\right)^{0.5}\right]^2$$

(BDS Eqn's 5.5.5.3-1&2)

Note: Eqn. 5.5.5.3-1 is the same as Rankine k_a when δ = 0

Effective Fluid Unit Weight:

$EFW_{ka} = \gamma_s * k_a$

$EFW_{k0} = \gamma_s * k_0$

CALCULATED STATIC ACTIVE AND AT REST EARTH PRESSURE VALUES:

				Active Pressures		**At-Rest Pressures**		
Slope (H:V)	β (deg)	(rad)	Γ	k_a	EFW_{ka} (lb/ft^3)	k_0	EFW_{k0} (lb/ft^3)	
level	0	0.000	2.960	**0.254**	**30.5**	**0.441**	**52.9**	Above GW
level	0	0.000	2.960	**0.254**	**77.0**	**0.441**	**87.8**	Below GW

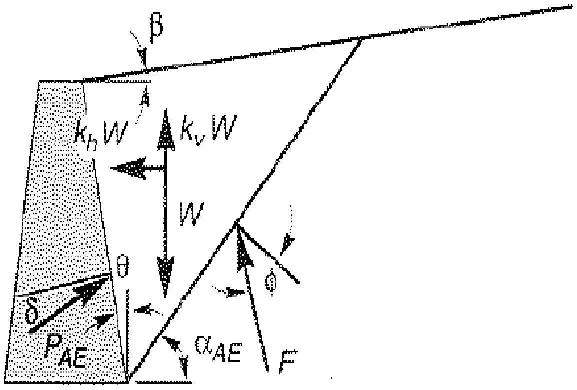
MONONOBE-OKABE SEISMIC EARTH PRESSURES:

PGA = 0.40 g's

k_h = 1/2*PGA = 0.200

k_v = 0.000

Reference:
Kramer, S.L., 1996, *Geotechnical Earthquake Engineering*, Prentice Hall, NJ, pp. 478-481



ψ = **arctan** $(k_h/(1-k_v))$

ψ = 0.197396 (radians)

$$K_{AE} = \frac{\cos^2(\phi-\psi-\theta_2)}{\cos\psi\cos^2\theta_2\cos(\psi+\theta_2+\delta)} \times \left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\psi-\beta)}{\cos(\delta+\psi+\theta_2)\cos(\beta-\theta_2)}}\right]^{-2}$$

$$\boxed{\phi \geq \beta + \psi}$$

$\Delta k_{ae} = k_{AE} - k_a$

$\Delta P_{ae} = 1/2 * \gamma_s * \Delta k_{ae} * H^2$

Slope (H:V)	β (deg)	 (rad)	k_a	k_{AE}	Δk_{ae}	**ΔP_{ae}** (lb/ft)	**$EFW_{\Delta Kae}$** (pcf)
Level	0	0.000	0.254	0.399	0.145	**8.7*H^2**	**17.4**

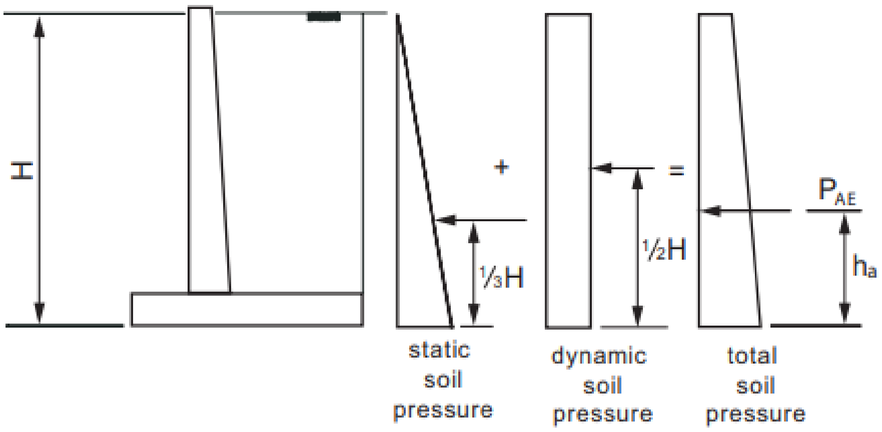


Figure 3 Seismic Loading
(Reference: 2010 California Amendments)

PROJECT: Camino Del Mar Bridge Replacement
PROJECT NO. 20180876.001A DATE: 06/2020
SUBJECT: Earth Pressures for retention walls PERFORMED BY: ST
At-rest, Active, Passive and Seismic REVIEWED BY: JB & KR

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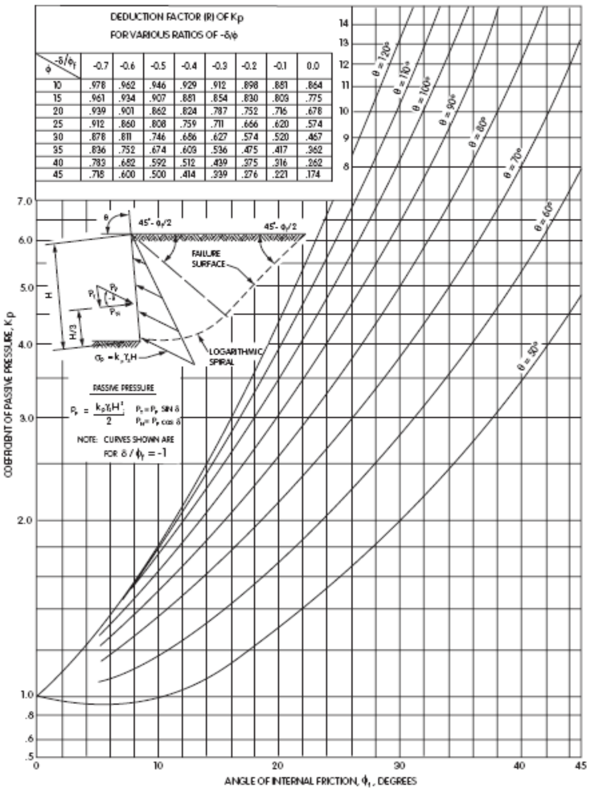
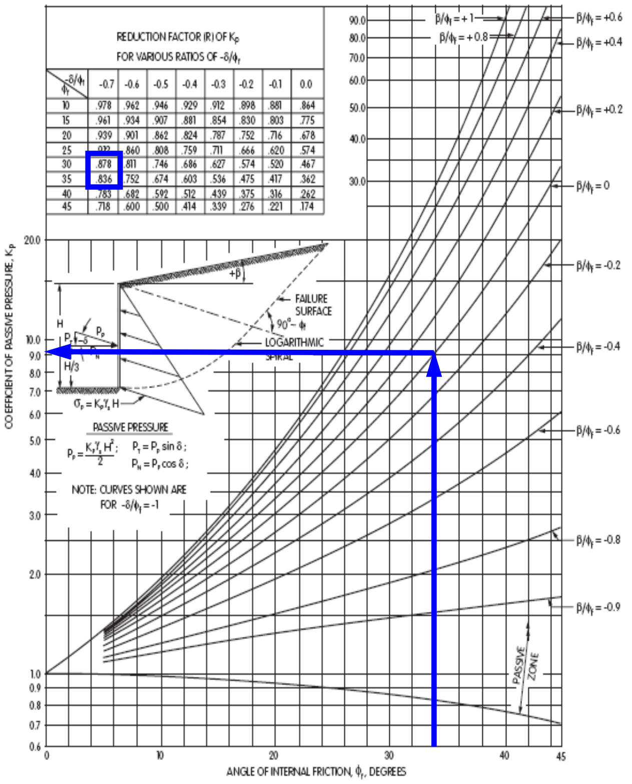
PASSIVE LATERAL EARTH PRESSURE COEFFICIENT, k_p

Use Caltrans charts for log-spirol



φ \ -δ/φf	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

BDS Figure 5.5.5.4-1 - Sloping Wall and Level Backfill



φf \ -δ/φf	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

BDS Figure 5.5.5.4-2 - Vertical Wall with Sloping Backfill

$-\delta/\phi'_f$ = -0.71 beta/phi 0

CALCULATED EARTH PRESSURE VALUES (Above GW):

						Passive Pressure		
Slope (H:V)	β (deg)	(rad)	β/ϕ'_f	un-reduced k_p	**Reduction Factor, R**	k_p	EFW_{kp} (lb/ft³)	Design. (lb/ft³)
level	0	0.000	0.00	9.0	0.844	**7.60**	**911.5**	**900**
						below GW	**499.9**	**500**
					with LRFD Factor = 0.5:			450
					below GW with LRFD Factor = 0.5:			250

G.2 LIQUEFACTION AND SEISMIC SETTLEMENT CALCULATIONS

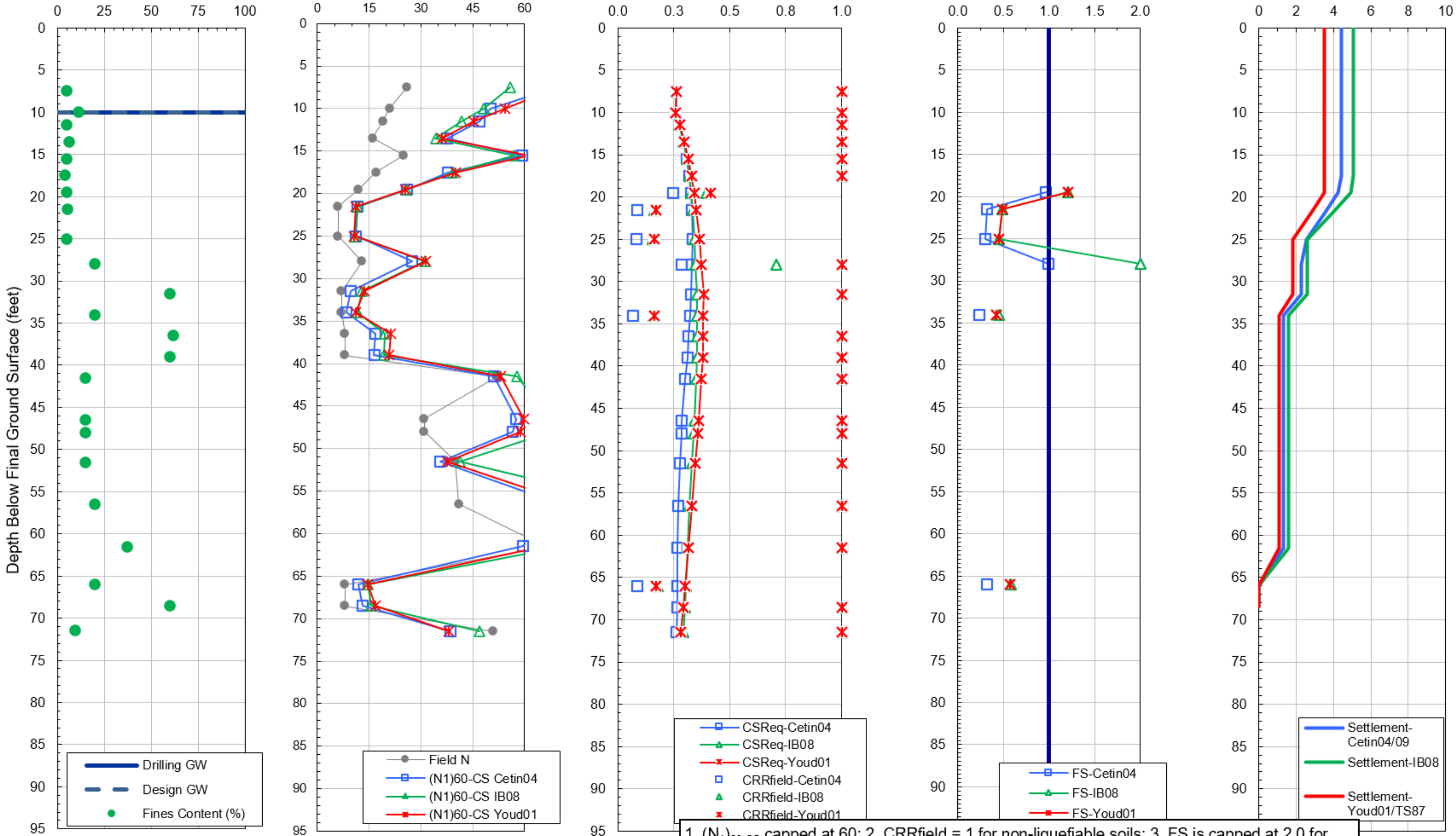
Boring ID: **R-20-001 (0 to 70 ft)**

M_w= **6.6**
PGA= **0.41g**

Groundwater Depth During Drilling (ft) = **10.0 ft**
Design Groundwater Depth (ft) = **10.0 ft**

Existing Ground Elevation = **16.0 ft**
Final Ground Elevation = **16.0 ft**

Ana. by: J. Bonfiglio
Checked by: Z. Zafir



1. $(N_1)_{60\text{-}CS}$ capped at 60; 2. CRRfield = 1 for non-liquefiable soils; 3. FS is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils including soils above G.W.T.

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Project Name: **Camino Del Mar Bridge Replacement**
Project No.: **20180876.001A**
Project Location: **Del Mar, California**

LIQUEFACTION ANALYSIS
(R-20-001)

Date 5/11/2020

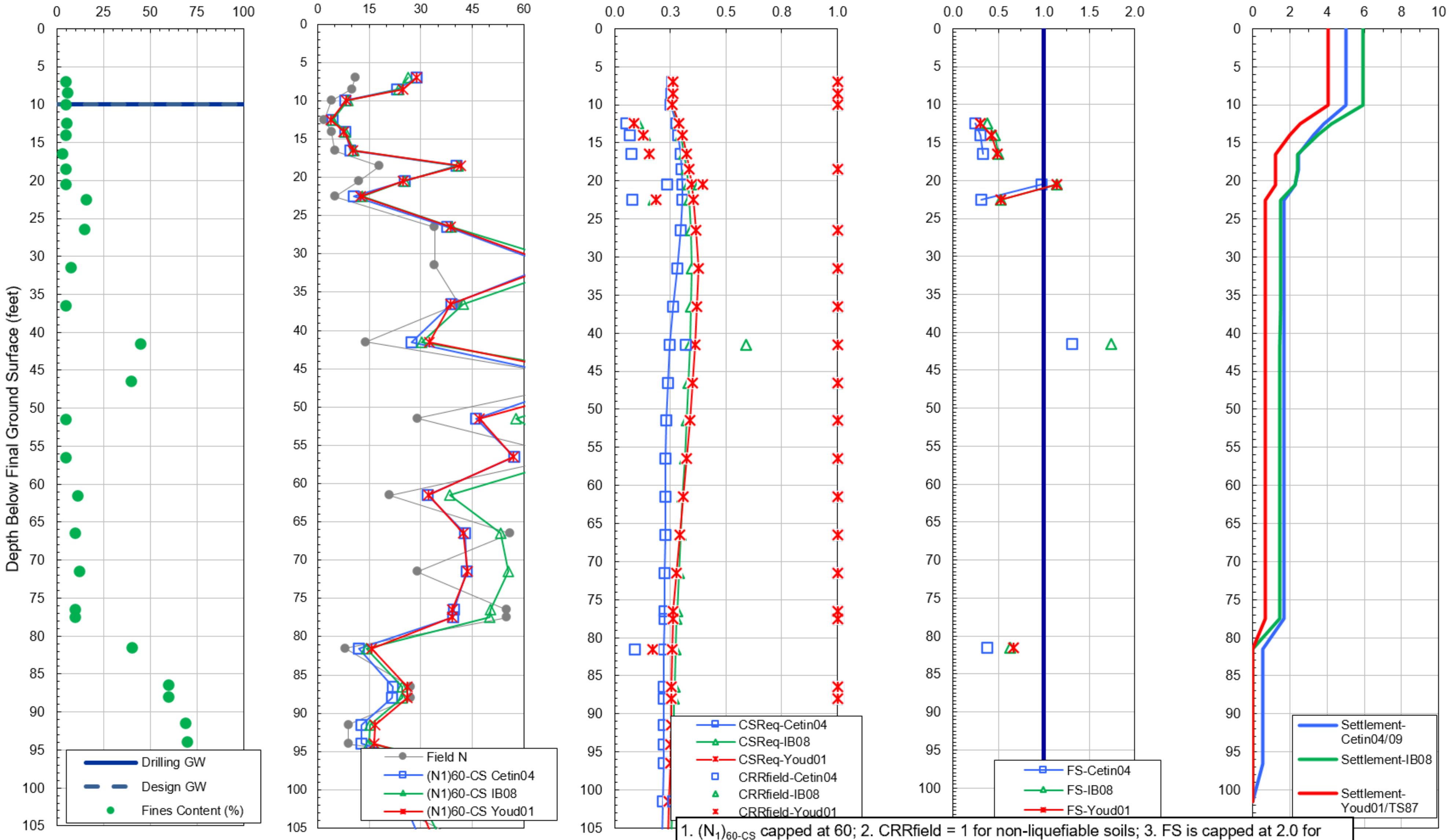
Boring ID: **R-20-002 (0 to 70 ft)**

M_w= **6.6**
PGA= **0.41g**

Groundwater Depth During Drilling (ft) = **10.0 ft**
Design Groundwater Depth (ft) = **10.0 ft**

Existing Ground Elevation = **16.0 ft**
Final Ground Elevation = **16.0 ft**

Ana. by: J. Bonfiglio
Checked by: Z. Zafir



1. $(N_1)_{60\text{-}CS}$ capped at 60; 2. CRRfield = 1 for non-liquefiable soils; 3. FS is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils including soils above G.W.T.

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Project Name: **Camino Del Mar Bridge Replacement**
Project No.: **20180876.001A**
Project Location: **Del Mar, California**

LIQUEFACTION ANALYSIS
(R-20-002)

Date 5/11/2020

Seismic Settlement of Dry Sands
Tokimatsu & Seed (1987)

Project No. 20180876.001A
Project Name Camino Del Mar Bridge Replacement
Analysis by J. Bonfiglio
Checked by Z. Zafir

M = 6.63 Moment Magnitude (Use Modal value
PHA = 0.41 g (Peak horizontal acceleration; use PGA_M)
γ = 120 pcf (unit weight of soil
Ko = 0.5 (at-rest coefficient)

																				Results				
Boring	**Depth at middle of sampler (ft)**	**Layer Thickness (ft)**	**Soil Classification**	**Anticipated Fines Content (%)**	r_d	σ_0 **(psf)**	σ'_m **(psf)**	σ'_m **(tsf)**	**N (blows/ft)**	**SAMPLER TYPE (1) SPT w/out liners (2) SPT w/ liners (3) MC (4) CAL**	**Sampler Correction,** C_S	**Overbuden Correction,** C_N	**Fine Content Correction**	N_1 **(blows/ft)**	**Gmax (psf)**	**Effective Shear Strain,** γ_{eff} **(Geff/Gmax)**	**Effective Shear Strain,** γ_{eff} **(from Fig. 11)**	**Effective Shear Strain,** γ_{eff} **(%)**	**Volumetric Strain (from Figure 13) (%)**	**Seismic Settlement for M7.5 (in)**	**Seismic Settlement for M5.25 (in)**	**Seismic Settlement for M6 (in)**	**Seismic Settlement for M6.75 (in)**	**Seismic Settlement for M8.5 (in)**
R-20-001	3	6	SP-SM	5	0.993	360	240	0.12	26	1	1.1	1.70	1.0	50	1138556	8.37E-05	1.6E-04	1.6E-02	0.0000	0.000	0.00	0.00	0.00	0.00
R-20-001	7	2	SP-SM	5	0.985	840	560	0.28	26	1	1.1	1.54	1.0	45	1685058	1.31E-04	1.9E-04	1.9E-02	0.0000	0.000	0.00	0.00	0.00	0.00
R-20-001	9	2	SP-SM	11	0.980	1080	720	0.36	21	1	1.1	1.36	1.1	33	1712637	1.65E-04	2.9E-04	2.9E-02	0.0150	0.007	0.00	0.00	0.01	0.01
																				0.000	0.00	0.00	0.00	0.00
																				0.000	0.00	0.00	0.00	0.00
																				0.00	0.00	0.00	0.00	0.00
																				0.00	0.00	0.00	0.00	0.00
																				0.007	0.00	0.00	0.01	0.01

Double the value for bi-directional shaking

Select= 0.01 for M 6.63

(use to read Fig. 11)

(use to read Fig. 13)

(use to read Fig. 11)

(use to read Fig. 13)

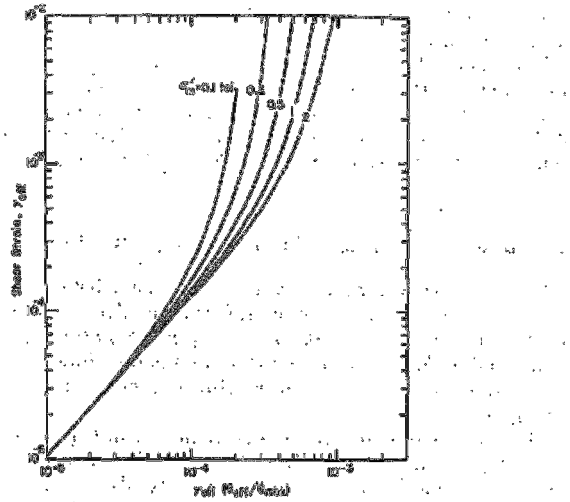


FIG. 11.—Plot for Determination of Induced Strain in Sand Deposits

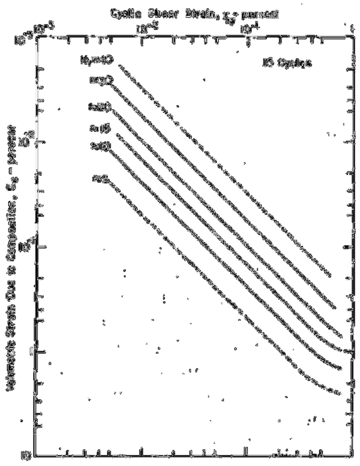


FIG. 13.—Relationship between Volumetric Strain, Shear Strain, and Penetration Resistance for Dry Sands

Seismic Settlement of Dry Sands
Tokimatsu & Seed (1987)

Project No. 20180876.001A
Project Name Camino Del Mar Bridge Replacemer
Analysis by J. Bonfiglio
Checked by Z. Zafir

M = 6.63 Moment Magnitude (Use Modal value
PHA = 0.41 g (Peak horizontal acceleration; use PG_M)
γ = 120 pcf (unit weight of soil)
Ko = 0.5 (at-rest coefficient)

																				Results				
Boring	Depth at middle of sampler (ft)	Layer Thickness (ft)	Soil Classification	Anticipated Fines Content (%)	r_d	σ_0 (psf)	σ'_m (psf)	σ'_m (tsf)	N (blows/ft)	SAMPLER TYPE (1) SPT w/out liners (2) SPT w/ liners (3) MC (4) CAL	Sampler Correction, C_S	Overbuden Correction, C_N	Fine Content Correction	N_1 (blows/ft)	Gmax (psf)	Effective Shear Strain, γ_{eff} (Geff/Gmax)	Effective Shear Strain, γ_{eff} (from Fig. 11)	Effective Shear Strain, γ_{eff} (%)	Volumetric Strain (from Figure 13) (%)	Seismic Settlement for M7.5 (in)	Seismic Settlement for M5.25 (in)	Seismic Settlement for M6 (in)	Seismic Settlement for M6.75 (in)	Seismic Settlement for M8.5 (in)
R-20-002	2.5	5	SP	3.3	0.995	300	200	0.10	10	1	1.1	1.70	1.0	20	763895	1.04E-04	2.4E-04	2.4E-02	0.0250	0.030	0.01	0.02	0.03	0.04
R-20-002	6	2	SP-SM	5	0.987	720	480	0.24	11	1	1.1	1.67	1.0	21	1212090	1.56E-04	2.7E-04	2.7E-02	0.0250	0.012	0.00	0.01	0.01	0.02
R-20-002	8	2	SP-SM	5.6	0.983	960	640	0.32	10	1	1.1	1.44	1.0	17	1297835	1.94E-04	4.8E-04	4.8E-02	0.0550	0.026	0.01	0.02	0.02	0.03
R-20-002	9.5	1	SP-SM	5	0.979	1140	760	0.38	4	1	1.1	1.32	1.0	7	1046004	2.84E-04	1.2E-03	1.2E-01	0.4000	0.096	0.04	0.06	0.08	0.12
																				0.000	0.00	0.00	0.00	0.00
																				0.00	0.00	0.00	0.00	0.00
																				0.00	0.00	0.00	0.00	0.00
																				0.164	0.07	0.10	0.14	0.21

Double the value for bi-directional shakin
Select= 0.27 for M 6.63

(use to read Fig. 11)

(use to read Fig. 13)

(use to read Fig. 11)

(use to read Fig. 13)

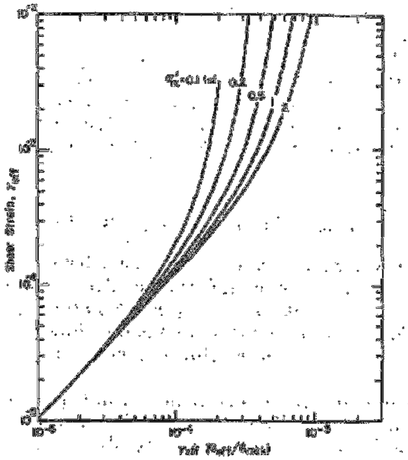


FIG. 11.—Plot for Determination of Induced Strain in Sand Deposits

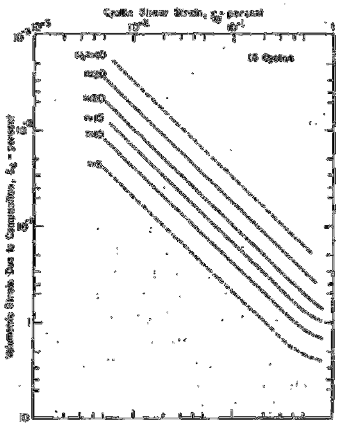
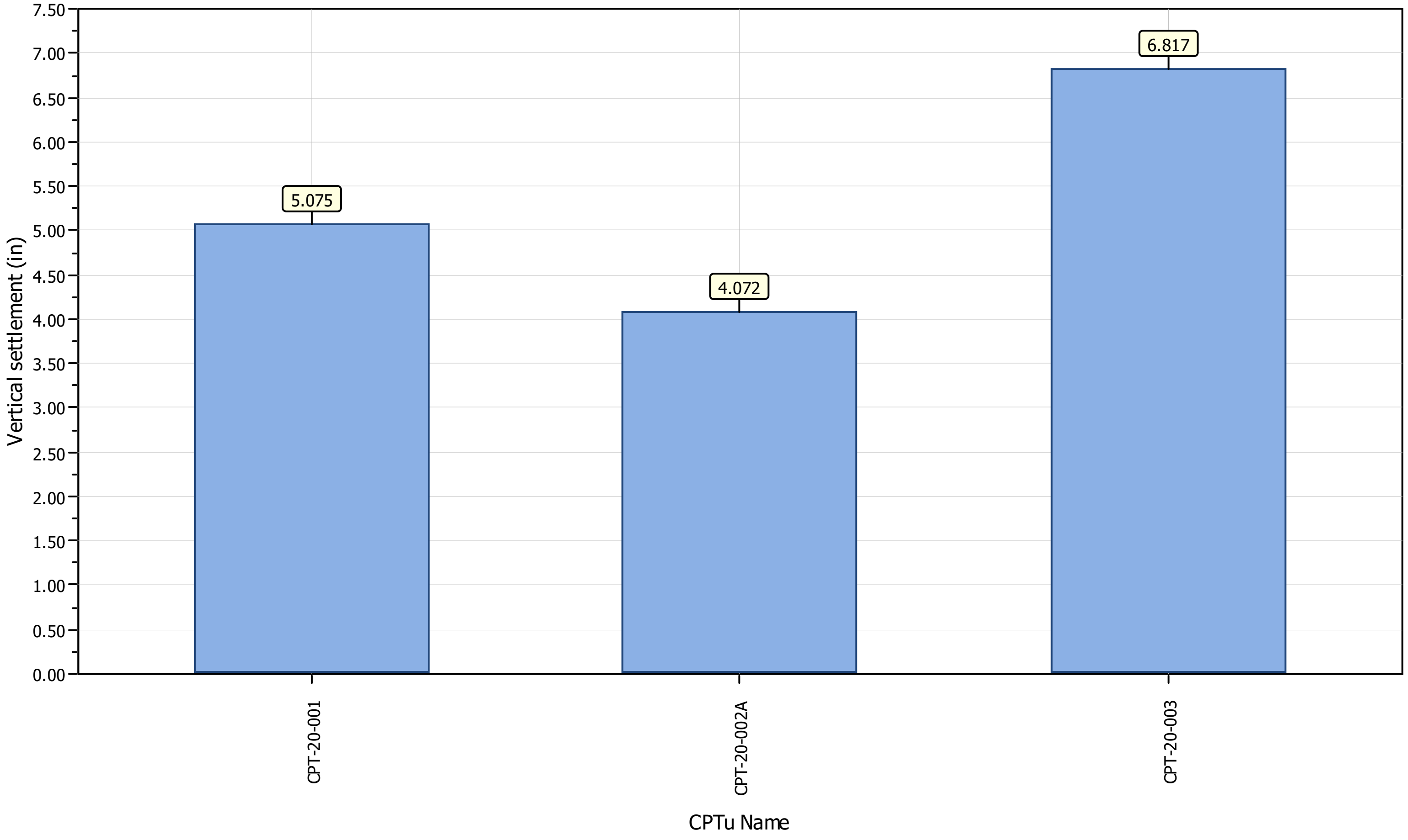


FIG. 13.—Relationship between Volumetric Strain, Shear Strain, and Penetration Resistance for Dry Sands

Project title : Camino Del Mar Bridge Replacement

Location : Del Mar, CA

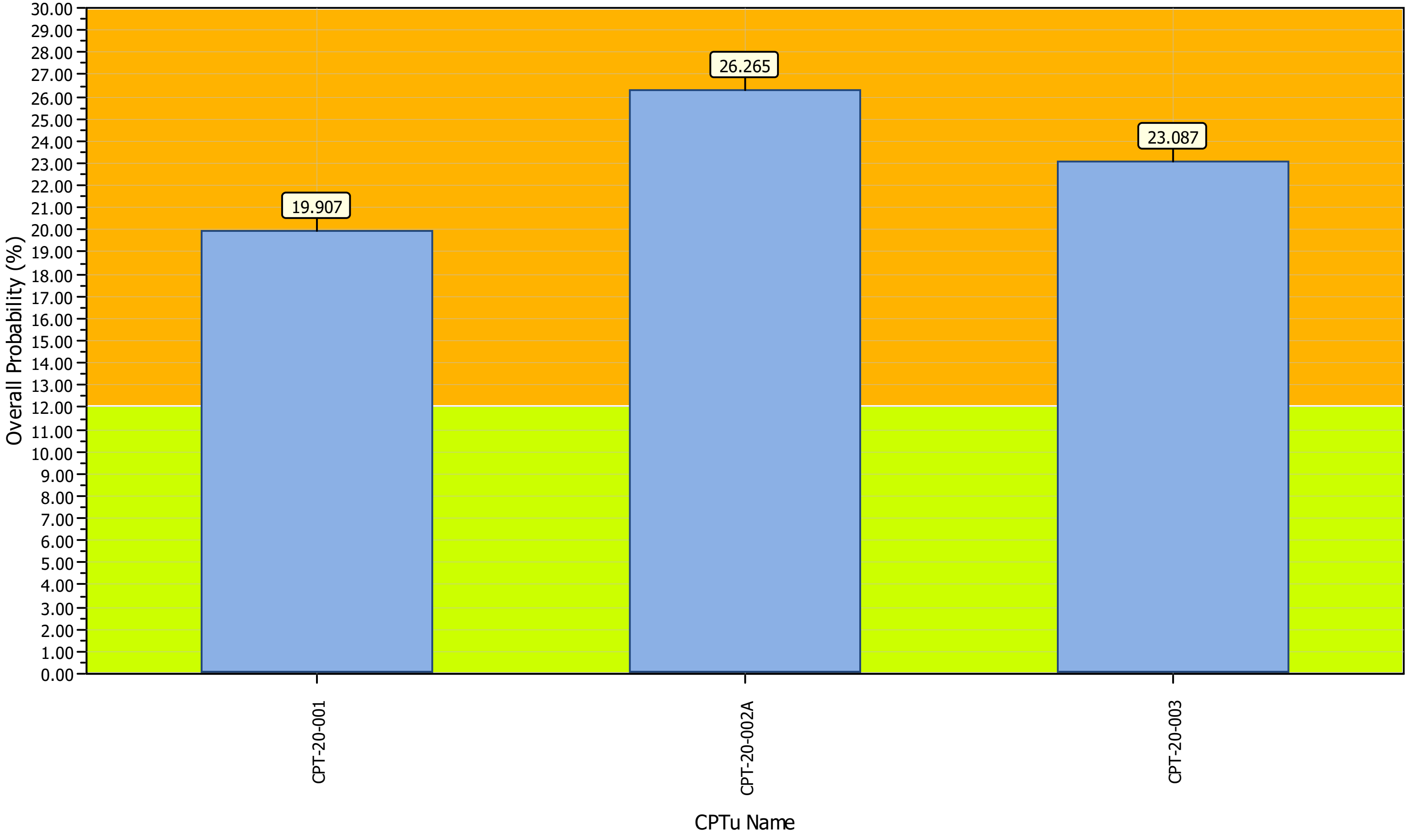
Overall vertical settlements report



Project title : Camino Del Mar Bridge Replacement

Location : Del Mar, CA

Overall Probability for Liquefaction report



Probability color scheme

🟥 Very High Probability

🟧 High Probability

🟩 Low Probability

Basic statistics

Total CPT number: 3

0% low probability

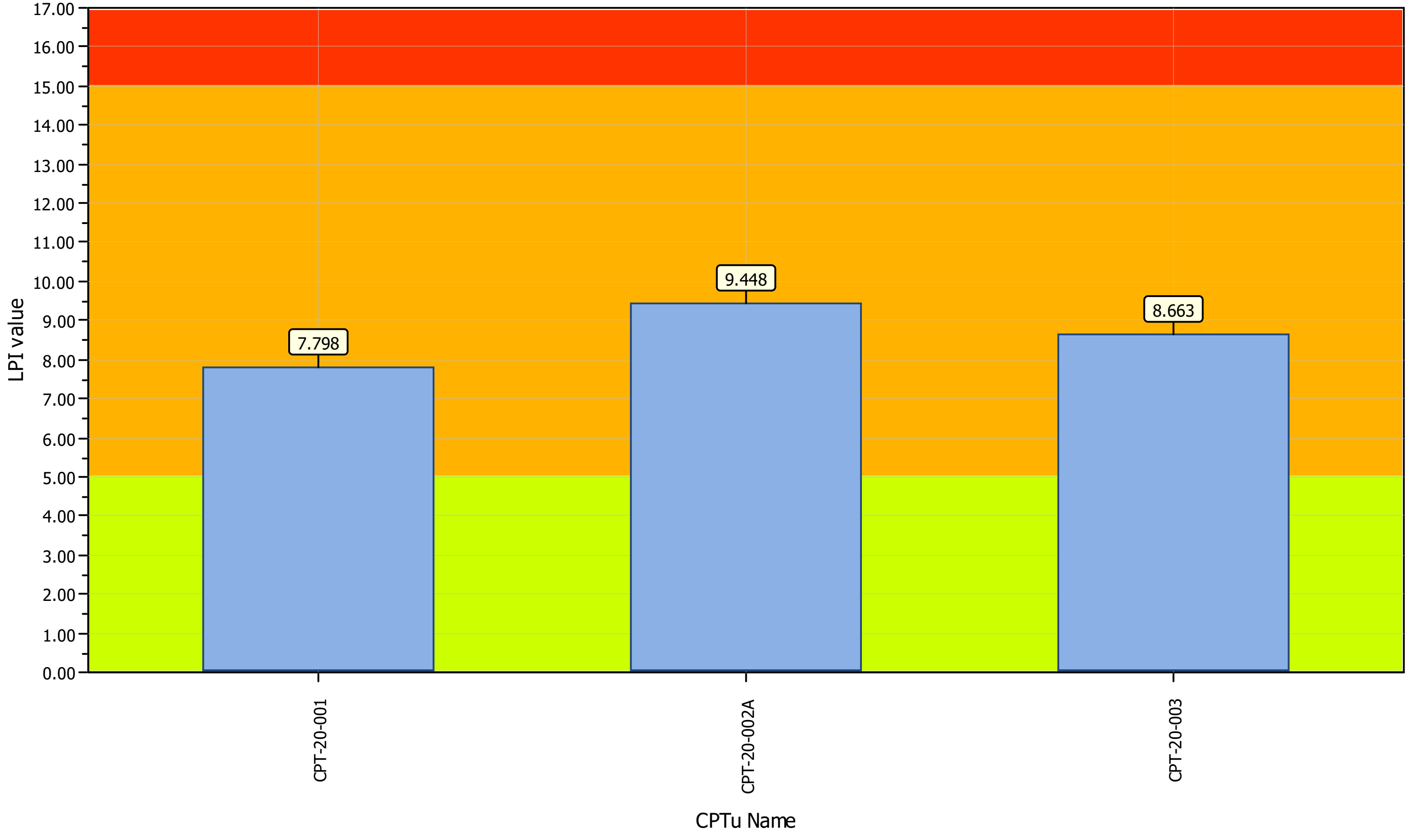
100% high probability

0% very high probability

Project title : Camino Del Mar Bridge Replacement

Location : Del Mar, CA

Overall Liquefaction Potential Index report



LPI color scheme

🟥 Very high risk

🟧 High risk

🟩 Low risk

Basic statistics

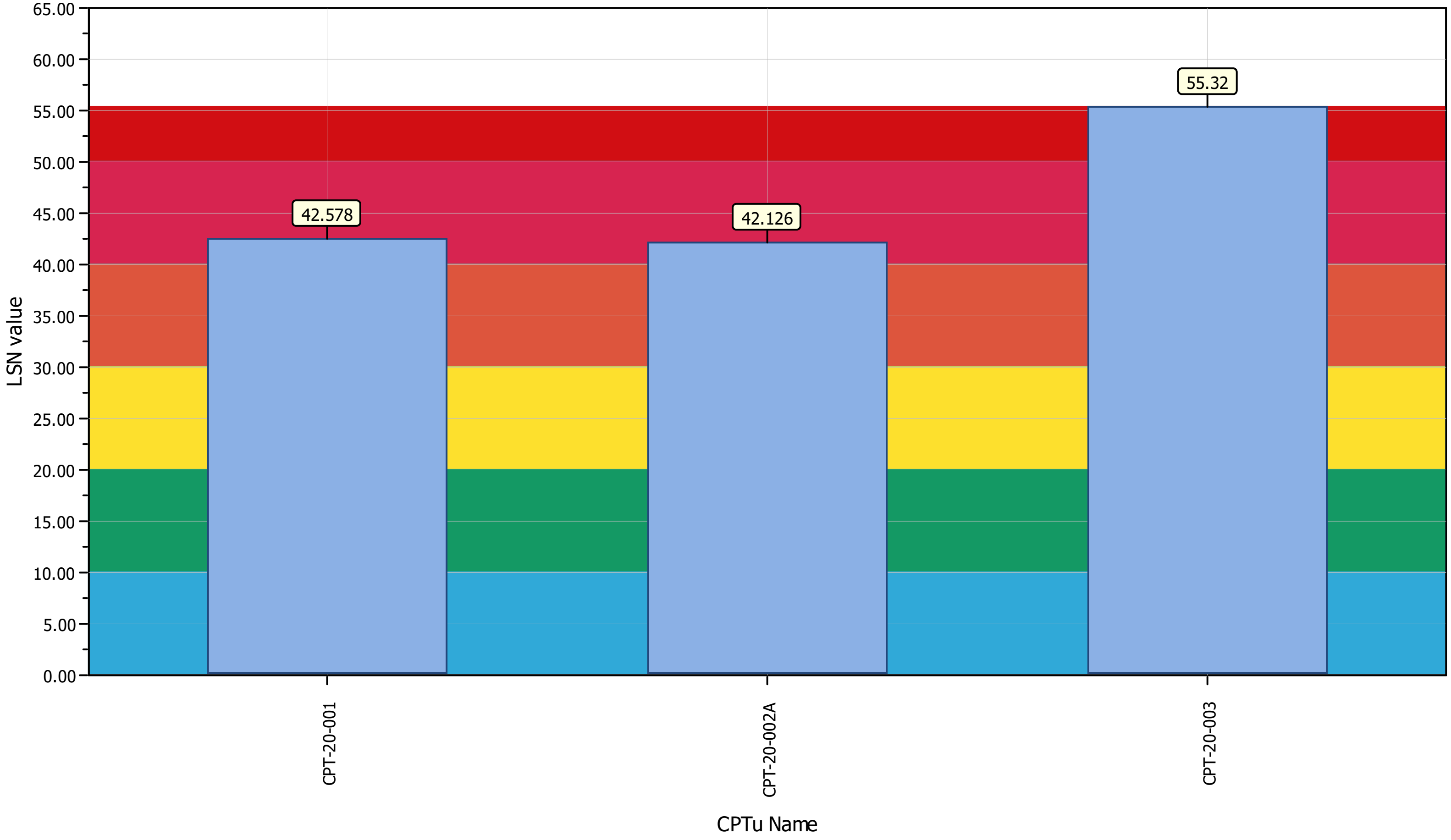
Total CPT number: 3

0% low risk

100% high risk

0% very high risk

Overall Liquefaction Severity Number report



LSN color scheme

- 🟥 Severe damage
- 🟥 Major expression of liquefaction
- 🟧 Moderate to severe exp. of liquefaction
- 🟨 Moderate expression of liquefaction
- 🟩 Minor expression of liquefaction
- 🟦 Little to no expression of liquefaction

Basic statistics

Total CPT number: 3

0% little liquefaction

0% minor liquefaction

0% moderate liquefaction

0% moderate to major liquefaction

67% major liquefaction

33% severe liquefaction

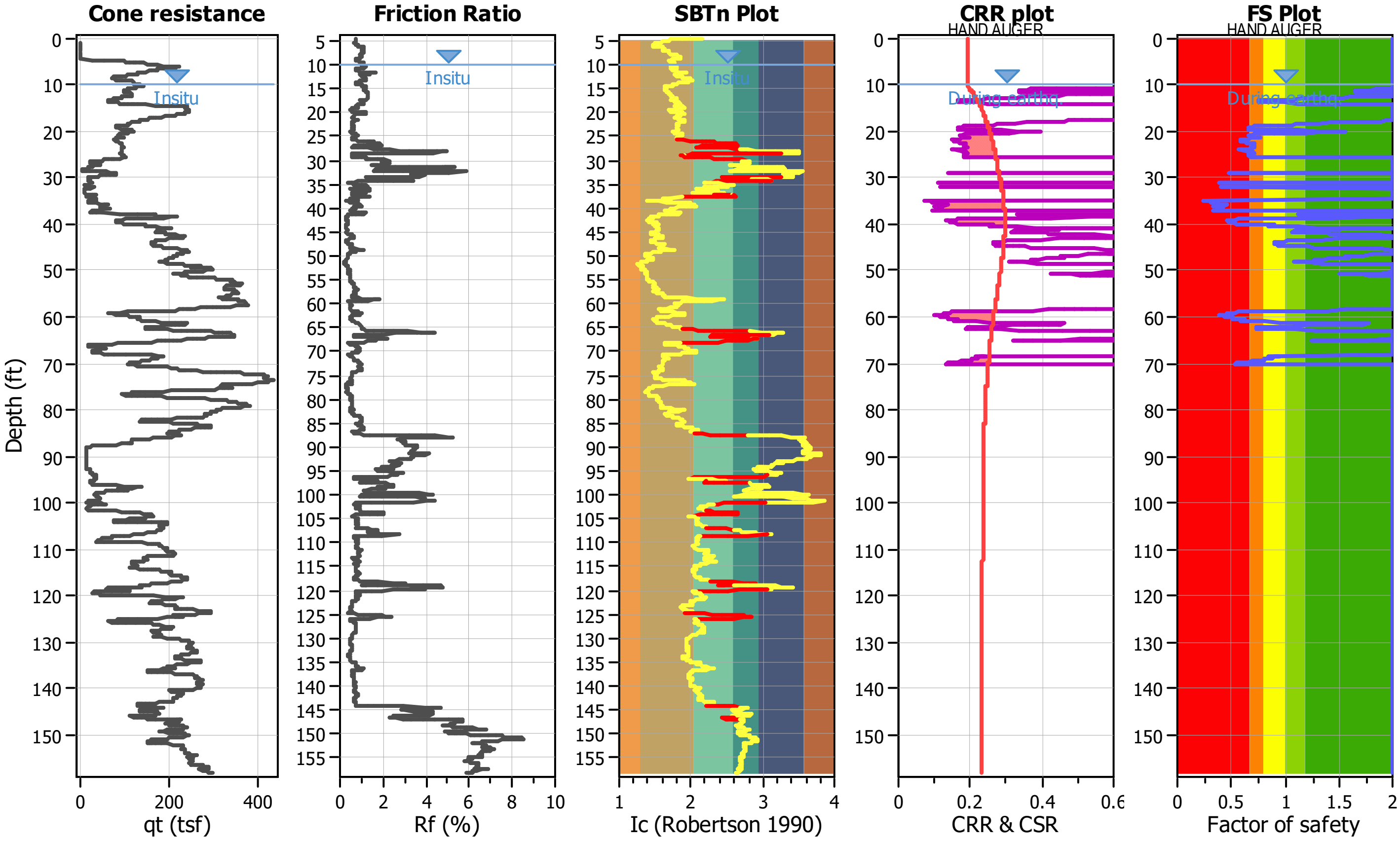
LIQUEFACTION ANALYSIS REPORT

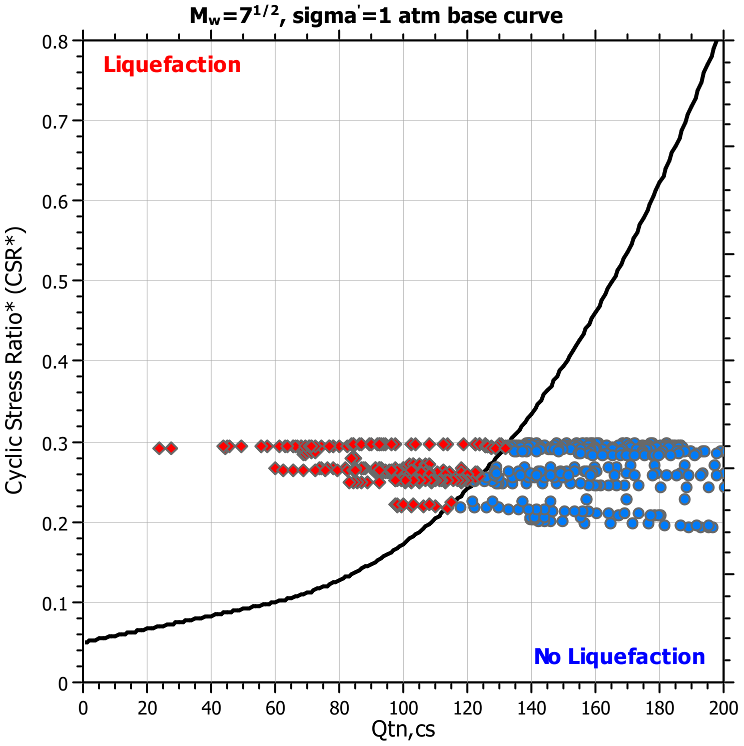
Project title : Camino Del Mar Bridge Replacement **Location : Del Mar, CA**

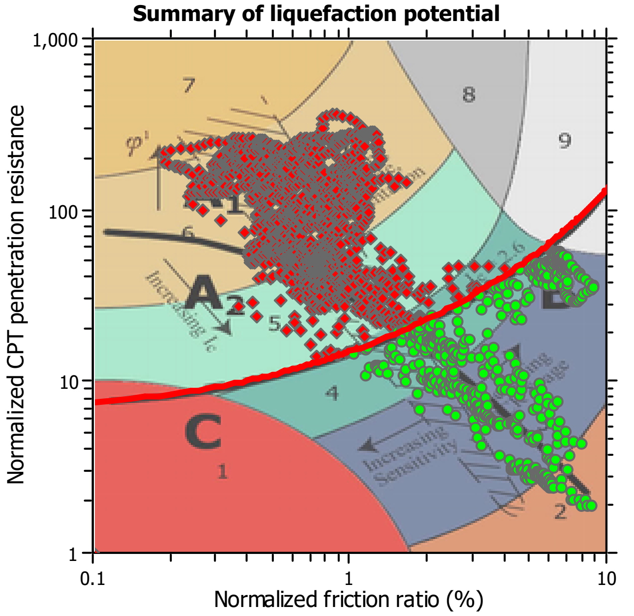
CPT file : CPT-20-001

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	70.00 ft
Earthquake magnitude M_w:	6.63	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.41	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

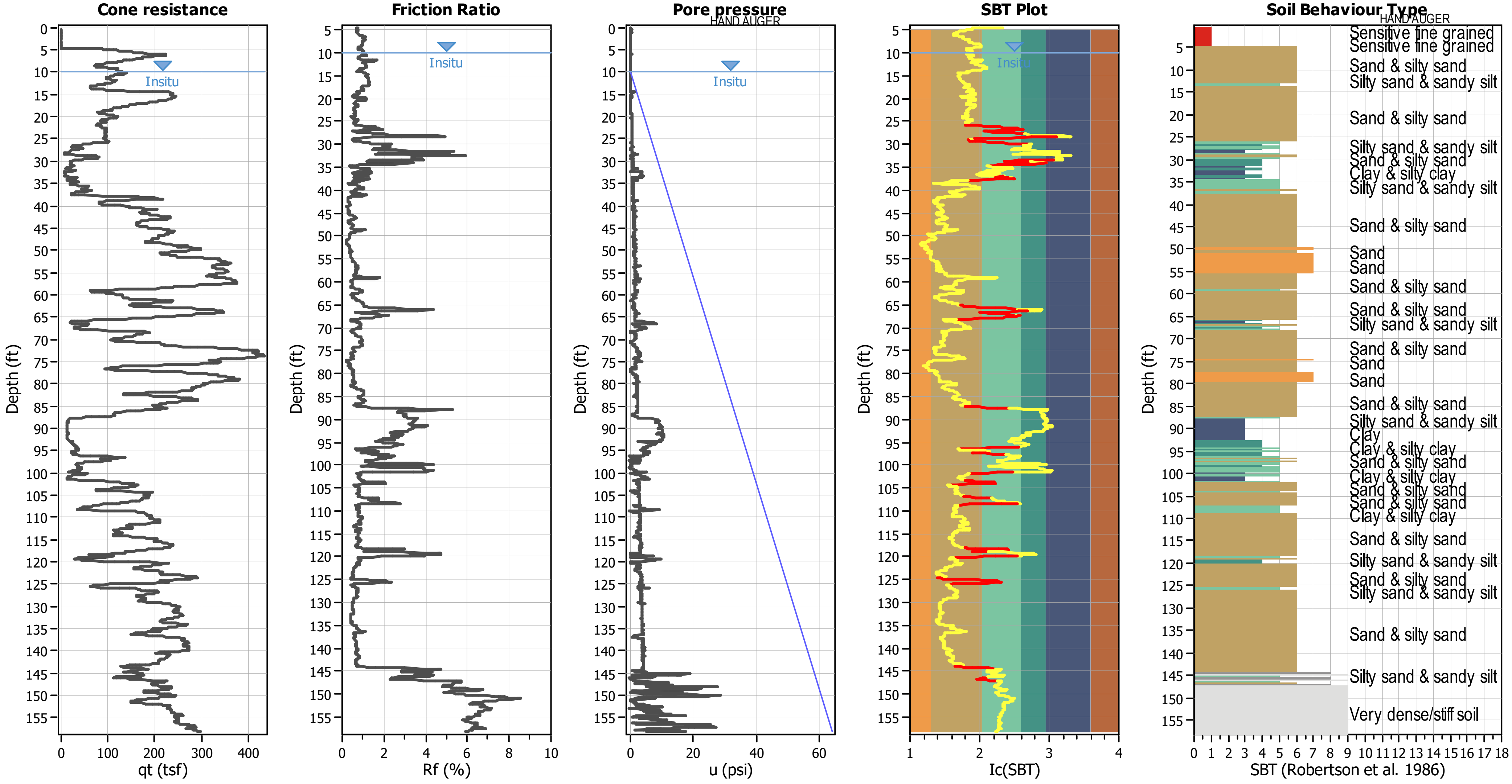

Cone resistance **Friction Ratio** **SBTn Plot** **CRR plot** **FS Plot**


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


Summary of liquefaction potential

Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.63
Peak ground acceleration: 0.41
Depth to water table (insitu): 10.00 ft

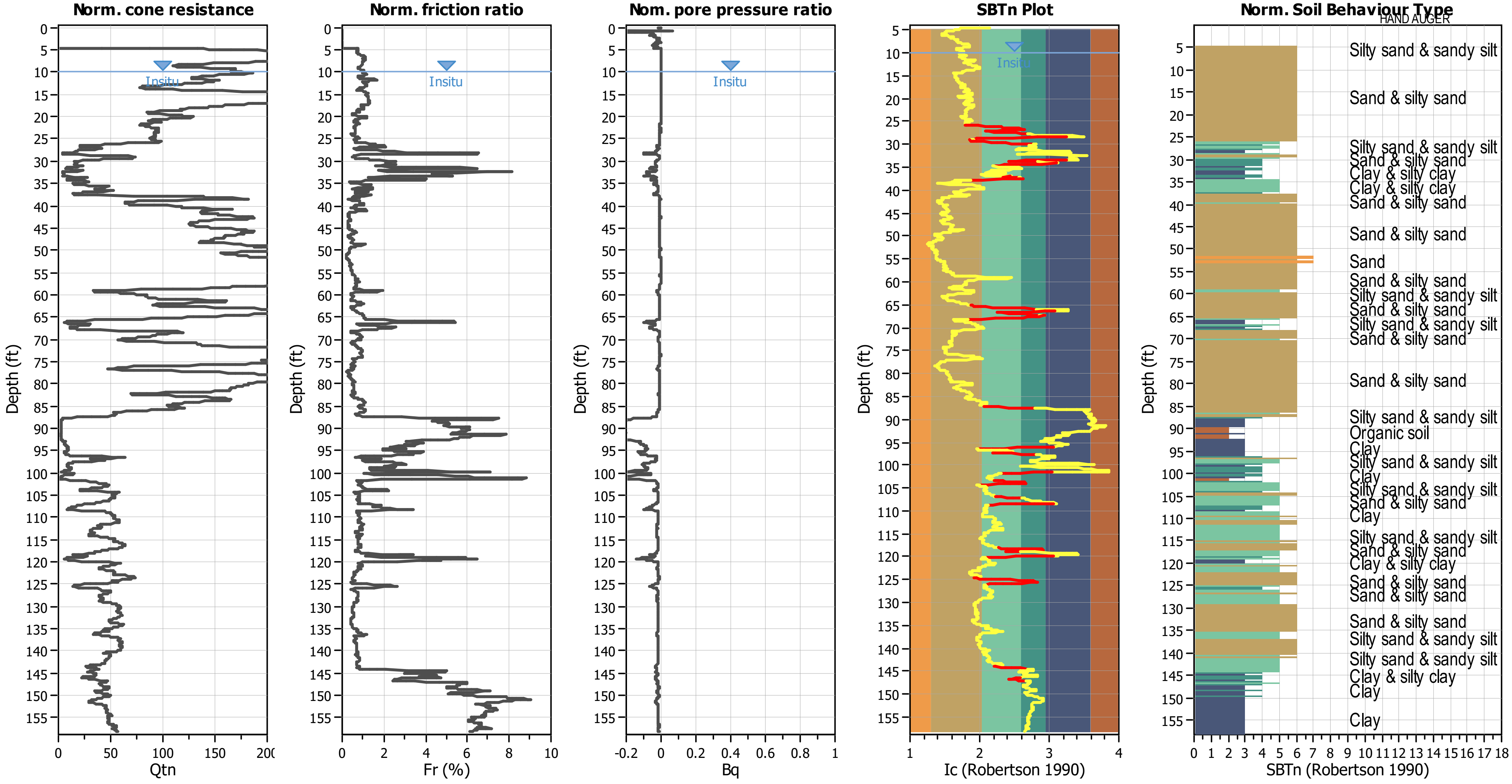
Depth to water table (erthq.): 10.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: Yes
Limit depth: 70.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.63
Peak ground acceleration: 0.41
Depth to water table (insitu): 10.00 ft

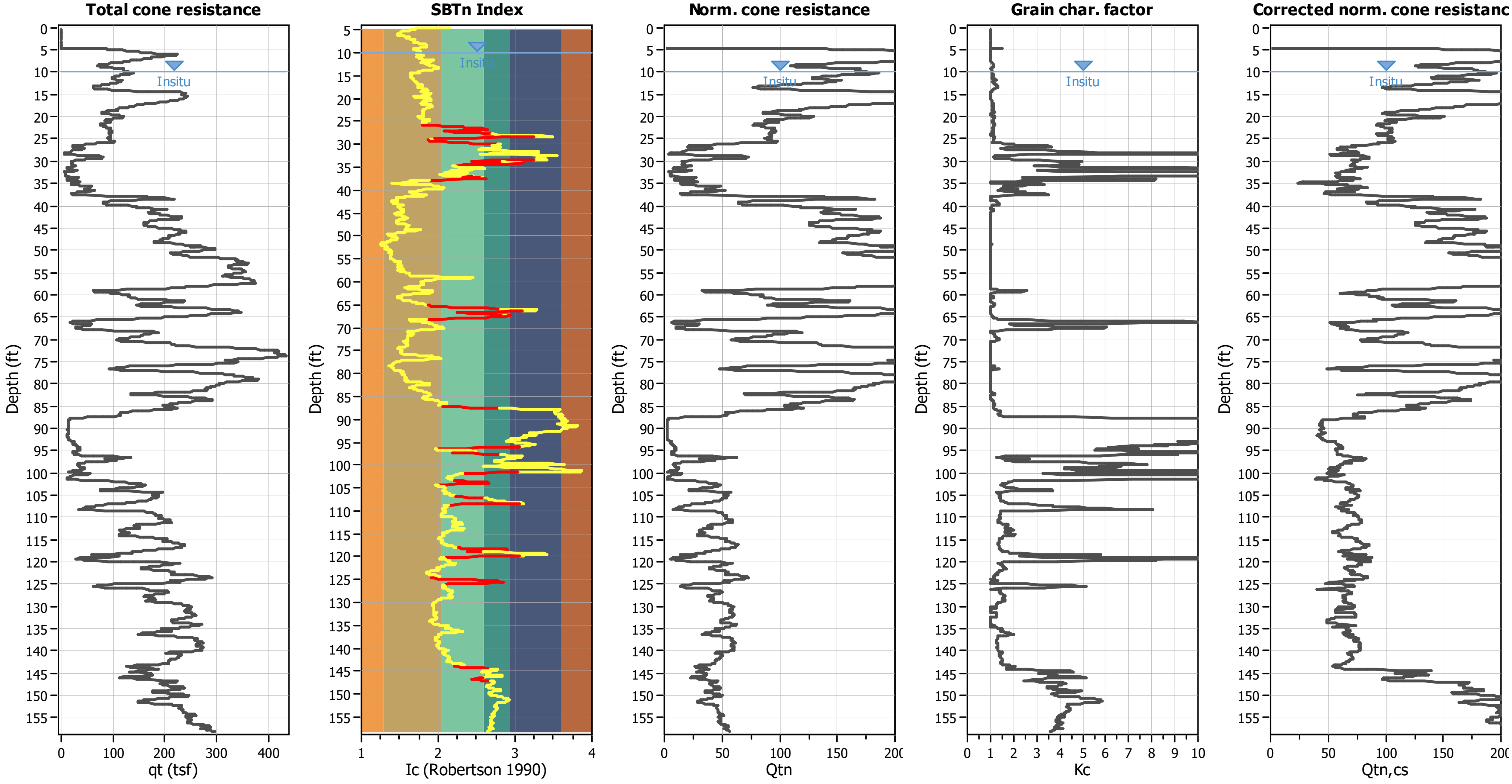
Depth to water table (erthq.): 10.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: Yes
Limit depth: 70.00 ft

SBTn legend

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

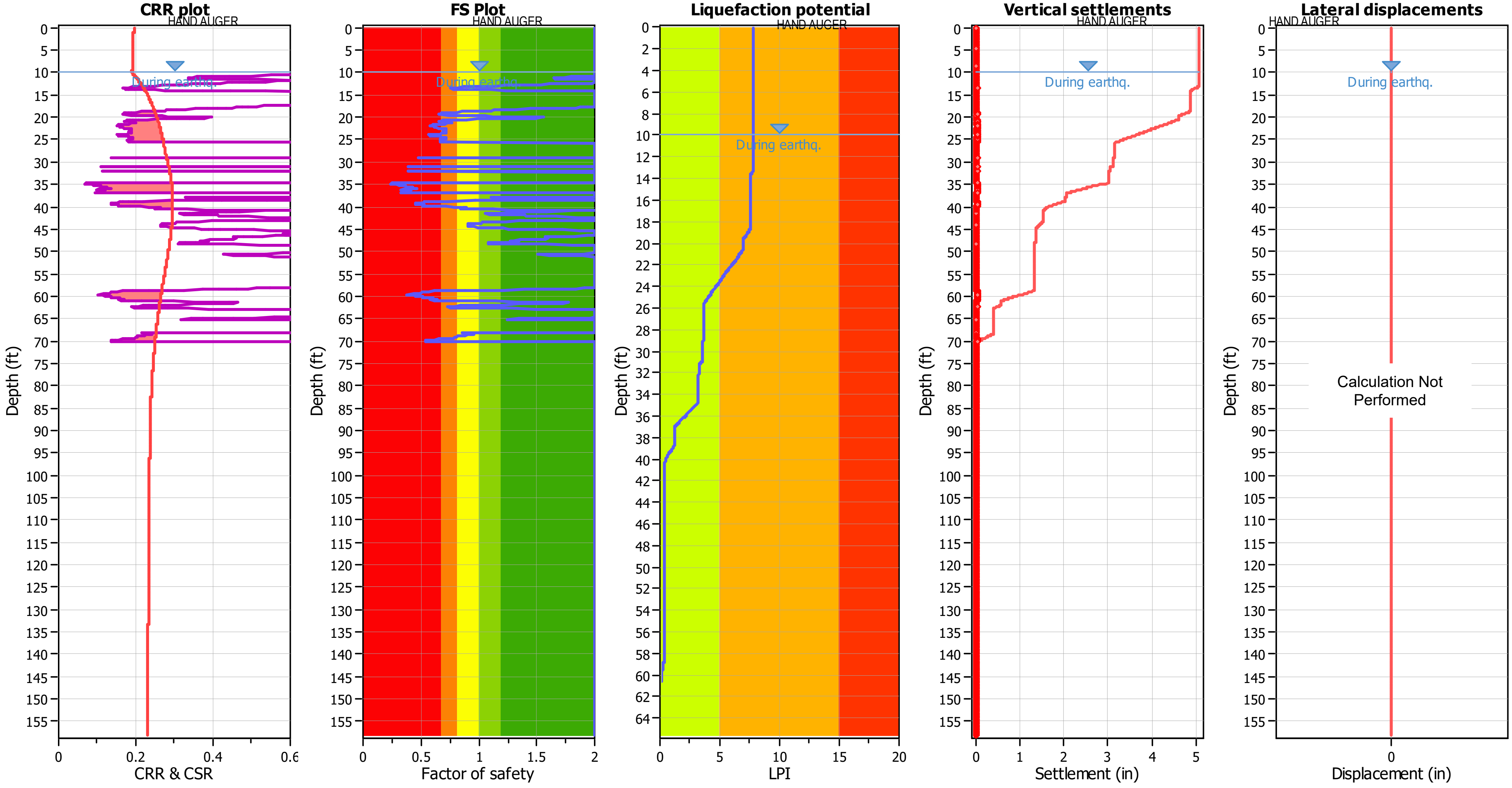
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

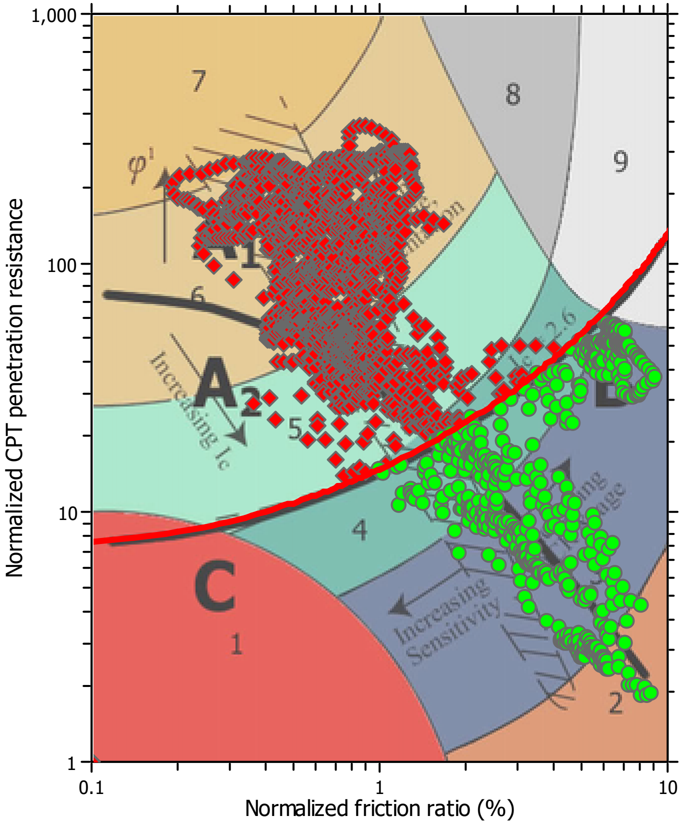
F.S. color scheme

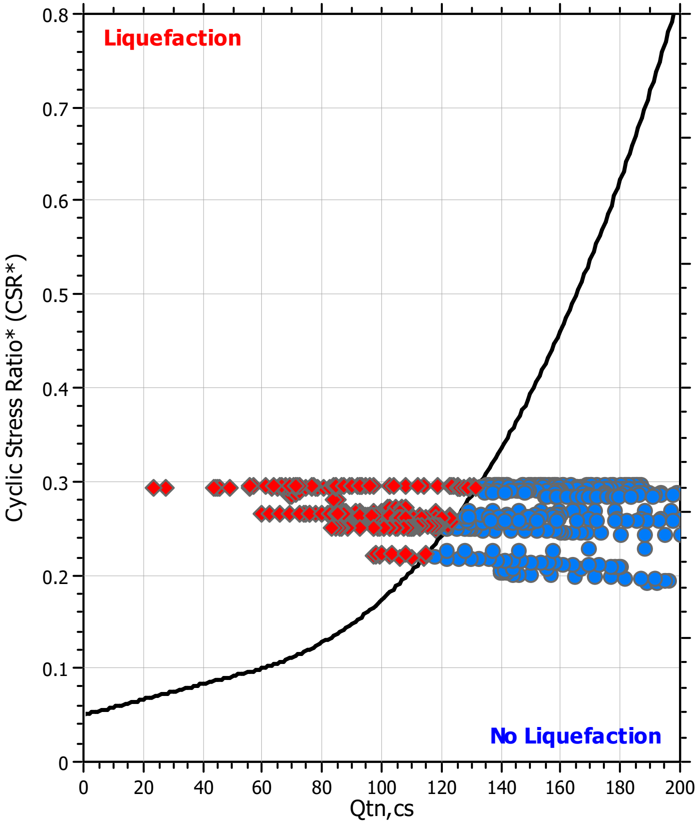
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

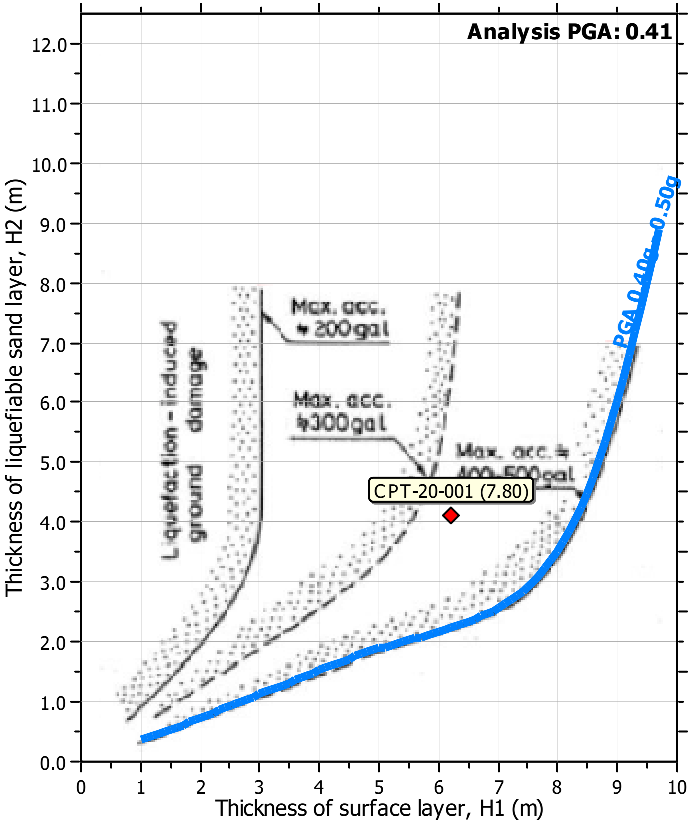
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



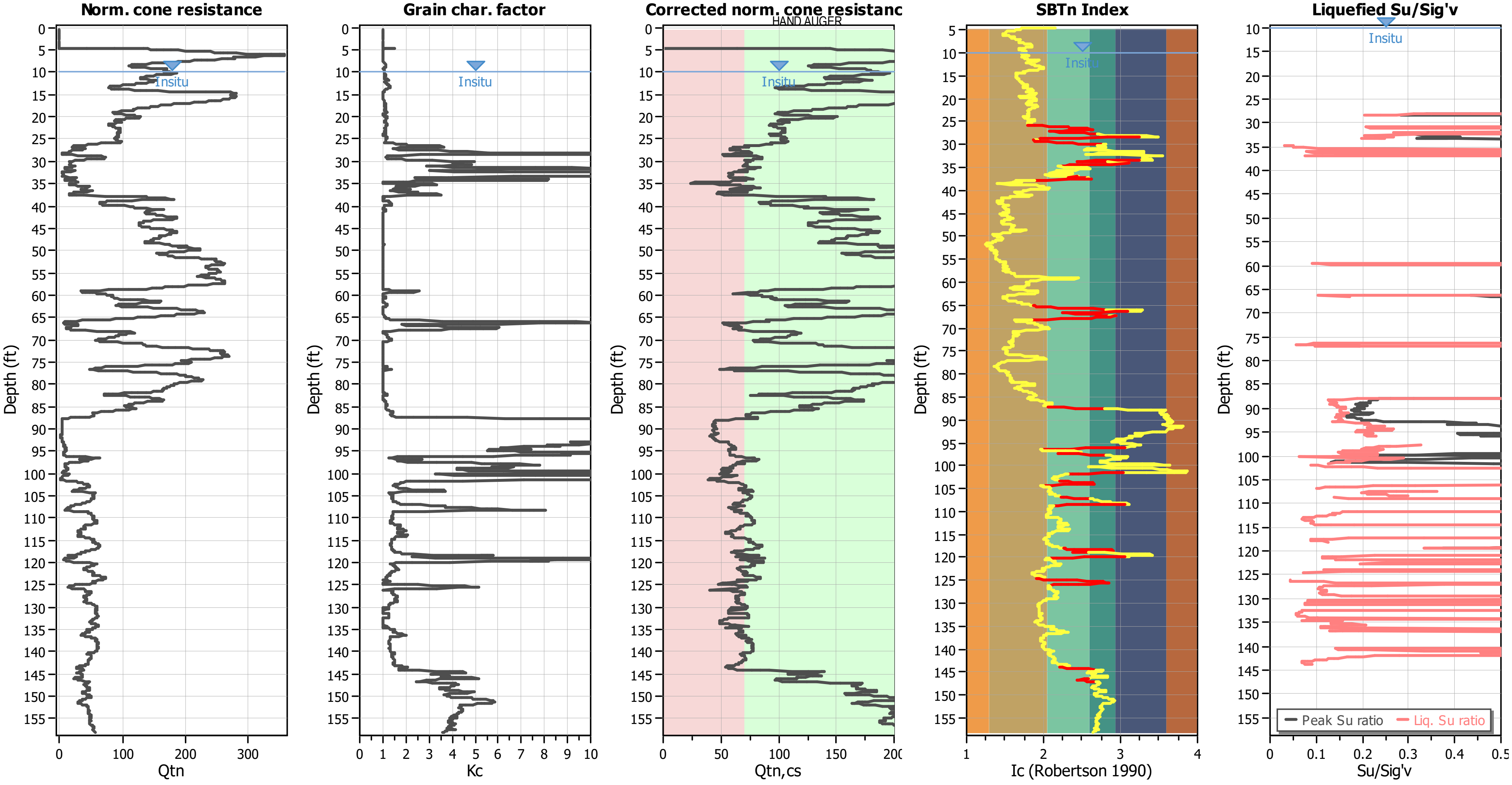




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.63
Peak ground acceleration: 0.41
Depth to water table (insitu): 10.00 ft

Depth to water table (erthq.): 10.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

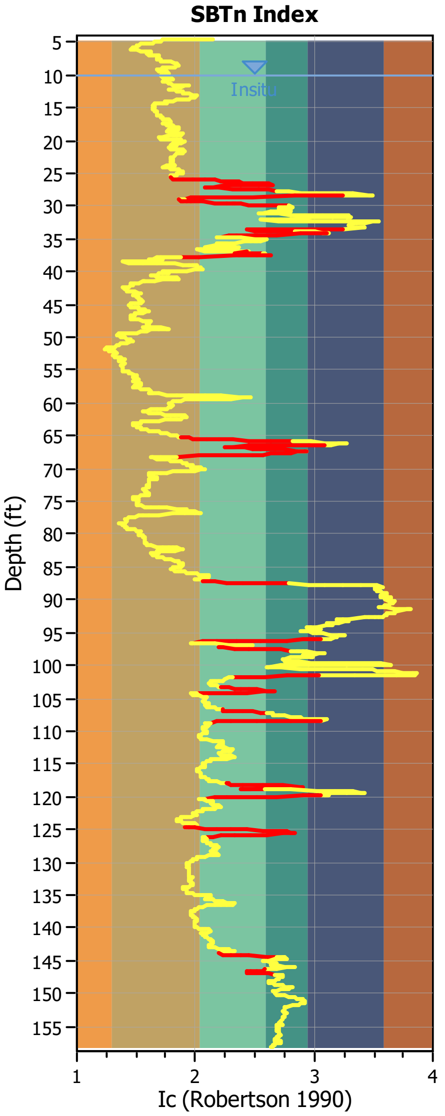
Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: Yes
Limit depth: 70.00 ft

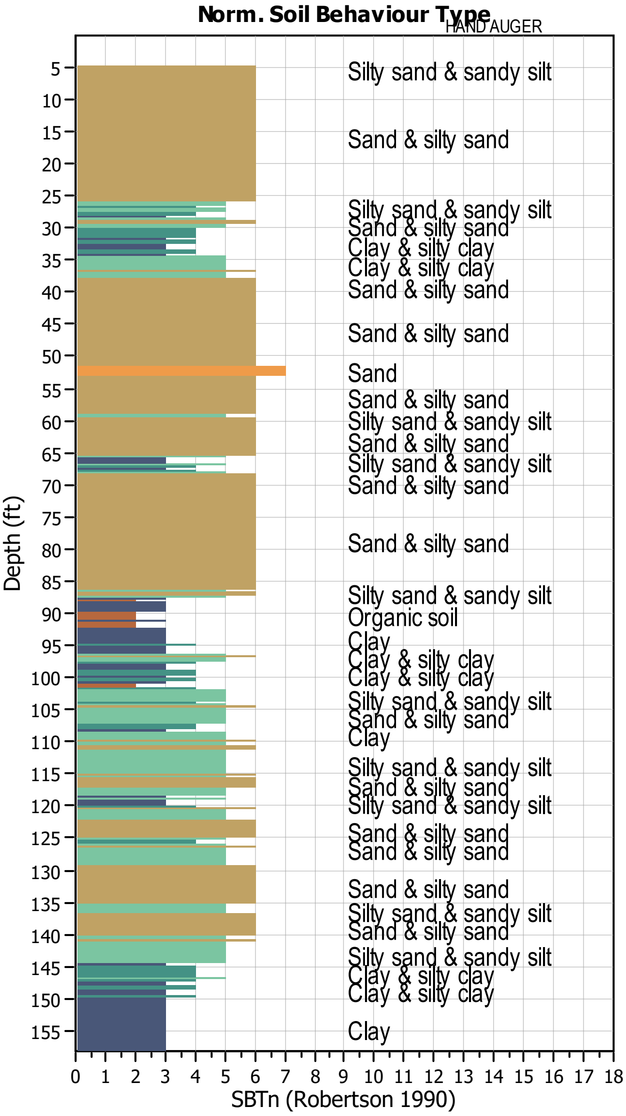
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between 1.80 < I_c < 3.0) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





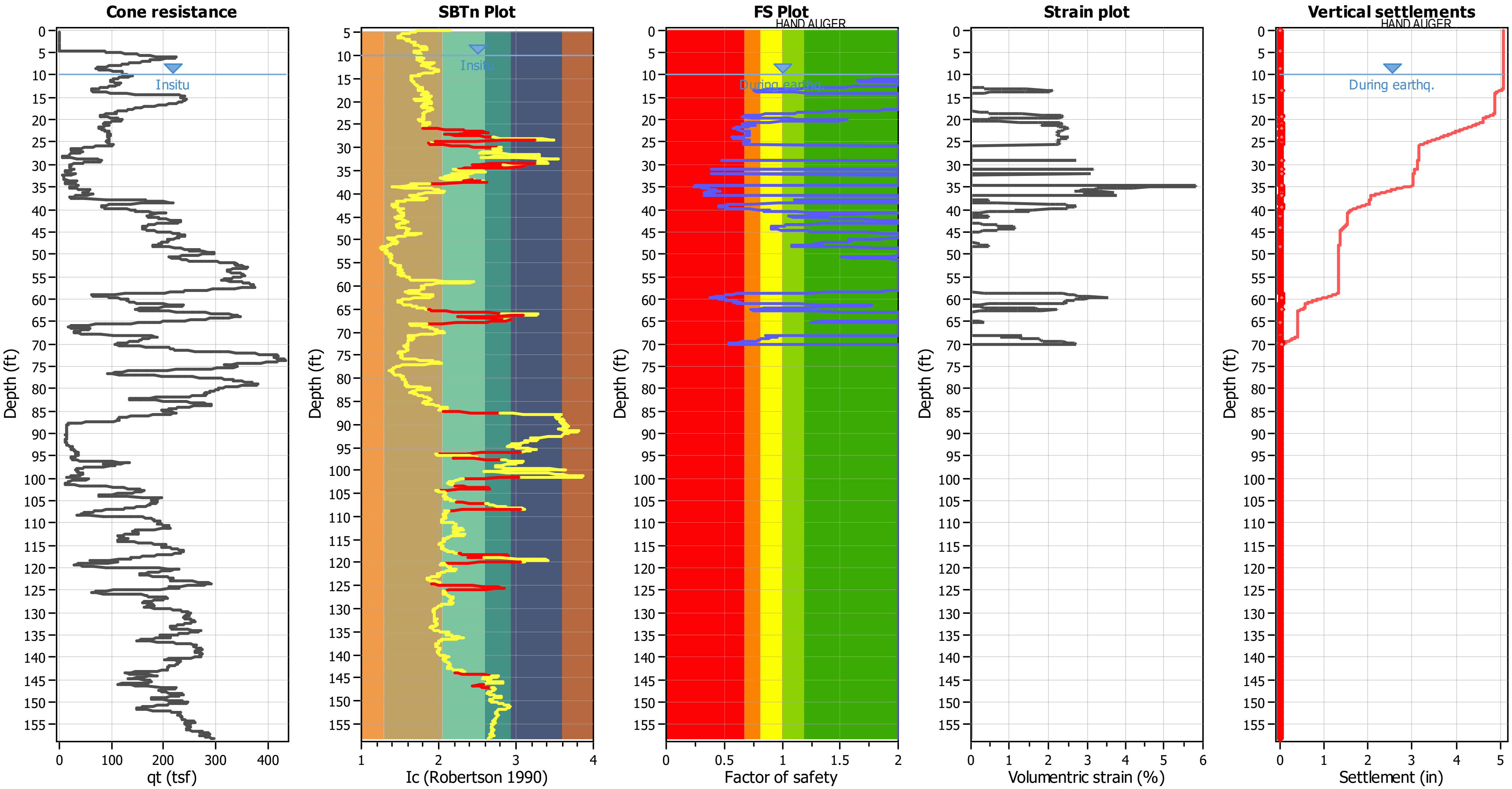
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	2411
Total points excluded:	254
Exclusion percentage:	10.54%
Number of layers detected:	31

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

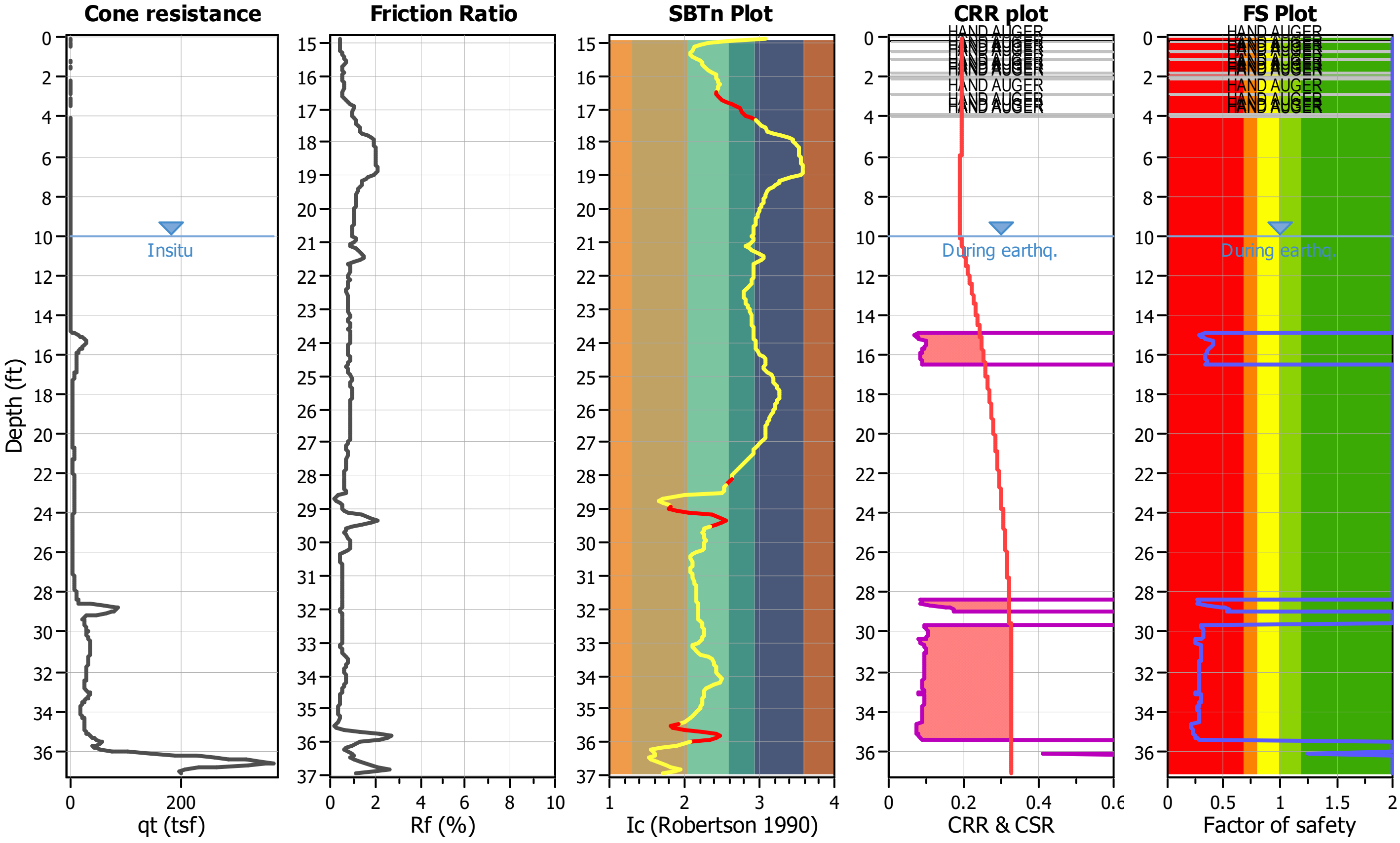
LIQUEFACTION ANALYSIS REPORT

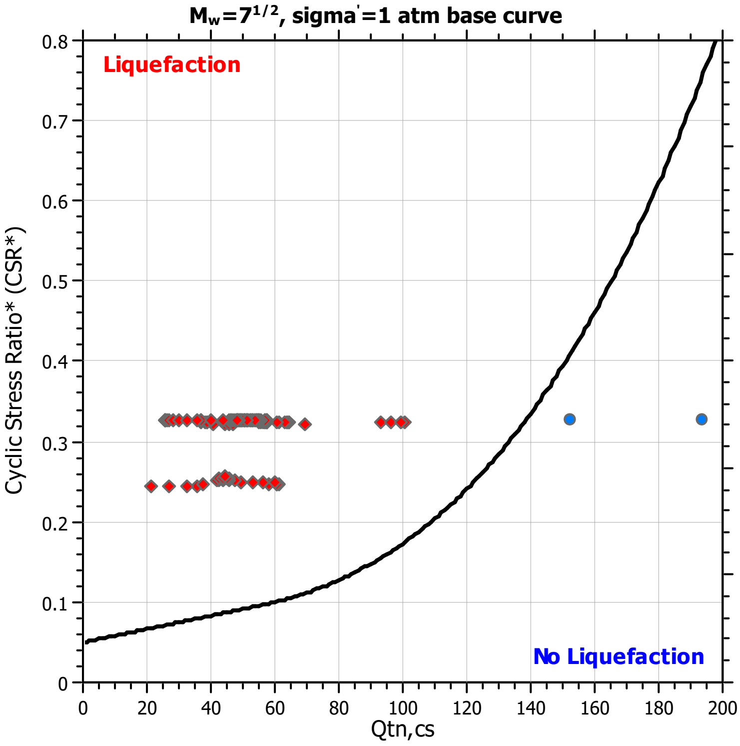
Project title : Camino Del Mar Bridge Replacement **Location : Del Mar, CA**

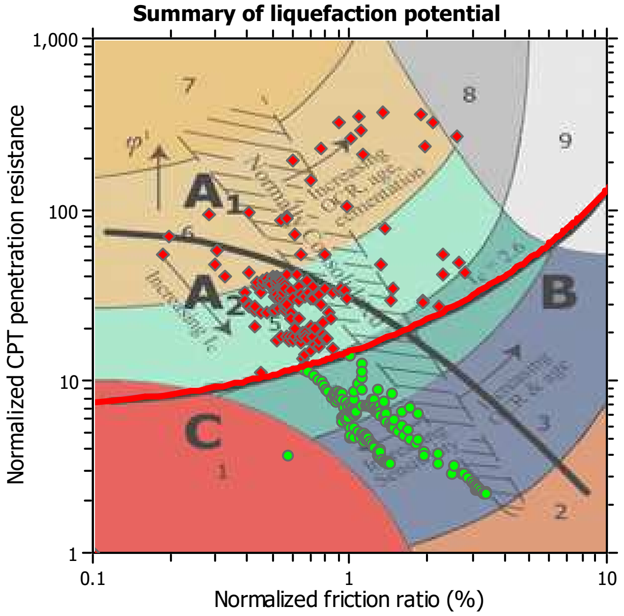
CPT file : CPT-20-002A

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	70.00 ft
Earthquake magnitude M_w:	6.63	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.41	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

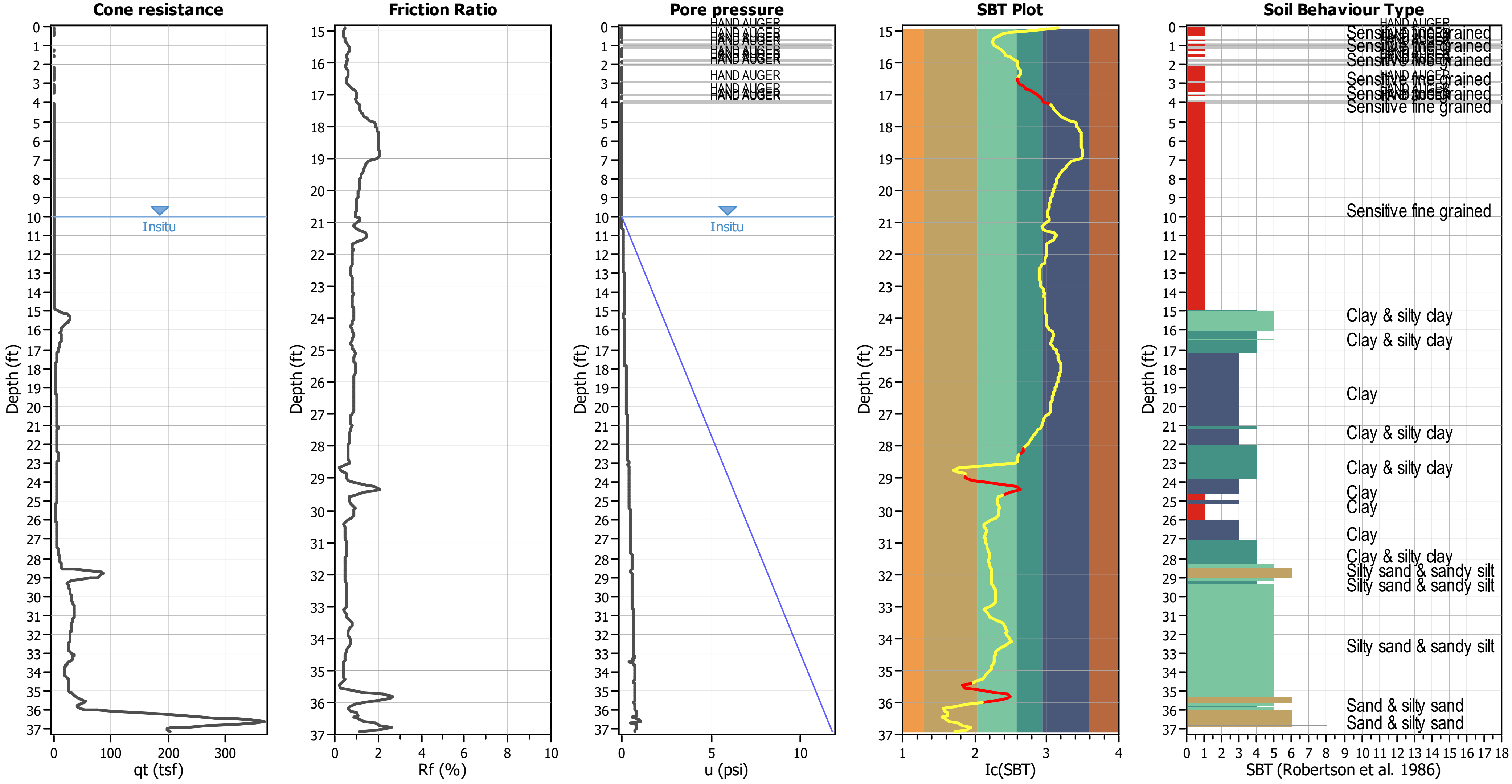






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



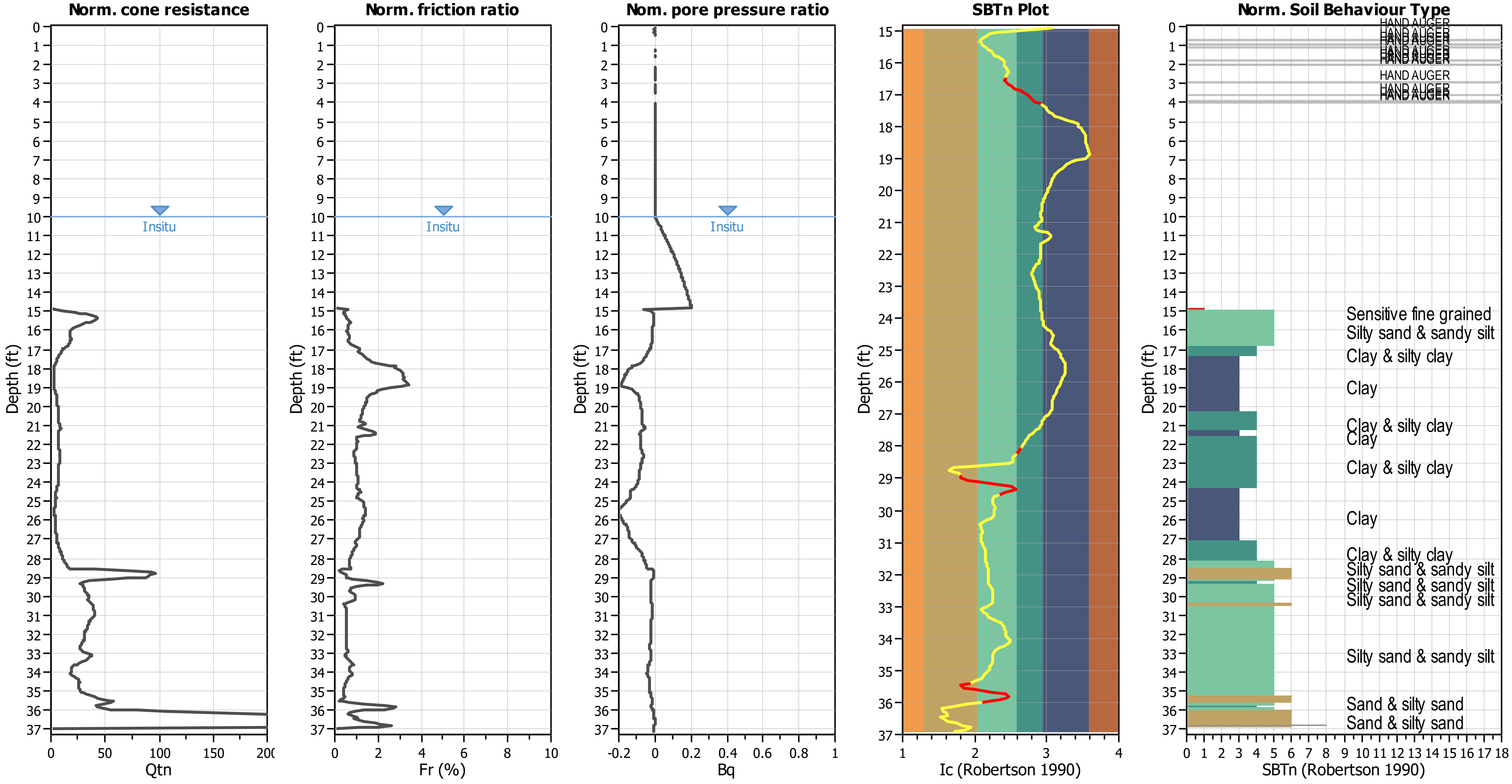
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



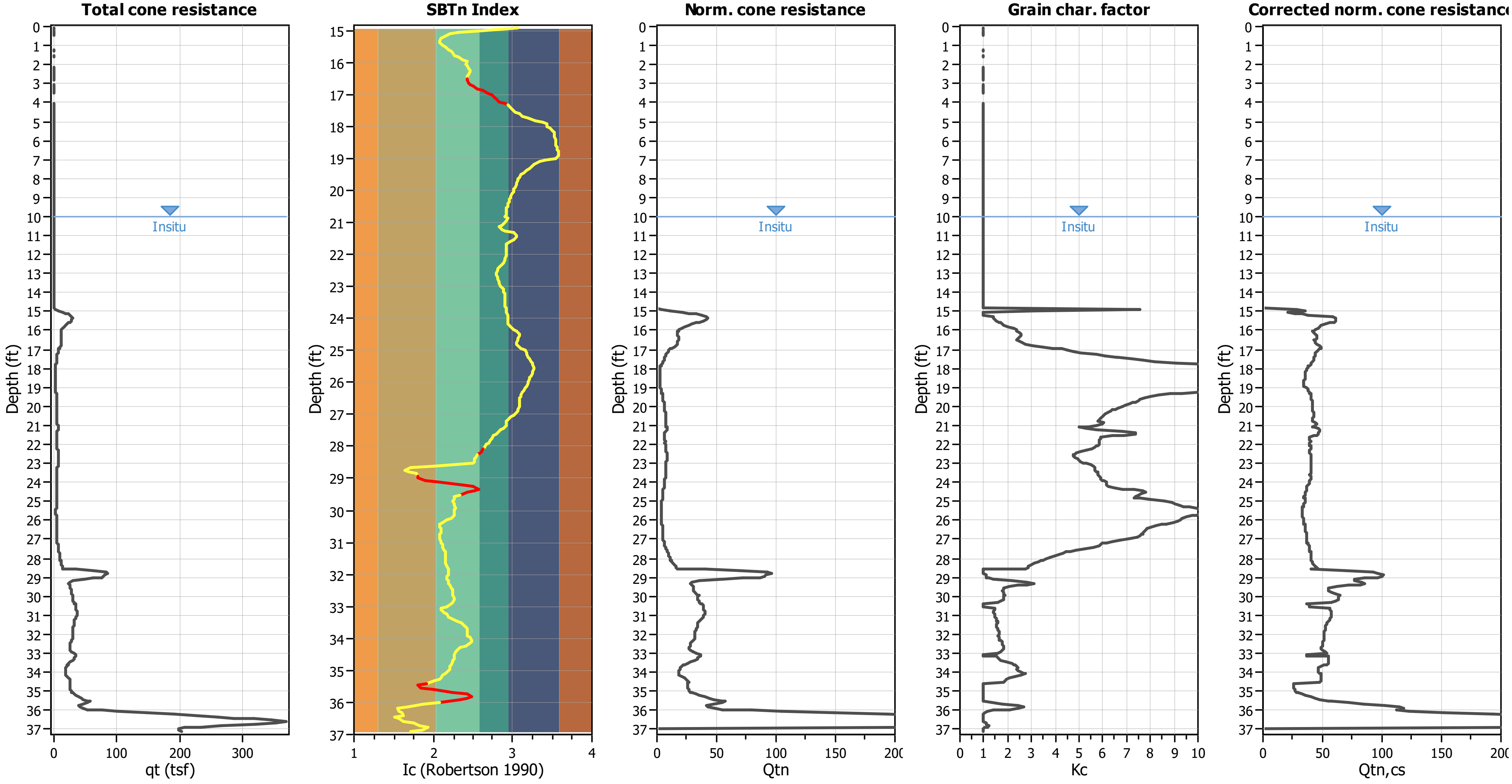
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

SBTn legend

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

Liquefaction analysis overall plots (intermediate results)



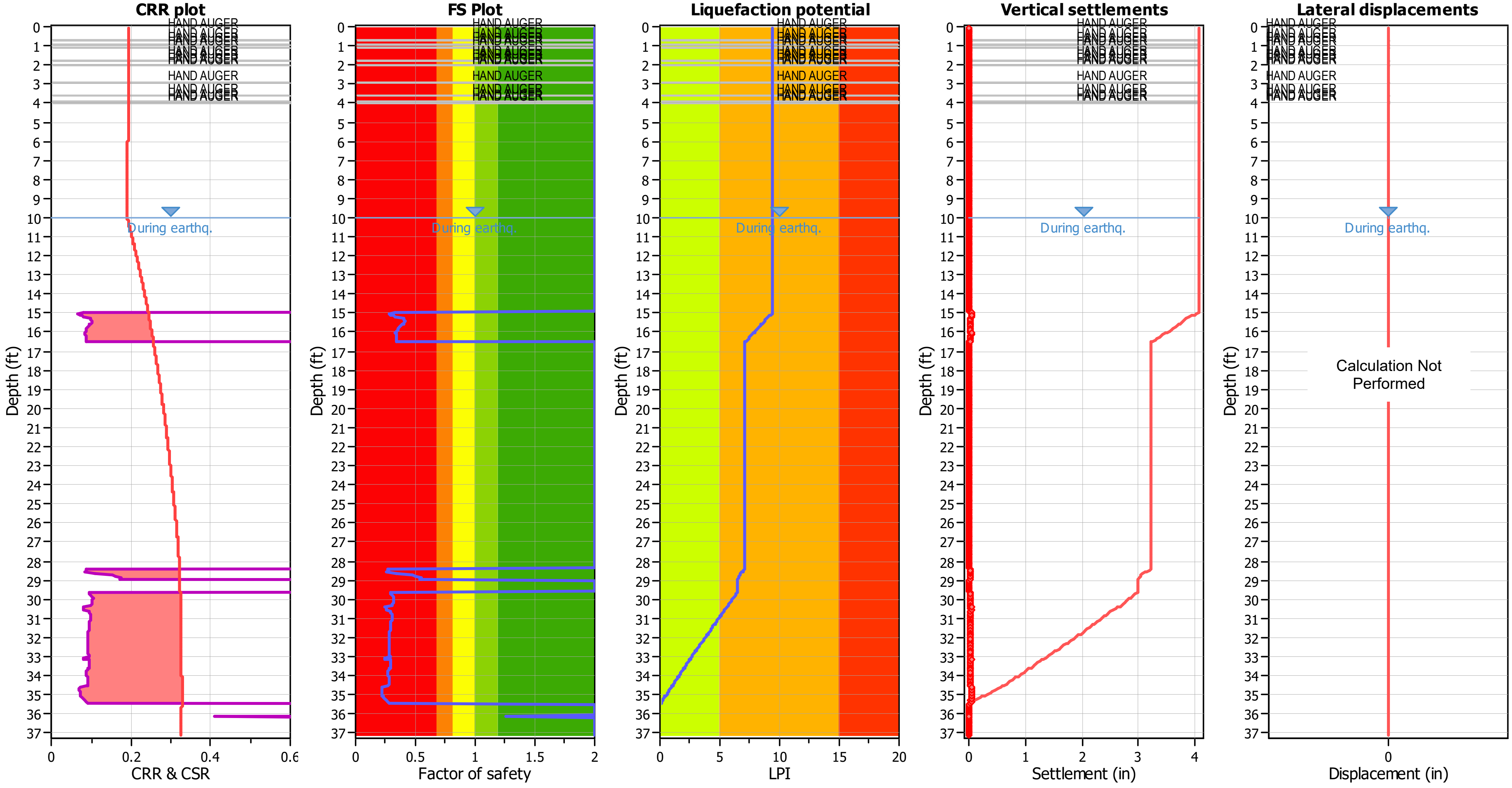
Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.63
Peak ground acceleration: 0.41
Depth to water table (insitu): 10.00 ft

Depth to water table (erthq.): 10.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_{σ} applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: Yes
Limit depth: 70.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

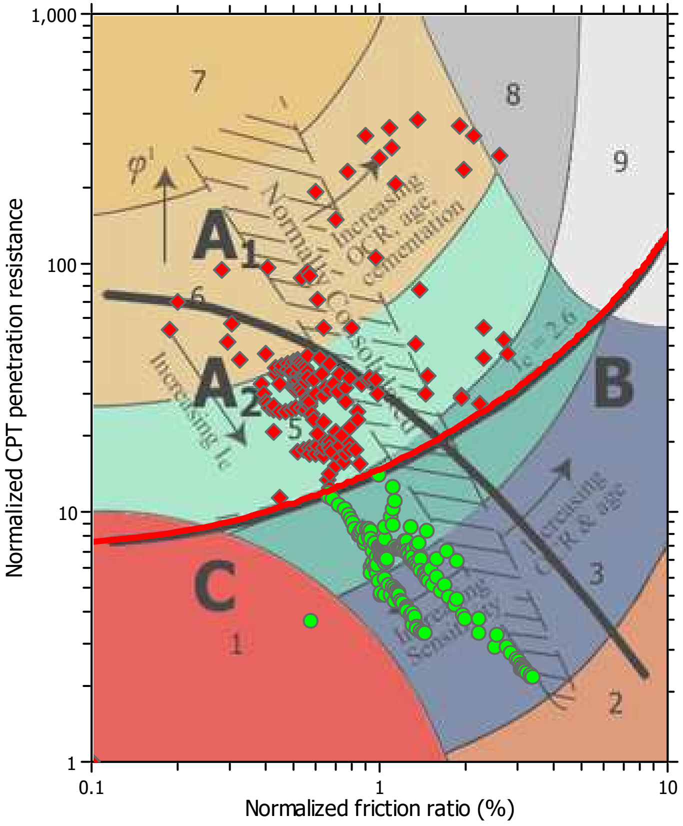
F.S. color scheme

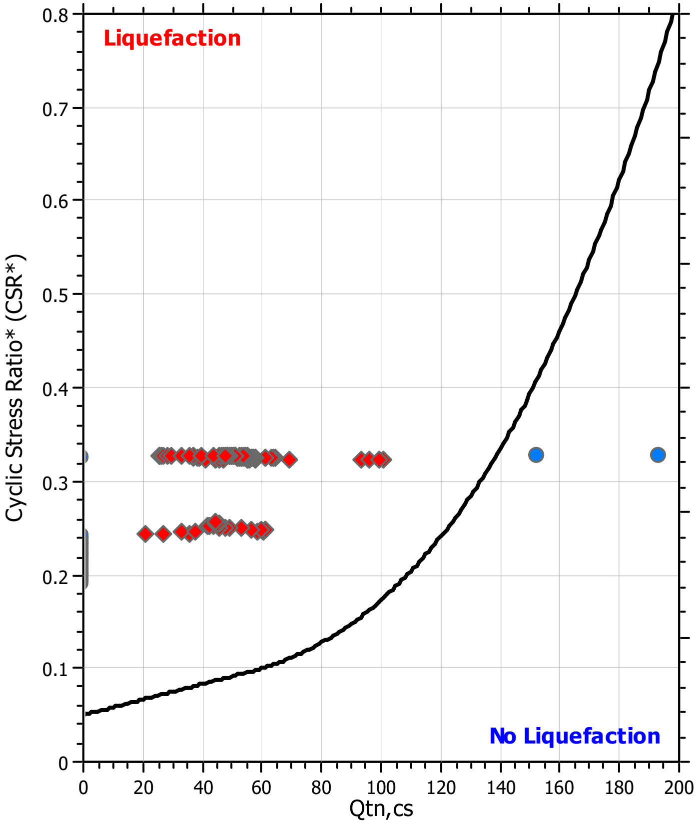
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

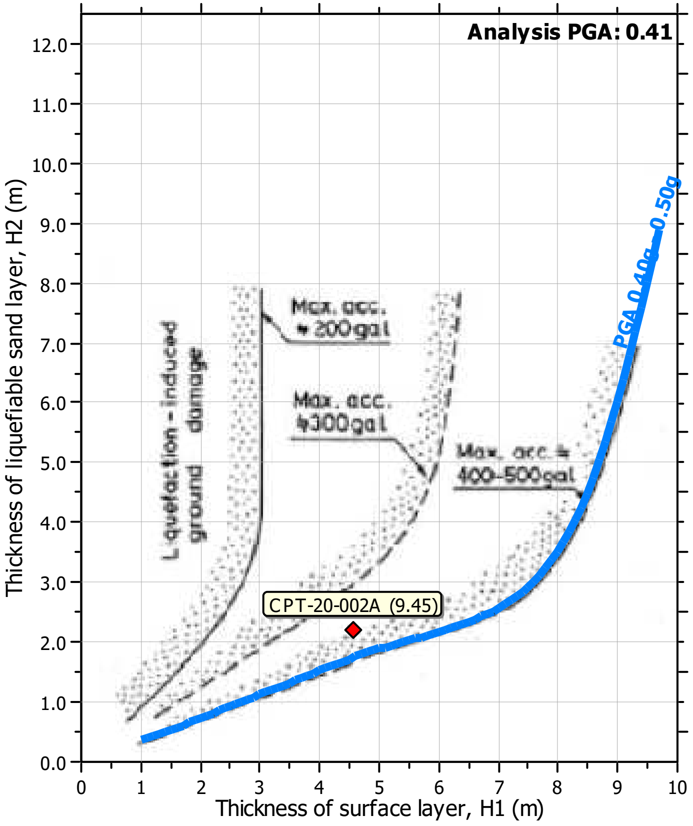
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



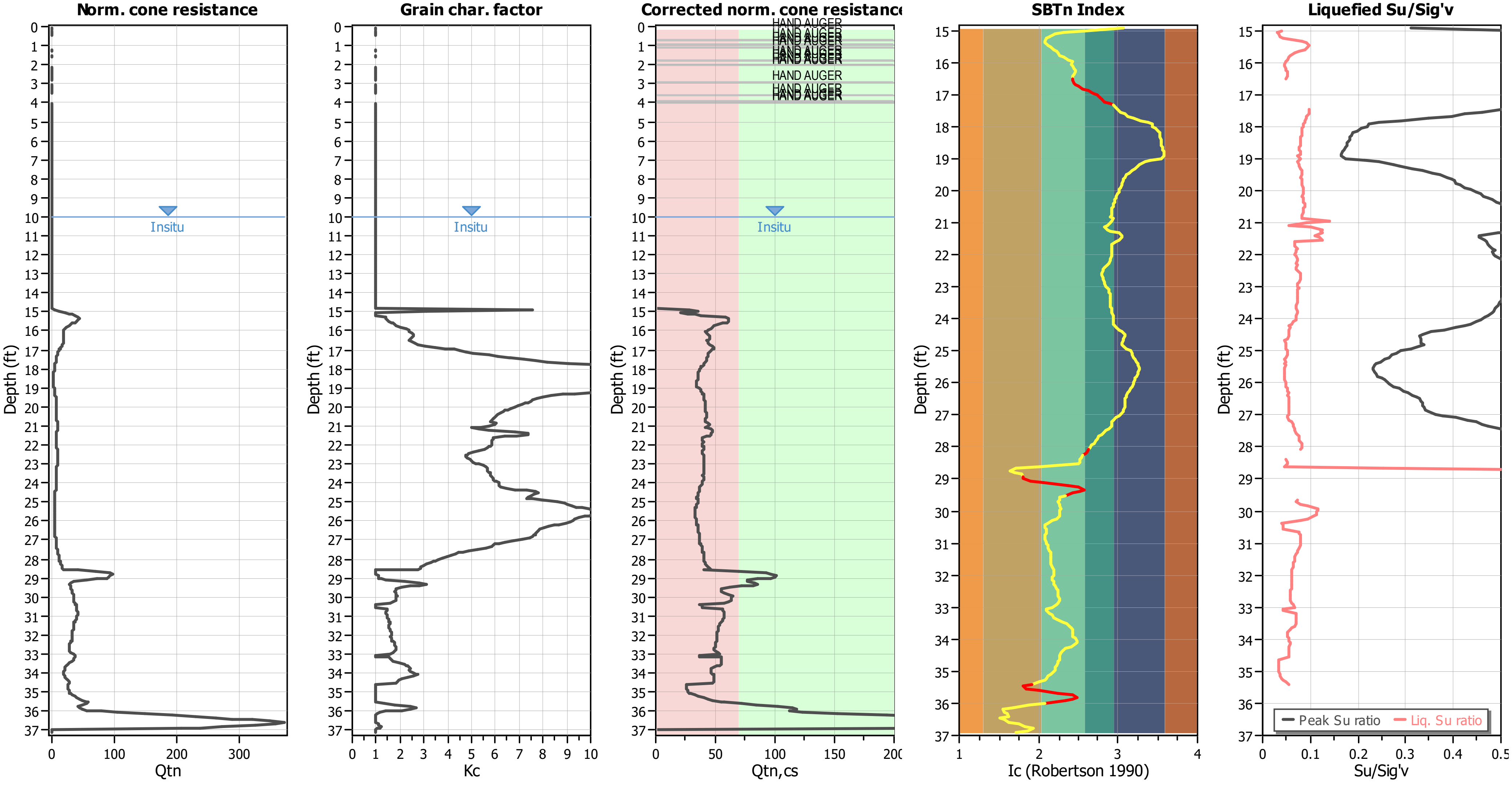




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

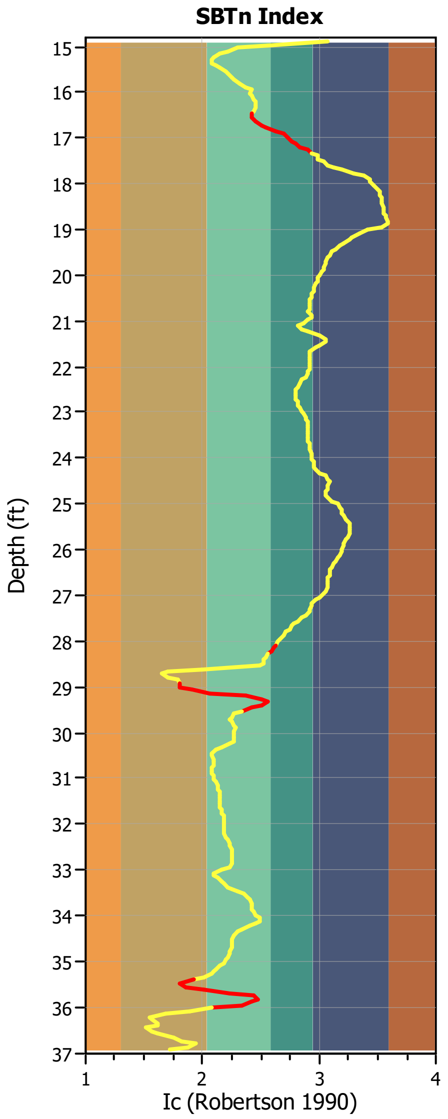
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

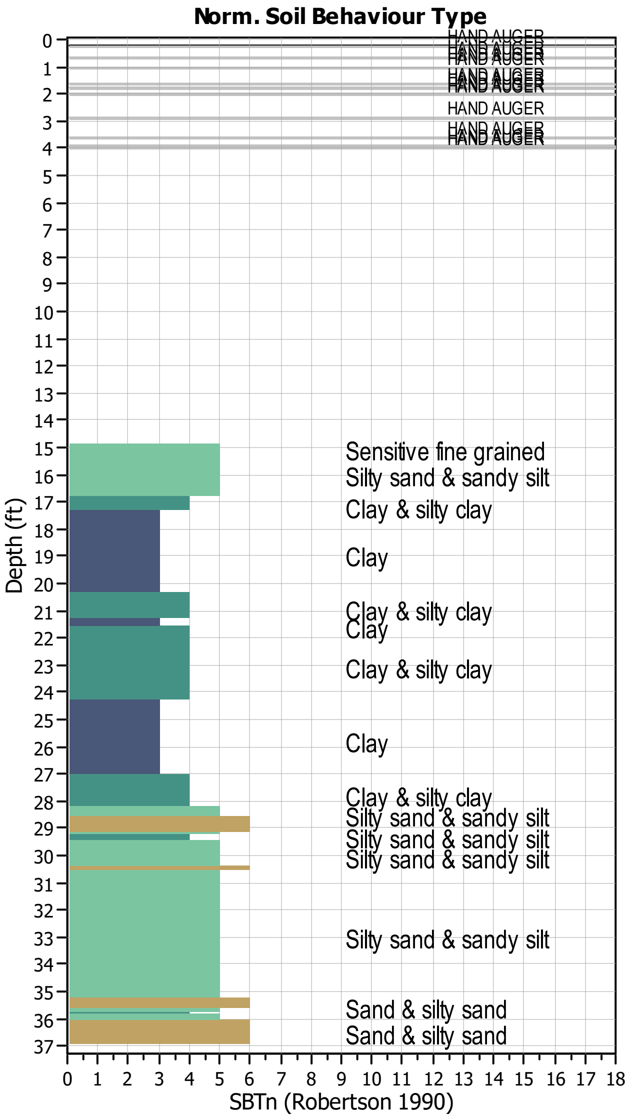
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





Transition layer algorithm properties

I_c minimum check value: 1.70

I_c maximum check value: 3.00

I_c change ratio value: 0.0250

Minimum number of points in layer: 4

General statistics

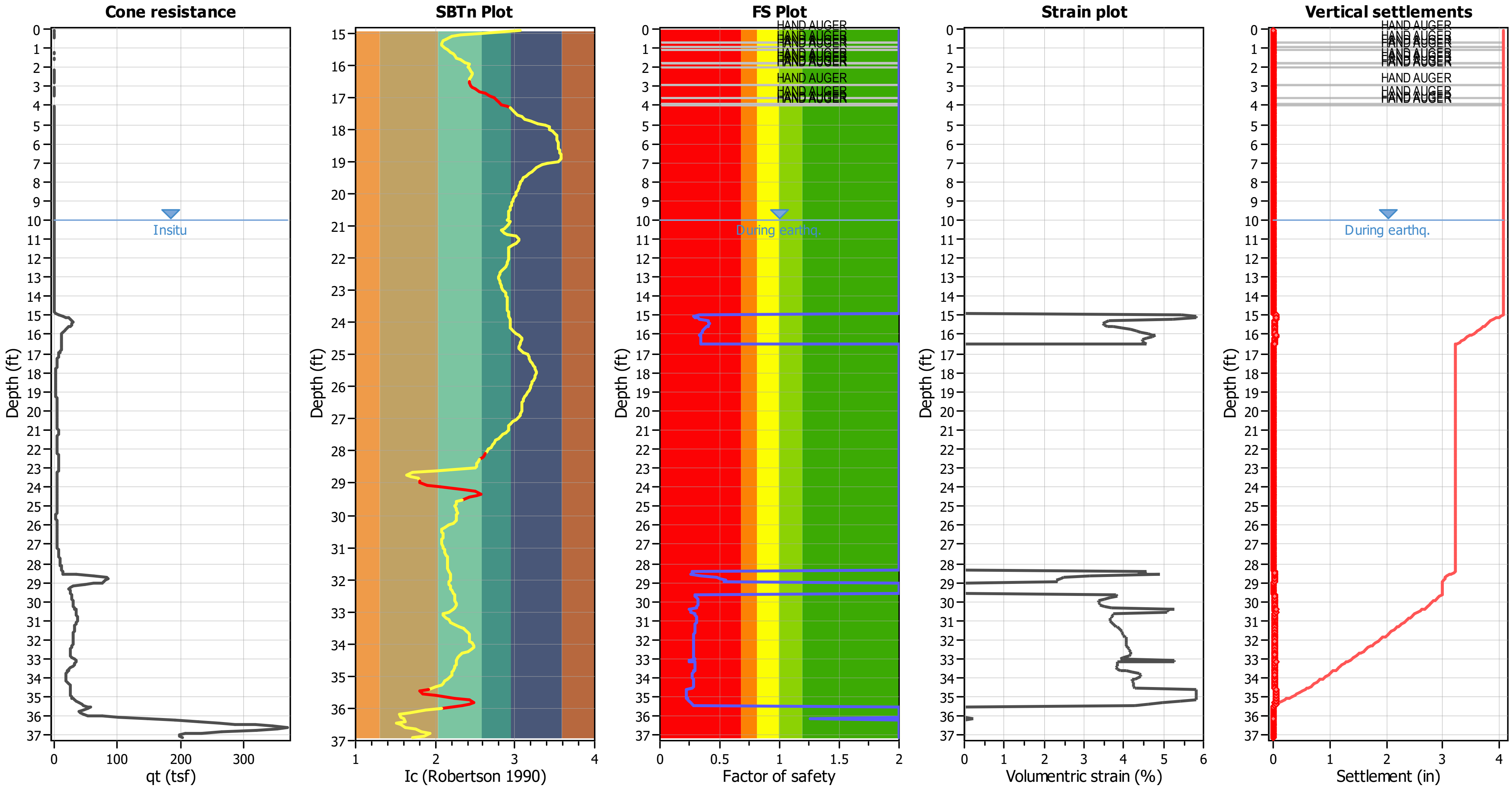
Total points in CPT file: 566

Total points excluded: 40

Exclusion percentage: 7.07%

Number of layers detected: 6

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

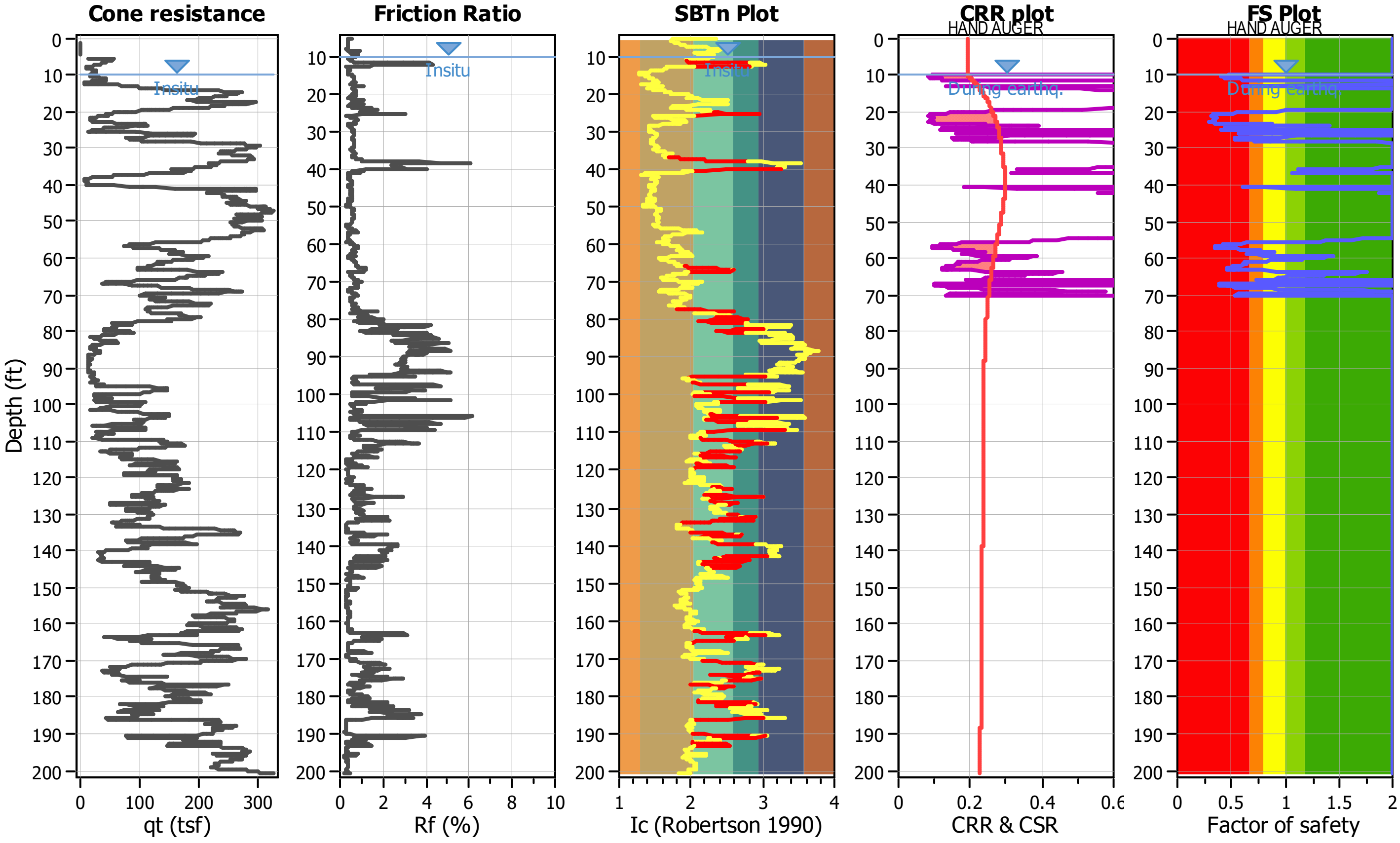
LIQUEFACTION ANALYSIS REPORT

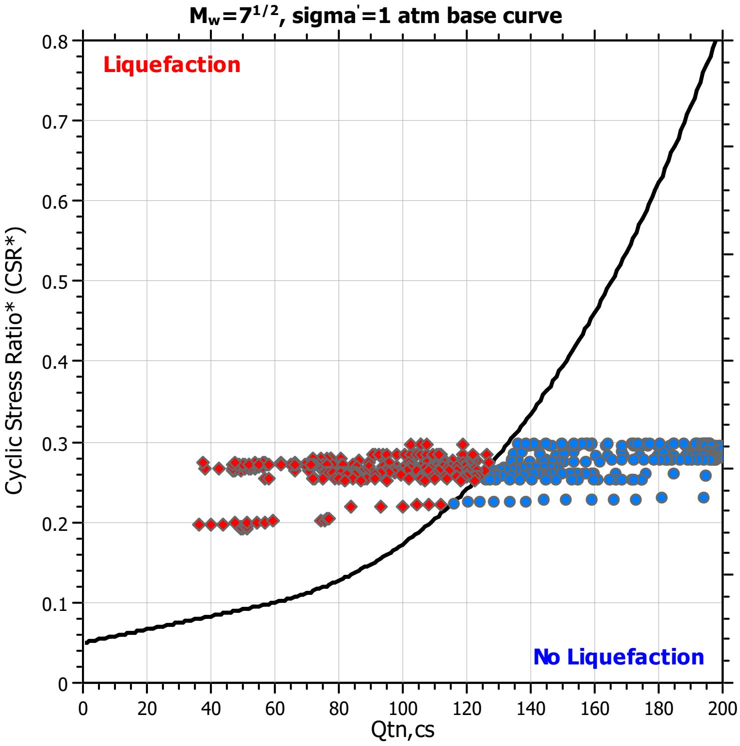
Project title : Camino Del Mar Bridge Replacement **Location : Del Mar, CA**

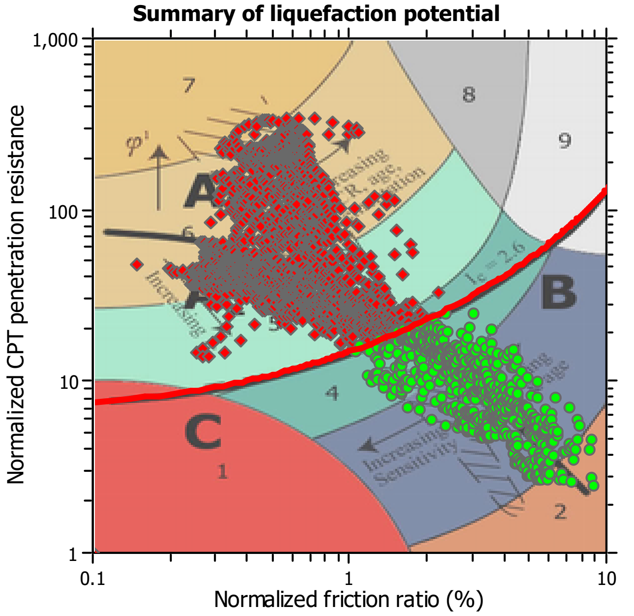
CPT file : CPT-20-003

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	70.00 ft
Earthquake magnitude M_w:	6.63	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.41	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







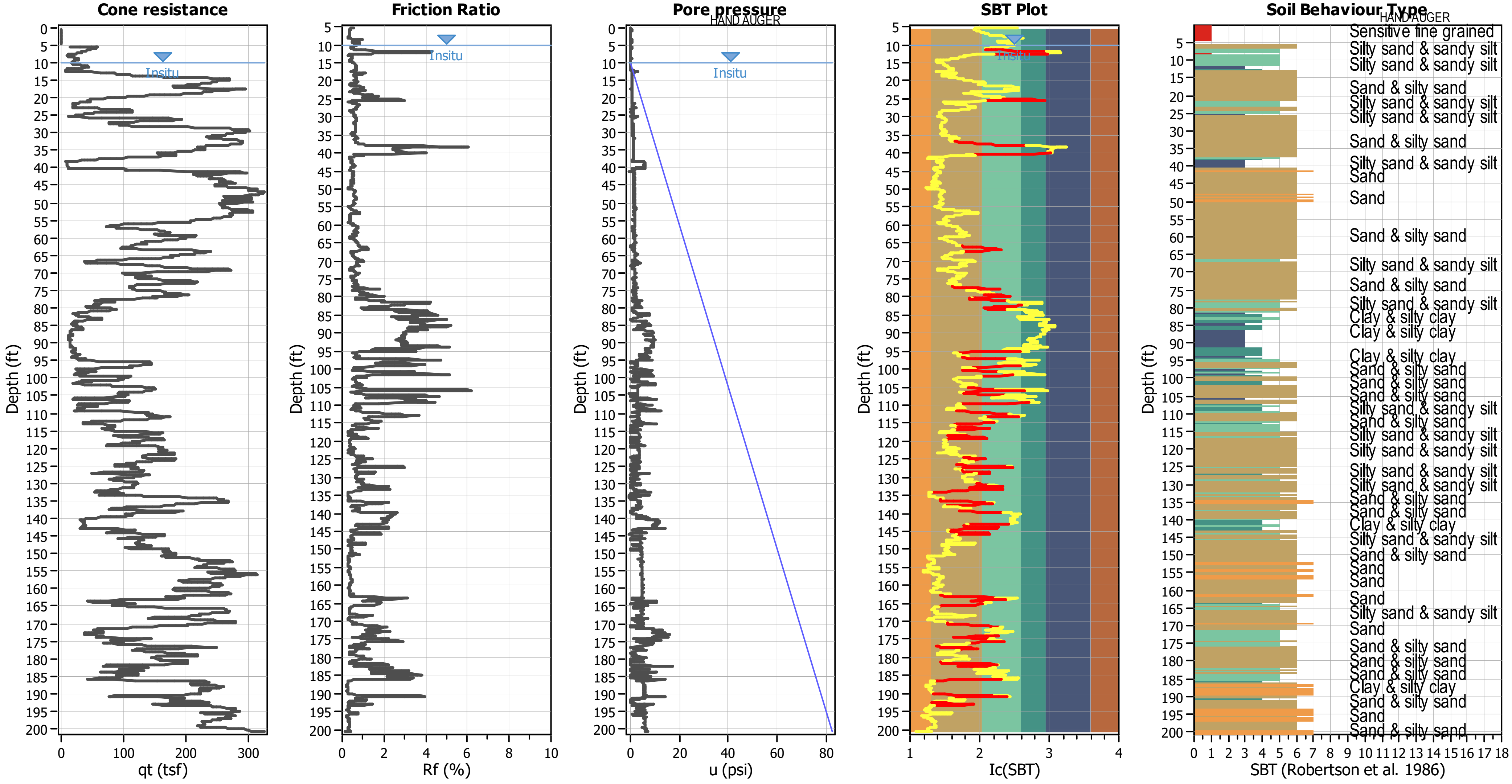
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

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

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

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

CPT basic interpretation plots



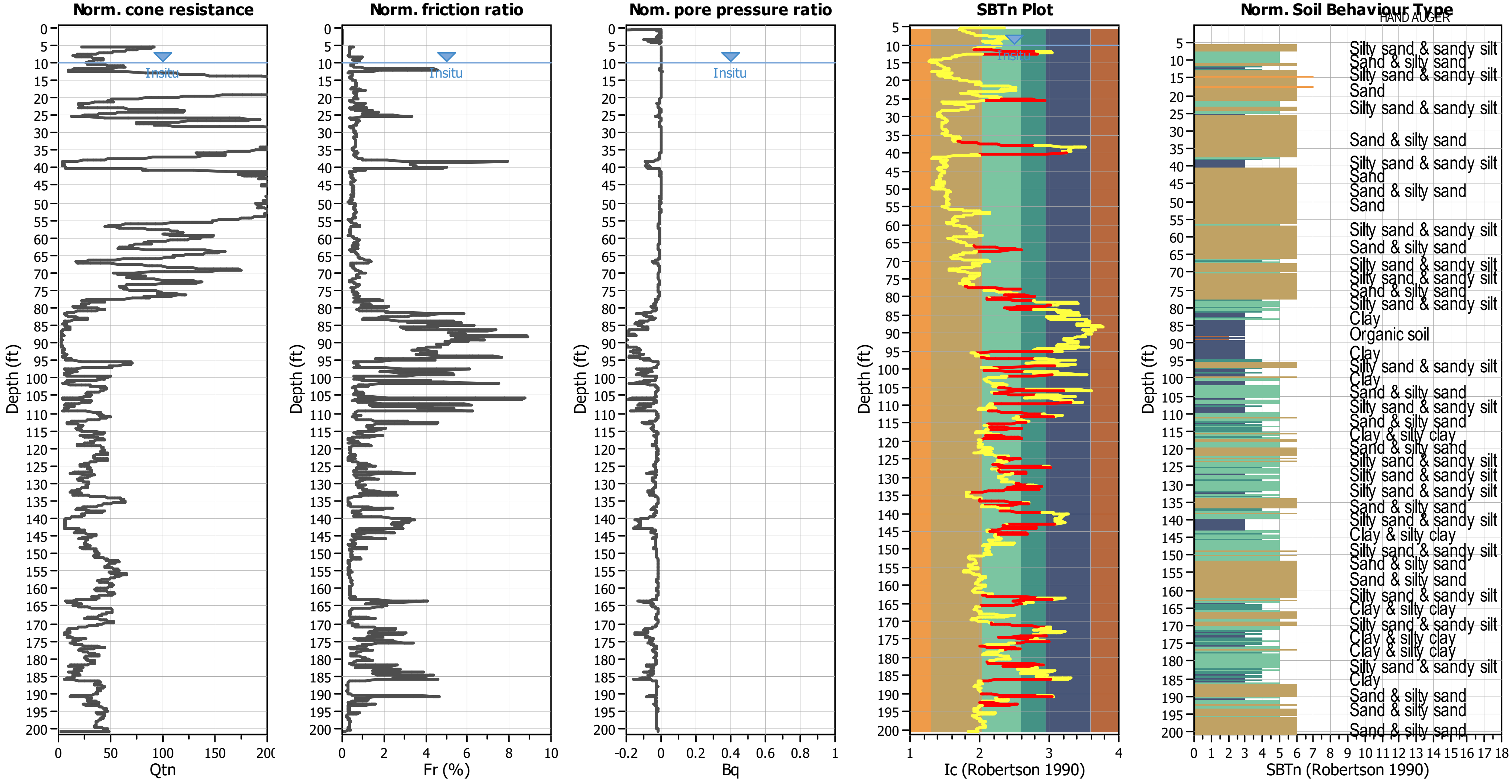
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



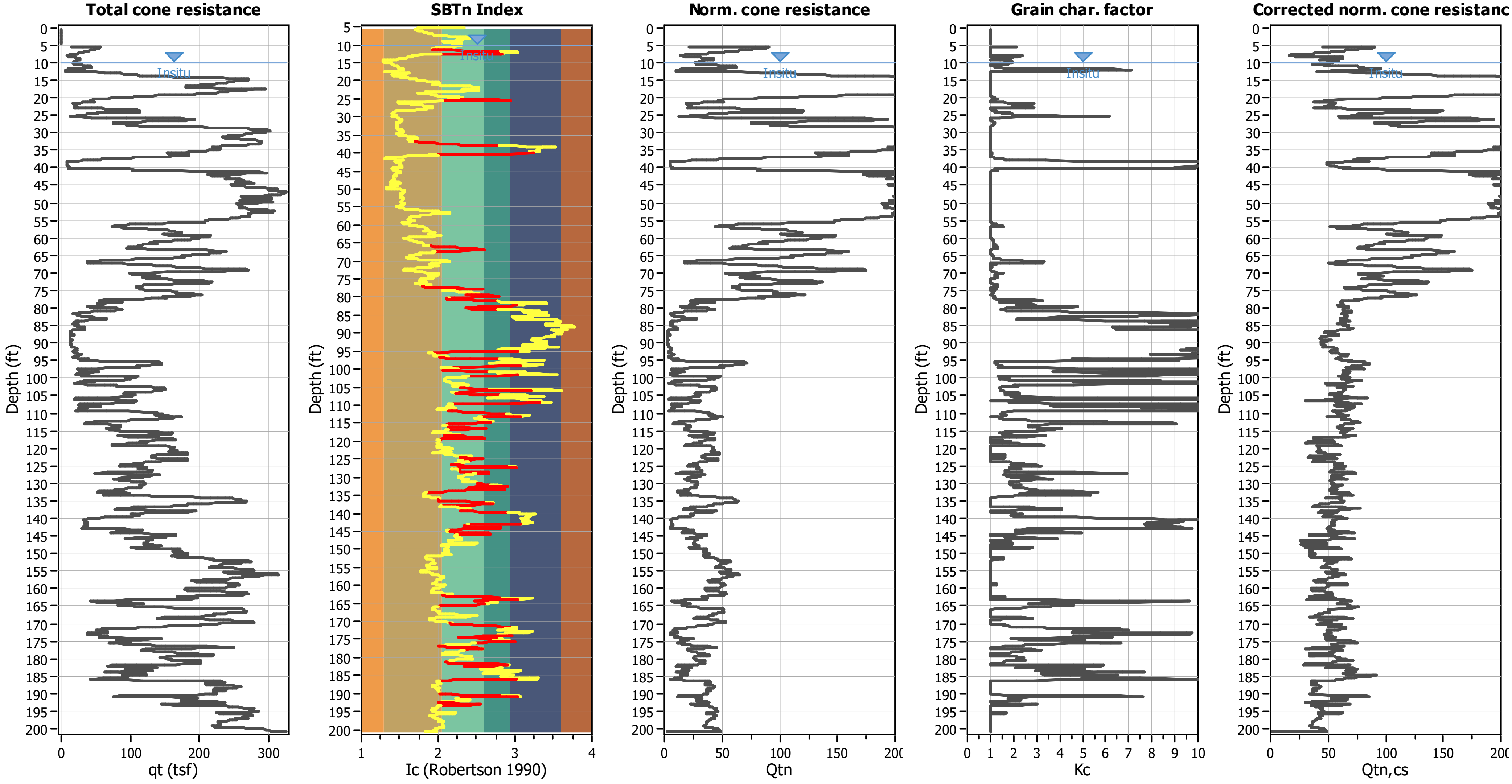
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

SBTn legend

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

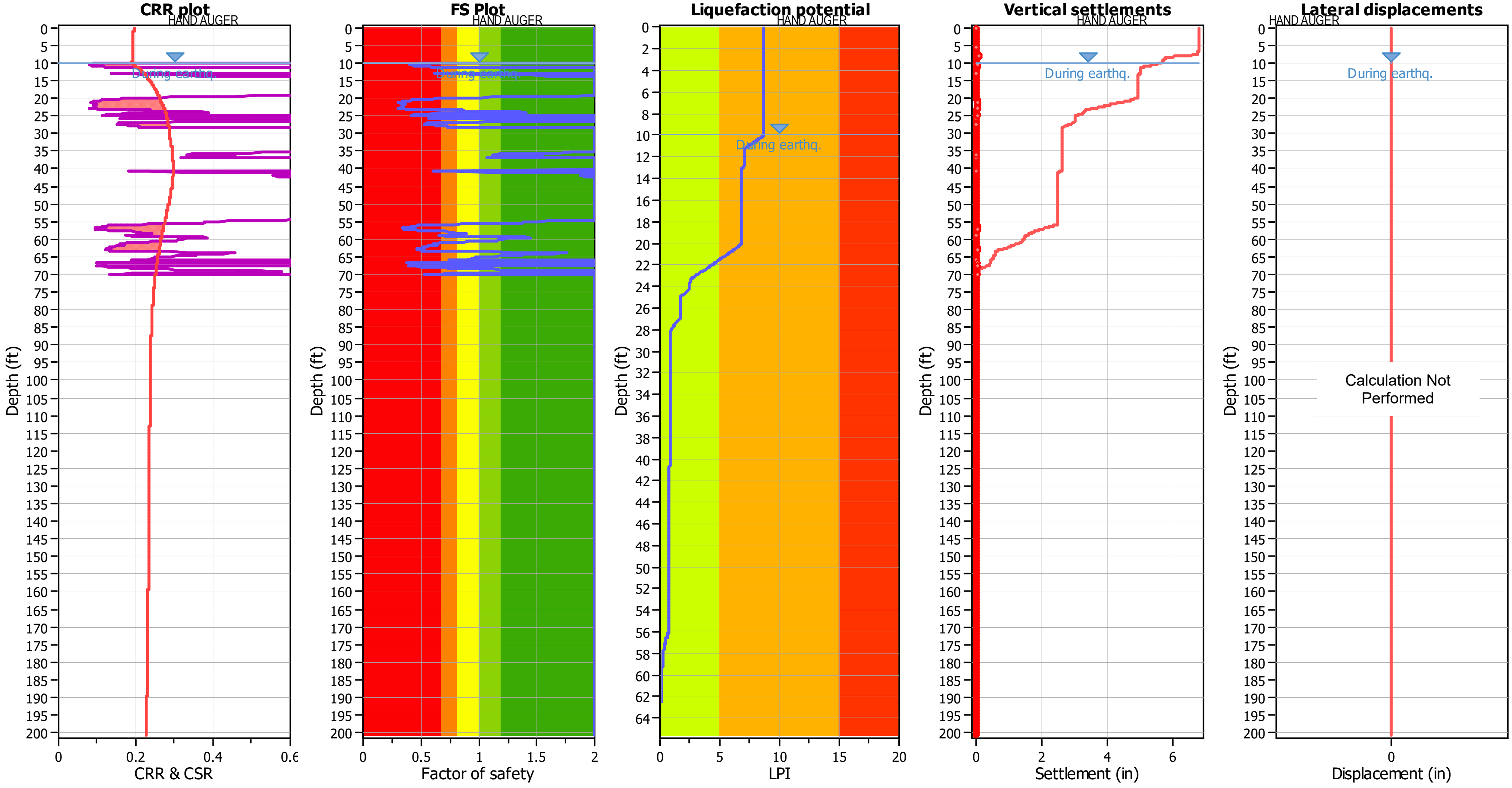
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

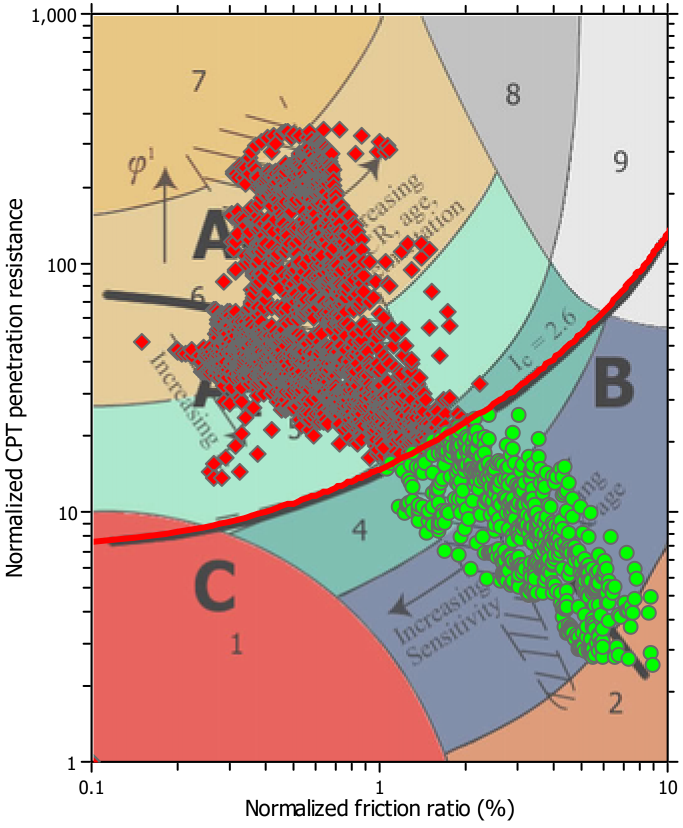
F.S. color scheme

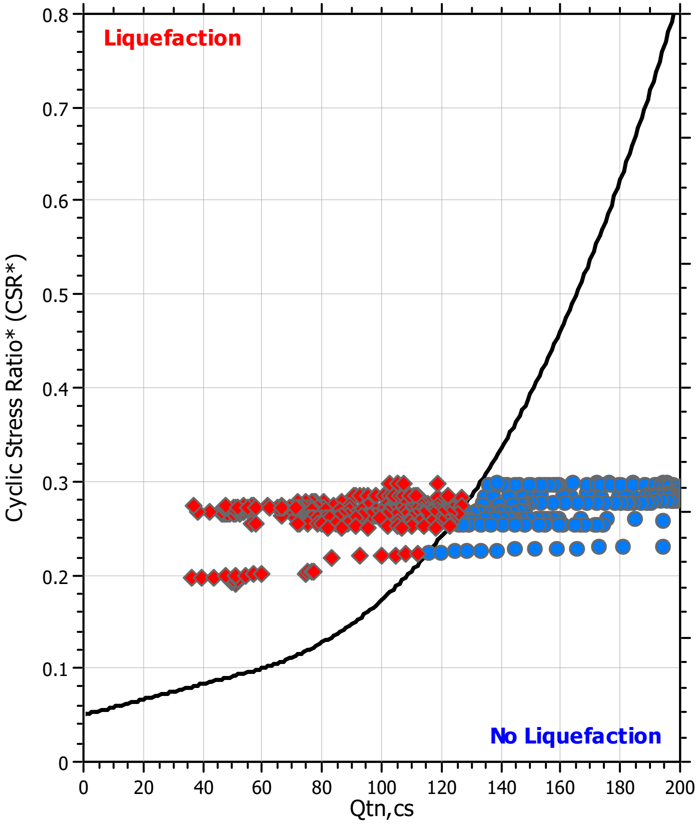
🟥 Almost certain it will liquefy
🟧 Very likely to liquefy
🟨 Liquefaction and no liq. are equally likely
🟩 Unlike to liquefy
🟩 Almost certain it will not liquefy

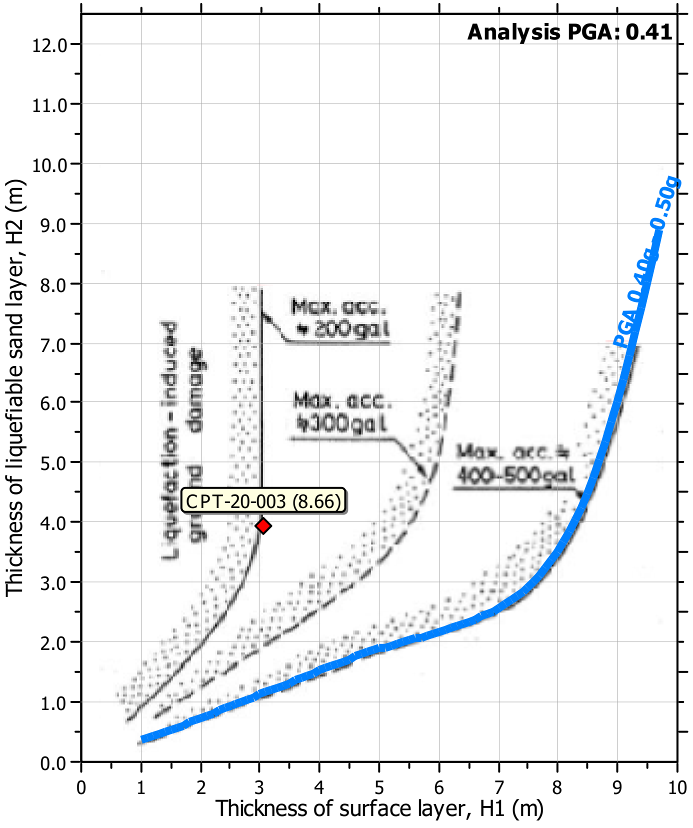
LPI color scheme

🟥 Very high risk
🟧 High risk
🟨 Low risk

Liquefaction analysis summary plots



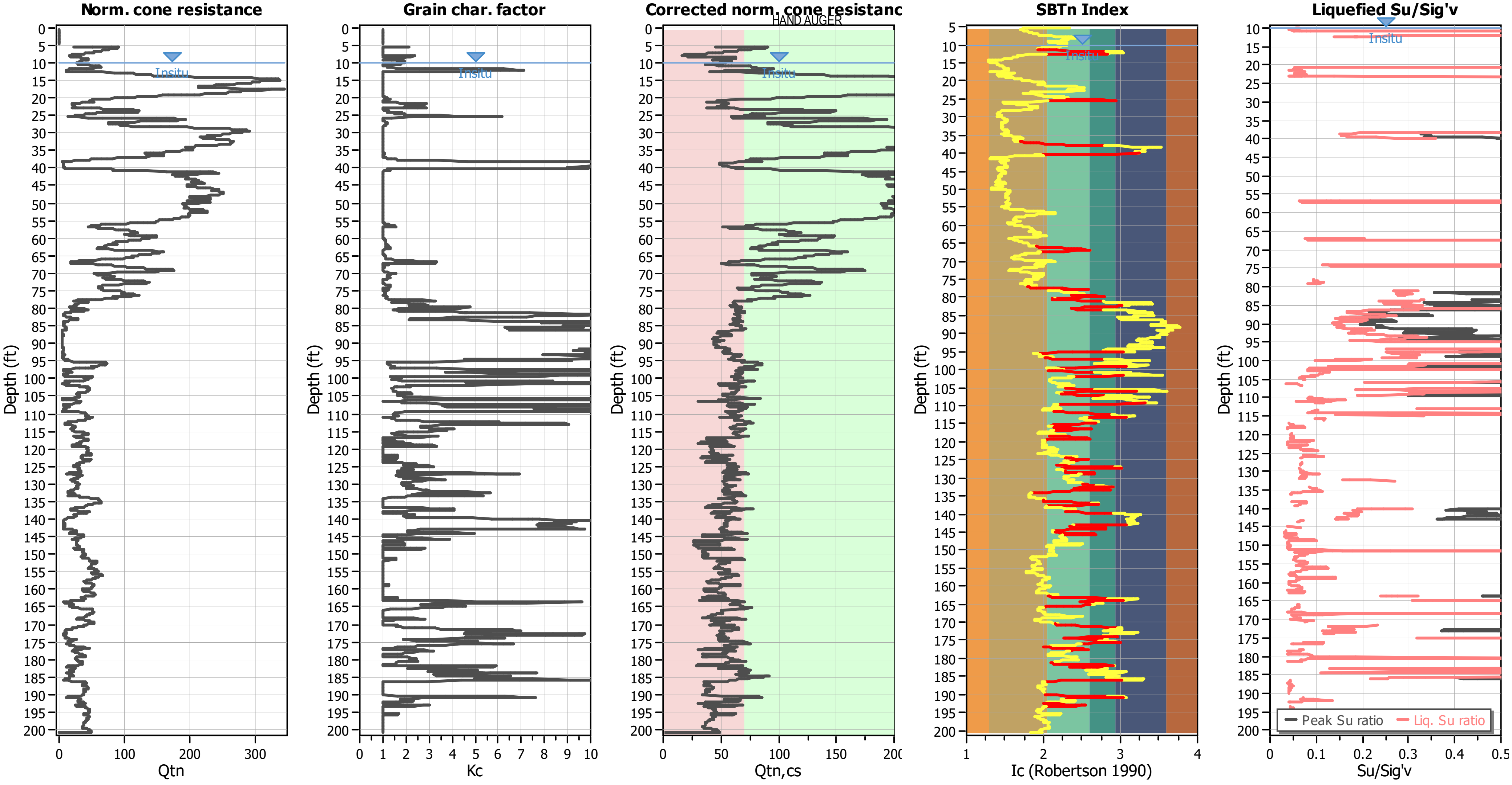




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.41	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	70.00 ft

TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

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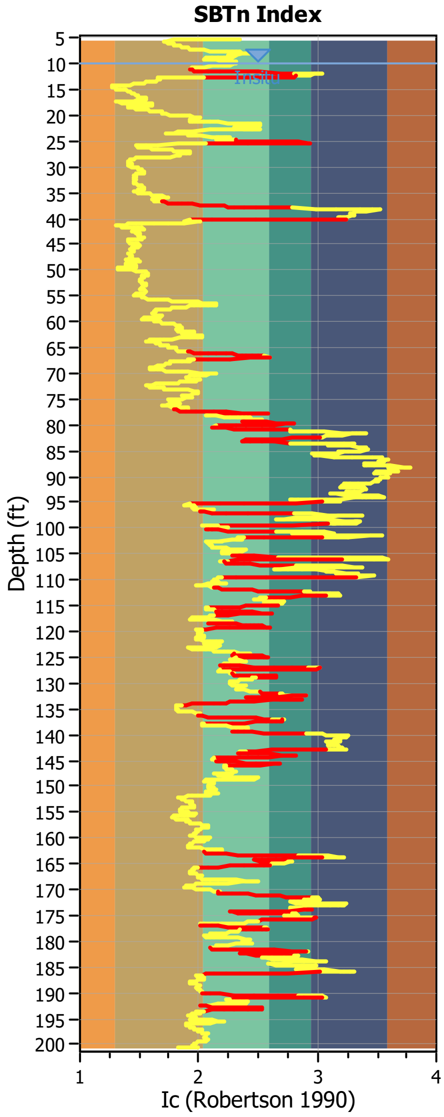
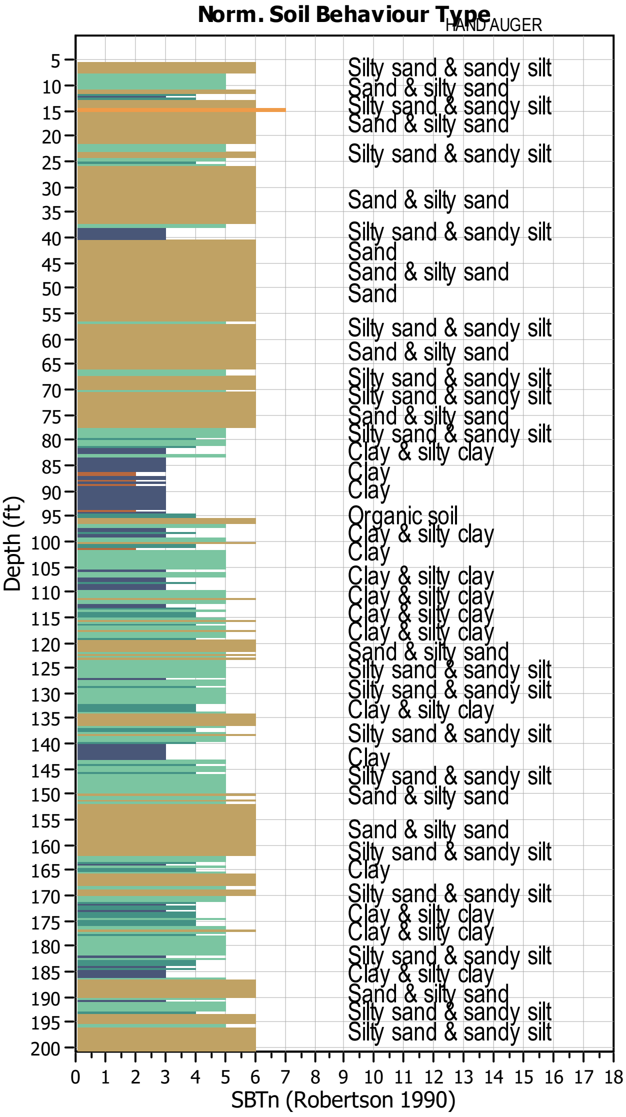
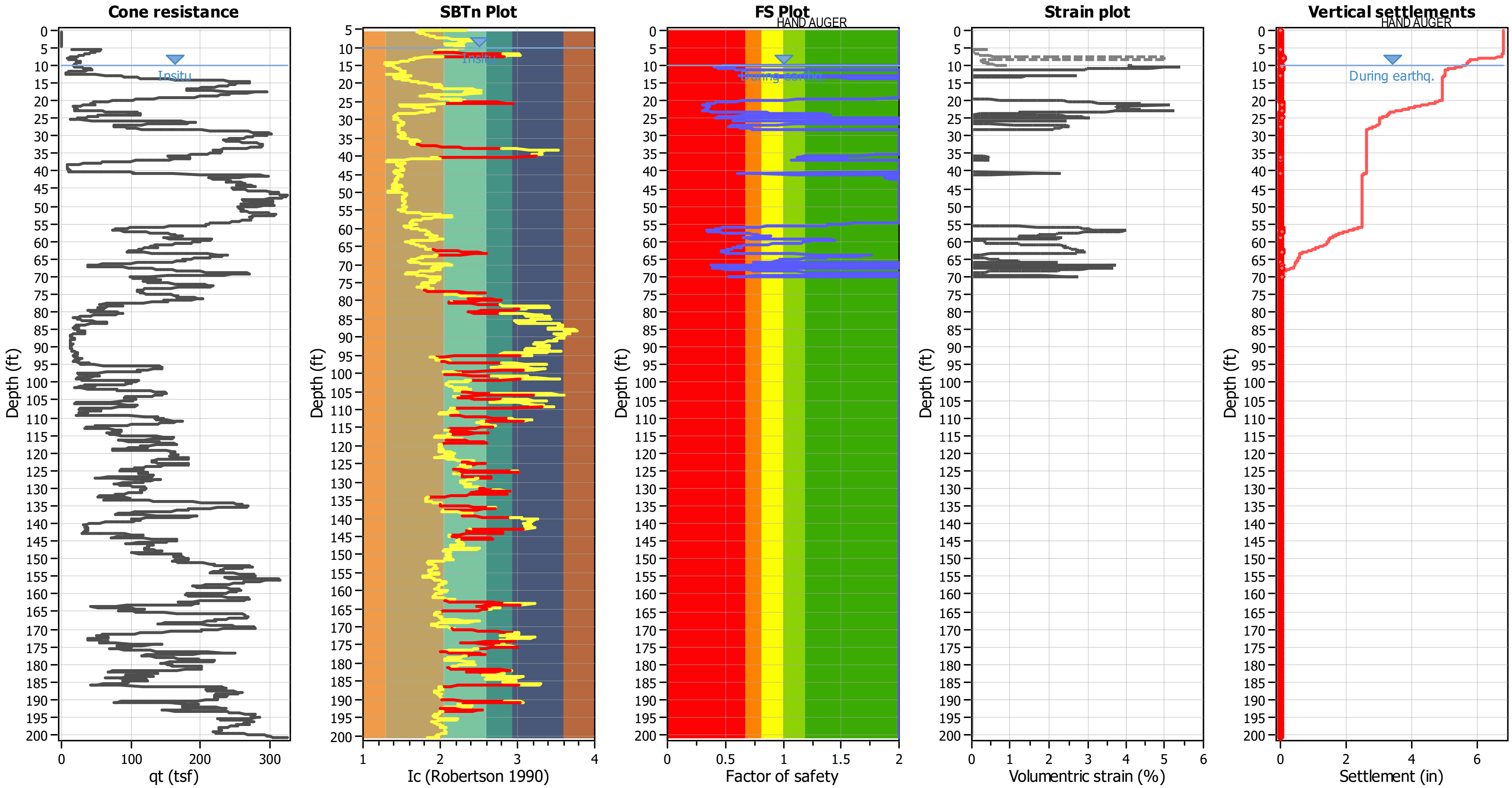
The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





Transition layer algorithm properties		**General statistics**	
I_c minimum check value:	1.70	Total points in CPT file:	3061
I_c maximum check value:	3.00	Total points excluded:	554
I_c change ratio value:	0.0250	Exclusion percentage:	18.10%
Minimum number of points in layer:	4	Number of layers detected:	67

Estimation of post-earthquake settlements

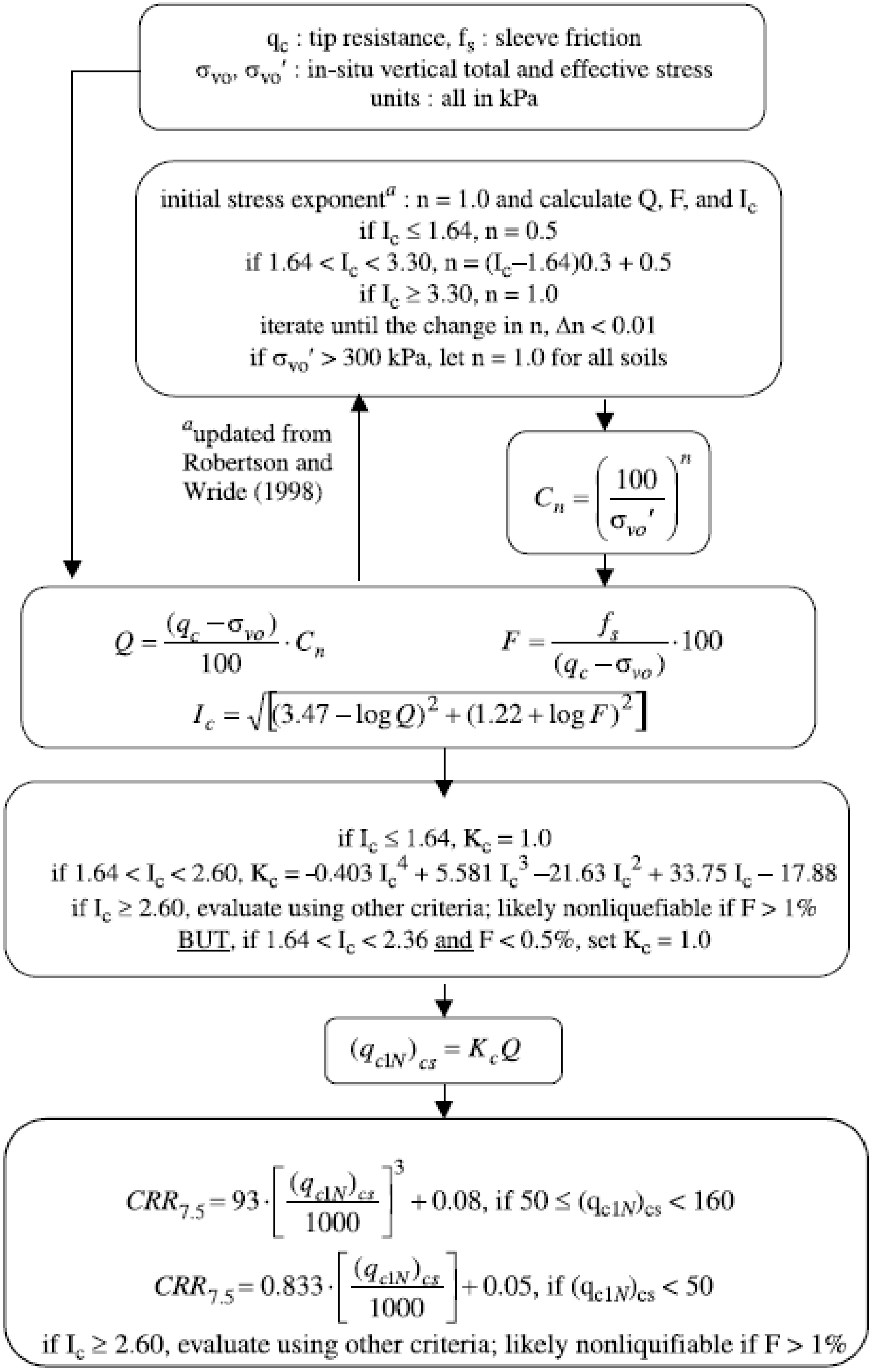


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



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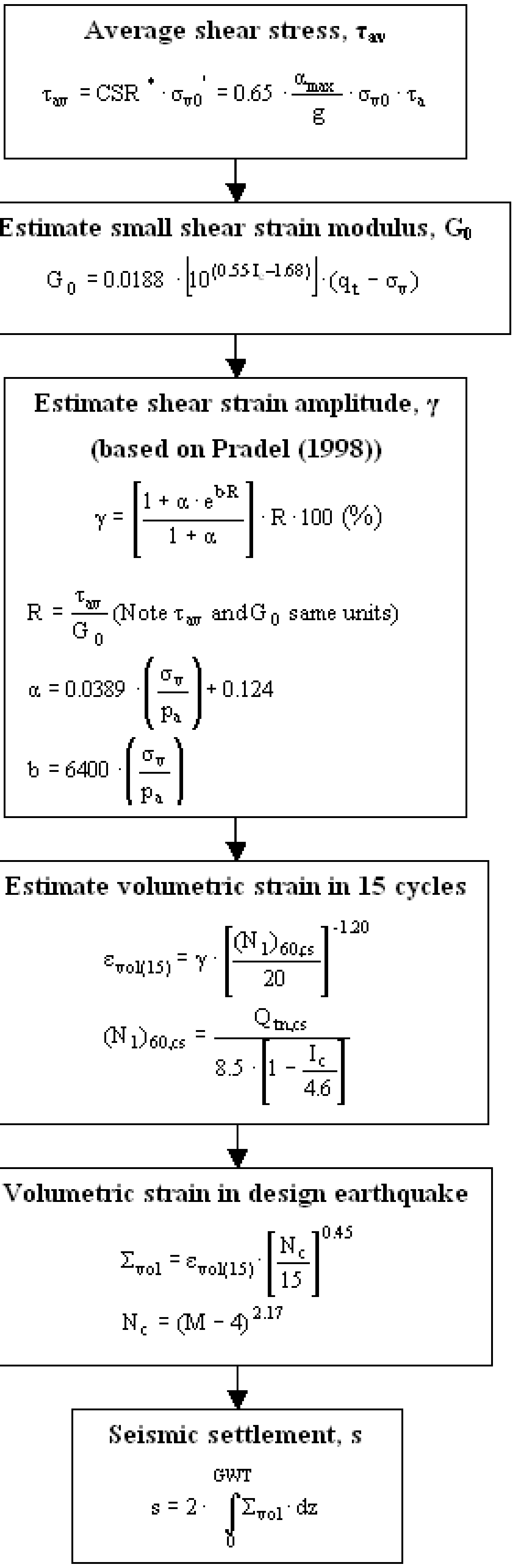
[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands



Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{\alpha_{max}}{g} \cdot \sigma_{v0} \cdot \tau_{a}$$

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

Estimate shear strain amplitude, γ (based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, “Estimation of seismic compression in dry soils using the CPT” FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

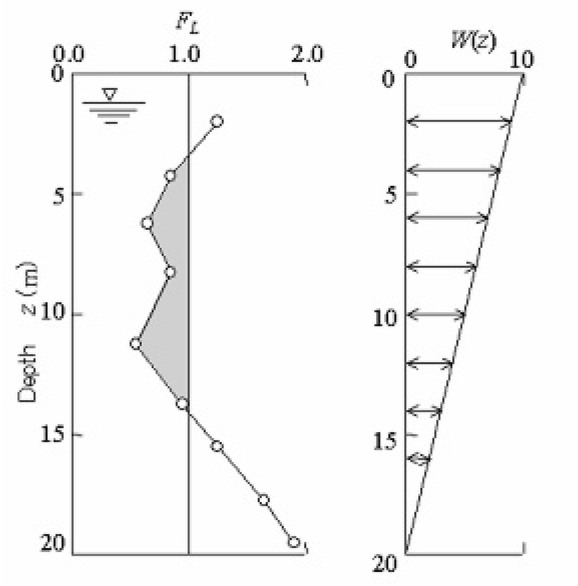
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_{0}^{20} (10 - 0,5_z) \times F_I \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

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- R. E. S. Moss, R. B. Seed, R. E. Kayen, J. P. Stewart, A. Der Kiureghian, K. O. Cetin, CPT-Based Probabilistic and Deterministic Assessment of In Situ Seismic Soil Liquefaction Potential, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 8, August 1, 2006

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G.3 LIQUEFACTION-INDUCED LATERAL SPREADING ANALYSES

PROJECT: Camino Del Mar Bridge Replacement
PROJECT NO.: 20180876.001A
SUBJECT: Liquefaction-Induced lateral spreading
Equations used in Calculations

DATE: 4/20/2020
PERFORMED BY: JB
REVIEWED BY: ZZ

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EQUATIONS USED IN CALCULATIONS

D_H repesents median values. The expected range lies within 1/2 to 2 times the median values.

Youd et al. (2002) Sloping Ground:

$$\log D_H = -16.213 + 1.532M - 1.406 \log R^* - 0.012R$$
$$+ 0.338 \log S + 0.540 \log T_{15} + 3.413 \log(100 - F_{15})$$
$$- 0.795 \log(D50_{15} + 0.1 \ \ \text{mm}) \qquad (6b)$$

Youd et al. (2002) Free Face:

$$\log D_H = -16.713 + 1.532M - 1.406 \log R^* - 0.012R$$
$$+ 0.592 \log W + 0.540 \log T_{15} + 3.413 \log(100 - F_{15})$$
$$- 0.795 \log(D50_{15} + 0.1 \ \ \text{mm}) \qquad (6a)$$

Bardet et al. (1999) Six Parameter Model:

$$Log(D_H + 0.01) = b_0 + b_{off} + b_1 M + b_2 Log(R) + b_3 R$$
$$+ b_4 Log(W_{ff}) + b_5 Log(S_{gs}) + b_6 Log(T_{15}) + b_7 Log(100 - F_{15}) + b_8 D50_{15}$$

Where: W_{ff} = W (percent); S_{gs} = S (in percent)

Model coefficients	
b_0	-14.551
b_{off}	-0.483
b_1	1.096
b_2	-0.873
b_3	-0.014
b_4	0.634
b_5	0.275
b_6	0.494
b_7	4.053
b_8	-0.814

Zhang et al. (2004):

$$D_r = 16 \cdot \sqrt{(N_1)_{78}} = 14 \cdot \sqrt{(N_1)_{60}} \quad [(N_1)_{60} \leq 42]$$

$$D_r = -85 + 76 \log(q_{c1N}) \quad (q_{c1N} \leq 200)$$

$$\text{LDI} = \int_0^{Z_{max}} \gamma_{max} dz$$

(see chart to right for γ_{max}=f(Dr,FS

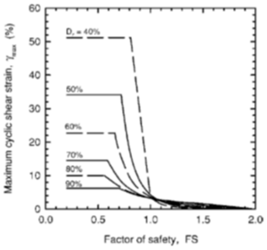
For sloping ground:

$$\frac{\text{LD}}{\text{LDI}} = S + 0.2 \quad (\text{for } \ 0.2\% < S < 3.5\%)$$

For free face:

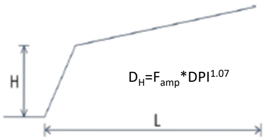
$$\frac{\text{LD}}{\text{LDI}} = 6 \cdot \left(\frac{L}{H}\right)^{-0.8} \quad (\text{for } \ 4 < L/H < 40)$$

D_H = LD = LDI*(LD/LDI)



Faris (2004) and Caltrans (2012):

For Free Face:



D_H=F_{amp}*$DPI^{1.07}$

$$F_{amp} = \begin{cases} 2 & \text{for} \quad L/H < 4 \\ 6\ (L/H)^{-0.8} & \text{for} \quad 4 < L/H < 9 \\ 1 & \text{for} \quad 9 < L/H \end{cases}$$

$$\gamma_{max} = \begin{cases} 0 & CSR \leq 0.04 \\ \left(\frac{CSR - 0.04}{l_1 - 0.04}\right) 0.01 & N_{1,60,cs} \leq 15 \ AND \ 0.04 \leq CSR \leq l_1 \\ 0.01 + \left(\frac{CSR - l_1}{l_2 - l_1}\right) 0.02 & N_{1,60,cs} \leq 17.5 \ AND \ l_1 \leq CSR \leq l_2 \\ 1.859 \left(1.1 - \sqrt{\frac{N_{1,60,cs} + \Delta n}{46}}\right)^3 \geq 0 & N_{1,60,cs} > 17.5 \ OR \ CSR > l_2 \end{cases}$$

For sloping Ground:

$$D_H \ = S \cdot DPI^{1.07} \quad \textit{for } 0.25 \leq S < 1 \textit{ and S is the slope (\%)}$$

$$l_1 = 0.04 + .00207 \, N_{1,60,cs}$$

$$l_2 = 0.04 + .0047 \, N_{1,60,cs}$$

$$\Delta n = N_{1,60,cs} \left(\frac{1}{\left(\frac{CSR - 0.04}{0.56}\right)^{1/3}} - 1 \right)$$

$$DPI = \int_{\substack{liq. \\ layer}} \gamma_{max} \, dz$$

Idriss & Boulanger (2008):

$$\text{LDI} = \int_0^{z_{max}} \gamma_{max} \cdot dz$$

$$\gamma_{lim} = 1.859 \left(1.1 - \sqrt{\frac{(N_1)_{60cs}}{46}}\right)^3 \geq 0$$

(and $\gamma_{lim} \leq 50$)

$$\gamma_{max} = 0 \quad \text{if} \quad FS_{liq} \geq 2$$

$$\gamma_{max} = \min\left(\gamma_{lim}, 0.035\,(2 - FS_{liq}) \left(\frac{1 - F_\alpha}{FS_{liq} - F_\alpha}\right)\right)$$

$$\text{if} \quad 2 > FS_{liq} > F_\alpha$$

$$\gamma_{max} = \gamma_{lim} \quad \text{if} \quad FS_{liq} \leq F_\alpha$$

$$F_\alpha = 0.032 + 0.69\sqrt{(N_1)_{60cs}} - 0.13\,(N_1)_{60cs}$$

with $(N_1)_{60cs}$ limited to values ≥ 7

Note: The Idriss & Boulanger (2008) method does not provide a way to convert from the Lateral Displacement Index (LDI) to an actual estimate for ground deformation that accounts for important factors such as earthquake magnitude, ground slope gradient, free face height and distance, etc. Idriss & Boulanger did not validate their method against an emprical dataset. For these reasons, the I&B method truly only produces an *index* of the expected lateral spreading ground deformtion. Despite these limitgations, the I&B calculation has been included for completeness an in cases where clients or projects require the I&B method.

REFERENCES:

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Caltrans (2012). *Guidelines on Foundation Loading and Defomration Due to Liqeufaction Induced Lateral Spreading*, January, 2012.

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Idriss, I.M. and Boulanger, R.W. (2008). *Soil Liquefaction During Earthquakes,* EERI Monograph MNO-12.

Youd, L.T., Hansen, C.M., Bartlett, S.F., 2002, "Revised Multilinear Regression Equations for Predicting Lateral Spread Displacement", *Journal of Geotechnical and Geoenvironmental Engineering*, Volume 128, No. 12, ASCE, December 2002.

Youd, L.T., Hansen, C.M., Bartlett, S.F., 1999, "Revised MLR Equations for Predicting Lateral Spread Displacement", *Proceedings of the Seventh U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction*, MCEER-99-0019, pp 99-117

Zhang, G., Robertson, P.K. and Brachman, R.W.I. (2004). "Estimating Liquefaction-Induced Lateral Displacements Using the Standard Penetration Test or Cone Penetration Test," JGGE, 130(8), 861-871.

PROJECT: Camino Del Mar Bridge Replacement
PROJECT NO.: 20180876.001A
SUBJECT: Liquefaction-Induced lateral spreading estimate
slope ground condition at piers
Based on boring CPT-20-001

DATE: 4/20/2020
PERFORMED BY: JB
REVIEWED BY: ZZ

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OBJECTIVE: Estimate liquefaction-induced lateral spreading displacements using the five methods indicated below based on subsurface information from CPT-20-001 (near North Abutment).

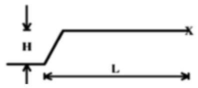
GIVENS:
See Geometry in Geologic Cross-Section A-A'
See Stratigraphy in Cross Section A-A' and Boring and CPT Logs
Factors of safety and CPT or SPT data are calculated in a separate liquefaction triggering analysis. See the calculation package in Appendix G.

ASSUMPTIONS: Several simplyfying assumptions are necessary to perform this type of calculation: Simple geometry, average soil properties, site conditions fall within the range of characteristics of the model databases, etc.

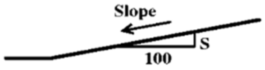
METHODOLGY: Five methods are used in the calculation:
1. Youd et al. (1999, 2002) and Bartlet & Youd (1995).
2. Bardet et al. (1999) 6-parameter model.
3. Zhang et al. (2004).
4. Faris (2004) as implemented by Caltrans (2012)
5. Idriss & Boulanger (2008)

Earthquake and Geometry Input

Variable Name	Variable Description	Input Values (enter SI or English)		Model Range				
				Youd et al (2002)	Bardet et al (1999)	Zhang et al (2004)	Faris (2004) Caltrans (2012)	Idriss & Boulanger (2008)
Mw	Moment magnitude of design EQ	6.63		6<Mw<8	6.4<Mw<9.2	6.4<Mw<9.2	not provdied	not provdied
R (km)	Equivalent distance to seismic source	3.5 km	2.2 mi	0.5km<R<100km	0.2km<R<100km	na	na	na
H (m)	Height of free face	4.0 m	13.0 ft	na	na	na	na	na
L (m)	Distance from toe of free face to site	9.1 m	30.0 ft	na	na	na	na	na
T_{15} (m)	Thickness of liq. layer w/ $(N1)_{60}$<=15	2.3 m	7.5 ft	$1m<T_{15}<15m$	$0.2m<T_{15}<20m$	na	na	na
F_{15} (%)	Average fines content (-#200) in T_{15} layer	5.0 %		$0\%<F_{15}<50\%$	$0\%<F_{15}<70\%$	na	na	na
$(D_{50})_{15}$ (mm)	Mean grain size in T_{15} layer	0.50 mm		$0.07<(D_{50})_{15}$ <3mm	$0.04<(D_{50})_{15}<1.5mm$	na	na	na
S (%)	Ground slope	4.3 %		0.1%<S<6%	0.05%<S<6%	0%<S<6.4%	not provdied	not provdied
W (%)	Free face ratio, W = H/L(100%)	43.3 %		1%<W<20%	1.6%<W<56%	2.5%<W<25%	not provdied	not provdied
Z_{T15} (m)	Depth to top of liq. layer	5.6 m	18.5 ft	1m<W<10m	na	na	not provdied	not provdied
Geometry	Free face of sloping ground condition	Sloping Ground		"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"



"Free Face" Case



"Sloping Ground" Case

Youd (2002) Intermediate Calculations

R_0	$=10^{(0.89M-5.64)}$	1.8 km	1.1 mi
R*	$= Req + R_0$	5.4 km	3.3 mi

Use Faris/Caltrans? Yes

Soil Profile Input use in Zhang et al. (2004) and Faris/Caltrans Only

Zhang et al., Faris/Caltrans and Idriss & Boulanger									Faris/Caltrans			Zhang et al.		I&B	
Layer No.	Soil Description	Depth to Top (feet)	Depth to Bot. (feet)	Layer Thickns (feet)	Use SPT or CPT $(N_1)_{60}$	q_{c1N}	FS_{liq}	FC (%)	CSR $_{M=7.5,\sigma_{vo'}=1}$	γ_{max} (%)	DPI (feet)	γ_{max} (%)	LDI (feet)	γ_{max} (%)	LDI (feet)
1	Poorly-Graded SAND with Silt (SP-SM)	0.0	10.0	10.0	41.2	200	NL	5	0.20	0.0	0.44	0.0	1.07	0.0	0.81
2	Poorly-Graded SAND with Silt (SP-SM)	10.0	13.0	3.0	31.1	140	NL	6	0.20	0.0	0.44	0.0	1.07	0.0	0.81
3	Poorly-Graded SAND with Silt (SP-SM)	13.0	18.5	5.5	39.7	190	NL	5	0.23	0.0	0.44	0.0	1.07	0.0	0.81
4	Poorly-Graded SAND with Silt (SP-SM)	18.5	22.0	3.5	21.3	93	0.74	5	0.26	5.3	0.44	10.8	1.07	8.3	0.81
5	Poorly-Graded SAND with Silt (SP-SM)	22.0	26.0	4.0	20.6	90	0.67	5	0.27	6.3	0.25	17.2	0.69	13.0	0.52

RESULTS

Lateral Spreading Displacement Estimate Results (D_H = Median)

	meters			feet		
	Low	Median	High	Low	Median	High
Youd et al. (2002):	**0.81**	**1.62**	**3.25**	**2.7**	**5.3**	**10.7**
Bardet et al. (1999):	**0.70**	**1.39**	**2.79**	**2.3**	**4.6**	**9.1**
Zhang et al. (2004):	**0.73**	**1.46**	**2.93**	**2.4**	**4.8**	**9.6**
Farris (2004) & Caltrans (2012):	**0.06**	**0.12**	**0.23**	**0.2**	**0.4**	**0.8**
Idriss & Boulanger (2008):	*See note below*			*See note below*		

CONCLUSIONS/INTERPRETATIONS:
Liquefiable layers accounted for in the upper 2H of the soil profile for lateral spreading calculations assuming mean grain size of 0.5mm.

Median "free-field" lateral spreading displacement for sloping condition at river channel range from approx. 0.5 to 5.5 feet. However, the piles would resist lateral flow resulting in lower lateral displacements.

PROJECT: Camino Del Mar Bridge Replacement
PROJECT NO.: 20180876.001A
SUBJECT: Liquefaction-Induced lateral spreading estimate
Based on boring CPT-20-002A

DATE: 4/21/2020
PERFORMED BY: JB
REVIEWED BY: ZZ

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OBJECTIVE: Estimate liquefaction-induced lateral spreading displacements using the five methods indicated below based on subsurface information from CPT-20-002A (near central portion of bridge).

GIVENS: See Geometry in Geologic Cross-Section A-A'
See Stratigraphy in Cross Section A-A' and Boring and CPT Logs
Factors of safety and CPT or SPT data are calculated in a separate liquefaction triggering analysis. See the calculation package in Appendix G.

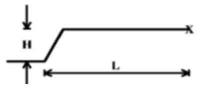
ASSUMPTIONS: Several simplyfying assumptions are necessary to perform this type of calculation: Simple geometry, average soil properties, site conditions fall within the range of characteristics of the model databases, etc.

METHODOLGY: Five methods are used in the calculation:

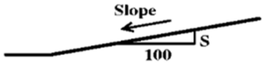
1. Youd et al. (1999, 2002) and Bartlet & Youd (1995).
2. Bardet et al. (1999) 6-parameter model.
3. Zhang et al. (2004).
4. Faris (2004) as implemented by Caltrans (2012)
5. Idriss & Boulanger (2008)

Earthquake and Geometry Input

Variable Name	Variable Description	Input Values (enter SI or English)		Model Range				
				Youd et al (2002)	Bardet et al (1999)	Zhang et al (2004)	Faris (2004) Caltrans (2012)	Idriss & Boulanger (2008)
Mw	Moment magnitude of design EQ	6.63		6<Mw<8	6.4<Mw<9.2	6.4<Mw<9.2	not provdied	not provdied
R (km)	Equivalent distance to seismic source	3.5 km	2.2 mi	0.5km<R<100km	0.2km<R<100km	na	na	na
H (m)	Height of free face	3.0 m	10.0 ft	na	na	na	na	na
L (m)	Distance from toe of free face to site	9.1 m	30.0 ft	na	na	na	na	na
T_{15} (m)	Thickness of liq. layer w/ $(N1)_{60}$<=15	2.8 m	9.3 ft	1m<T_{15}<15m	0.2m<T_{15}<20m	na	na	na
F_{15} (%)	Average fines content (-#200) in T_{15} layer	5.0 %		0%<F_{15}<50%	0%<F_{15}<70%	na	na	na
$(D_{50})_{15}$ (mm)	Mean grain size in T_{15} layer	0.50 mm		0.07<$(D_{50})_{15}$ <3mm	0.04<$(D_{50})_{15}$<1.5mm	na	na	na
S (%)	Ground slope	4.3 %		0.1%<S<6%	0.05%<S<6%	0%<S<6.4%	not provdied	not provdied
W (%)	Free face ratio, W = H/L(100%)	33.3 %		1%<W<20%	1.6%<W<56%	2.5%<W<25%	not provdied	not provdied
Z_{T15} (m)	Depth to top of liq. layer	0.0 m	0.0 ft	1m<W<10m	na	na	not provdied	not provdied
Geometry	Free face of sloping ground condition	Sloping Ground		"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"



"Free Face" Case



"Sloping Ground" Case

Youd (2002) Intermediate Calculations

R_0	$=10^{(0.89M-5.64)}$	1.8 km	1.1 mi
R*	= Req + R_0	5.4 km	3.3 mi

Use Faris/Caltrans? Yes

Soil Profile Input use in Zhang et al. (2004) and Faris/Caltrans Only

Zhang et al., Faris/Caltrans and Idriss & Boulanger									Faris/Caltrans			Zhang et al.		I&B	
Layer No.	Soil Description	Depth to Top (feet)	Depth to Bot. (feet)	Layer Thickns (feet)	Use SPT or CPT $(N_1)_{60}$	q_{c1N}	FS_{liq}	FC (%)	CSR $_{M=7.5,\sigma_{vo'}=1}$	γ_{max} (%)	DPI (feet)	γ_{max} (%)	LDI (feet)	γ_{max} (%)	LDI (feet)
1	Silty SAND to Sandy SILT (SM to ML)	0.0	1.8	1.8	2.0	24	0.35	30	0.19	73.9	5.10	51.2	4.76	50.0	4.45
2	Sensitive Fine-Grained (OL to CH)	1.8	13.0	11.2	3.4	6	NL	80	0.20	0.0	3.77	0.0	3.84	0.0	3.55
3	Silty SAND (SM)	13.0	14.7	1.7	7.8	43	0.36	15	0.22	39.0	3.77	51.2	3.84	42.0	3.55
4	Sandy SILT (ML)	14.7	20.5	5.8	4.1	31	0.27	50	0.24	53.6	3.11	51.2	2.97	48.8	2.83

RESULTS

Lateral Spreading Displacement Estimate Results (D_H = Median)

	meters			feet		
	Low	Median	High	Low	Median	High
Youd et al. (2002):	**0.91**	**1.82**	**3.65**	**3.0**	**6.0**	**12.0**
Bardet et al. (1999):	**0.77**	**1.55**	**3.10**	**2.5**	**5.1**	**10.2**
Zhang et al. (2004):	**3.27**	**6.53**	**13.07**	**10.7**	**21.4**	**42.9**
Farris (2004) & Caltrans (2012):	**0.80**	**1.61**	**3.21**	**2.6**	**5.2**	**10.5**
Idriss & Boulanger (2008):		*See note below*			*See note below*	

CONCLUSIONS/INTERPRETATIONS:
Liquefiable layers accounted for in the upper approximate 2H of the soil profile for lateral spreading calculations assuming mean grain size of 0.5mm.

Median "free-field" lateral spreading displacement for sloping condition at river channel range from approx. 5 to 6 feet (excluding Zhang due to unreasonbly high value). However, the piles would resist lateral flow resulting in lower lateral displacements.

PROJECT: Camino Del Mar Bridge Replacement
PROJECT NO.: 20180876.001A
SUBJECT: Liquefaction-Induced lateral spreading estimate
Sloping ground condition at piers
Based on boring CPT-20-003

DATE: 4/20/2020
PERFORMED BY: JB
REVIEWED BY: ZZ

OBJECTIVE: Estimate liquefaction-induced lateral spreading displacements using the five methods indicated below based on subsurface information from CPT-20-003 (near Sorth Abutment)

GIVENS:
See Geometry in geologic Cross-Section A-A'
See Stratigraphy in Cross Section A-A' and Boring and CPT Logs
Factors of safety and CPT or SPT data are calculated in a separate liquefaction triggering analysis. See the calculation package in Appendix G.

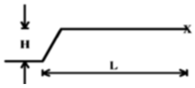
ASSUMPTIONS: Several simplyfying assumptions are necessary to perform this type of calculation: Simple geometry, average soil properties, site conditions fall within the range of characteristics of the model databases, etc.

METHODOLGY: Five methods are used in the calculation:

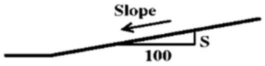
1. Youd et al. (1999, 2002) and Bartlet & Youd (1995).
2. Bardet et al. (1999) 6-parameter model.
3. Zhang et al. (2004).
4. Faris (2004) as implemented by Caltrans (2012)
5. Idriss & Boulanger (2008)

Earthquake and Geometry Input

Variable Name	Variable Description	Input Values (enter SI or English)		Model Range				
				Youd et al (2002)	Bardet et al (1999)	Zhang et al (2004)	Faris (2004) Caltrans (2012)	Idriss & Boulanger (2008)
Mw	Moment magnitude of design EQ	6.63		6<Mw<8	6.4<Mw<9.2	6.4<Mw<9.2	not provdied	not provdied
R (km)	Equivalent distance to seismic source	3.5 km	2.2 mi	0.5km<R<100km	0.2km<R<100km	na	na	na
H (m)	Height of free face	3.0 m	10.0 ft	na	na	na	na	na
L (m)	Distance from toe of free face to site	9.1 m	30.0 ft	na	na	na	na	na
T_{15} (m)	Thickness of liq. layer w/ $(N1)_{60}$<=15	1.7 m	5.5 ft	1m<T_{15}<15m	0.2m<T_{15}<20m	na	na	na
F_{15} (%)	Average fines content (-#200) in T_{15} layer	5.0 %		0%<F_{15}<50%	0%<F_{15}<70%	na	na	na
$(D_{50})_{15}$ (mm)	Mean grain size in T_{15} layer	0.50 mm		0.07<$(D_{50})_{15}$ <3mm	0.04<$(D_{50})_{15}$<1.5mm	na	na	na
S (%)	Ground slope	4.3 %		0.1%<S<6%	0.05%<S<6%	0%<S<6.4%	not provdied	not provdied
W (%)	Free face ratio, W = H/L(100%)	33.3 %		1%<W<20%	1.6%<W<56%	2.5%<W<25%	not provdied	not provdied
Z_{T15} (m)	Depth to top of liq. layer	3.0 m	10.0 ft	1m<W<10m	na	na	not provdied	not provdied
Geometry	Free face of sloping ground condition	Sloping Ground		"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"



"Free Face" Case



"Sloping Ground" Case

Youd (2002) Intermediate Calculations

R_0	$=10^{(0.89M-5.64)}$	1.8 km	1.1 mi
R*	= Req + R_0	5.4 km	3.3 mi

Use Faris/Caltrans? Yes

Soil Profile Input use in Zhang et al. (2004) and Faris/Caltrans Only

Zhang et al., Faris/Caltrans and Idriss & Boulanger									Faris/Caltrans			Zhang et al.		I&B	
Layer No.	Soil Description	Depth to Top (feet)	Depth to Bot. (feet)	Layer Thickns (feet)	Use SPT or CPT $(N_1)_{60}$	q_{c1N}	FS_{liq}	FC (%)	CSR $_{M=7.5,\sigma_{vo'}=1}$	γ_{max} (%)	DPI (feet)	γ_{max} (%)	LDI (feet)	γ_{max} (%)	LDI (feet)
1	Poorly-Graded SAND with Silt (SP-SM)	0.0	10.0	10.0	8.7	46	NL	5	0.19	0.0	1.82	0.0	2.31	0.0	1.94
2	Poorly-Graded SAND with Silt (SP-SM)	10.0	11.2	1.2	8.1	44	0.44	5	0.21	37.7	1.82	51.2	2.31	50.0	1.94
3	Poorly-Graded SAND with Silt (SP-SM)	11.2	12.7	1.5	3.2	28	NL	5	0.23	0.0	1.37	0.0	1.69	0.0	1.34
4	Poorly-Graded SAND with Silt (SP-SM)	12.7	13.7	1.0	25.7	113	0.91	5	0.24	1.8	1.37	4.2	1.69	4.3	1.34
5	Poorly-Graded SAND (SP)	13.7	19.7	6.0	48.6	253	NL	3	0.27	0.0	1.35	0.0	1.65	0.0	1.30
6	Silty SAND (SM)	19.7	23.0	3.3	8.4	45	0.38	15	0.29	41.1	1.35	50.1	1.65	39.3	1.30

RESULTS

Lateral Spreading Displacement Estimate Results (D_H = Median)

	meters			feet		
	Low	Median	High	Low	Median	High
Youd et al. (2002):	**0.69**	**1.37**	**2.75**	**2.3**	**4.5**	**9.0**
Bardet et al. (1999):	**0.60**	**1.19**	**2.39**	**2.0**	**3.9**	**7.8**
Zhang et al. (2004):	**1.58**	**3.17**	**6.34**	**5.2**	**10.4**	**20.8**
Farris (2004) & Caltrans (2012):	**0.27**	**0.53**	**1.07**	**0.9**	**1.7**	**3.5**
Idriss & Boulanger (2008):		*See note below*			*See note below*	

CONCLUSIONS/INTERPRETATIONS:
Liquefiable layers accounted for in the upper approximate 2H of the soil profile for lateral spreading calculations assuming mean grain size of 0.5mm.

Median "free-field" lateral spreading displacement for sloping condition at river channel range from approx. 2 to 4.5 feet (excluding Zhang due to unreasonbly high value). However, the piles would resist lateral flow resulting in lower lateral displacements.

PROJECT: Camino Del Mar Bridge Replacement
PROJECT NO.: 20180876.001A
SUBJECT: Liquefaction-Induced lateral spreading estimate
Sloping ground condition at piers
Based on boring R-20-001

DATE: 4/20/2020
PERFORMED BY: JB
REVIEWED BY: ZZ

KLEINFELDER
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OBJECTIVE: Estimate liquefaction-induced lateral spreading displacements using the five methods indicated below based on subsurface information from boring R-20-001 (near North Abutment)

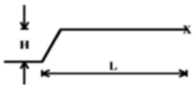
GIVENS:
See Geometry in geologic Cross-Section A-A'
See Stratigraphy in Cross Section A-A' and Boring and CPT Logs
Factors of safety and CPT or SPT data are calculated in a separate liquefaction triggering analysis. See the calculation package in Appendix G.

ASSUMPTIONS: Several simplyfying assumptions are necessary to perform this type of calculation: Simple geometry, average soil properties, site conditions fall within the range of characteristics of the model databases, etc.

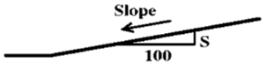
METHODOLGY: Five methods are used in the calculation:
1. Youd et al. (1999, 2002) and Bartlet & Youd (1995).
2. Bardet et al. (1999) 6-parameter model.
3. Zhang et al. (2004).
4. Faris (2004) as implemented by Caltrans (2012)
5. Idriss & Boulanger (2008)

Earthquake and Geometry Input

Variable Name	Variable Description	Input Values (enter SI or English)		Model Range				
				Youd et al (2002)	Bardet et al (1999)	Zhang et al (2004)	Faris (2004) Caltrans (2012)	Idriss & Boulanger (2008)
Mw	Moment magnitude of design EQ	6.63		6<Mw<8	6.4<Mw<9.2	6.4<Mw<9.2	not provdied	not provdied
R (km)	Equivalent distance to seismic source	3.5 km	2.2 mi	0.5km<R<100km	0.2km<R<100km	na	na	na
H (m)	Height of free face	4.0 m	13.0 ft	na	na	na	na	na
L (m)	Distance from toe of free face to site	9.1 m	30.0 ft	na	na	na	na	na
T_{15} (m)	Thickness of liq. layer w/ $(N1)_{60}$<=15	1.7 m	5.5 ft	1m<T_{15}<15m	0.2m<T_{15}<20m	na	na	na
F_{15} (%)	Average fines content (-#200) in T_{15} layer	5.0 %		0%<F_{15}<50%	0%<F_{15}<70%	na	na	na
$(D_{50})_{15}$ (mm)	Mean grain size in T_{15} layer	0.50 mm		0.07<$(D_{50})_{15}$ <3mm	0.04<$(D_{50})_{15}$<1.5mm	na	na	na
S (%)	Ground slope	4.3 %		0.1%<S<6%	0.05%<S<6%	0%<S<6.4%	not provdied	not provdied
W (%)	Free face ratio, W = H/L(100%)	43.3 %		🟥 1%<W<20%	1.6%<W<56%	🟥 2.5%<W<25%	not provdied	not provdied
Z_{T15} (m)	Depth to top of liq. layer	5.9 m	19.5 ft	1m<W<10m	na	na	not provdied	not provdied
Geometry	Free face of sloping ground condition	Sloping Ground		"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"



"Free Face" Case



"Sloping Ground" Case

Youd (2002) Intermediate Calculations

R_0	$=10^{(0.89M-5.64)}$	1.8 km	1.1 mi
R*	= Req + R_0	5.4 km	3.3 mi

Use Faris/Caltrans? Yes

Soil Profile Input use in Zhang et al. (2004) and Faris/Caltrans Only

Zhang et al., Faris/Caltrans and Idriss & Boulanger									Faris/Caltrans			Zhang et al.		I&B	
Layer No.	Soil Description	Depth to Top (feet)	Depth to Bot. (feet)	Layer Thickns (feet)	Use SPT or CPT $(N_1)_{60}$	q_{c1N}	FS_{liq}	FC (%)	CSR $_{M=7.5,\sigma_{vo'}=1}$	γ_{max} (%)	DPI (feet)	γ_{max} (%)	LDI (feet)	γ_{max} (%)	LDI (feet)
1	Poorly-Graded SAND with Silt (SP-SM)	0.0	7.5	7.5	56.0	314	NL	5.1	0.19	0.0	1.56	0.0	2.19	0.0	2.29
2	Poorly-Graded SAND with Silt (SP-SM)	7.5	10.0	2.5	46.0	233	NL	11	0.19	0.0	1.56	0.0	2.19	0.0	2.29
3	Poorly-Graded SAND with Silt (SP-SM)	10.0	11.5	1.5	42.0	205	NL	6	0.20	0.0	1.56	0.0	2.19	0.0	2.29
4	Poorly-Graded SAND with Silt (SP-SM)	11.5	13.5	2.0	34.0	156	NL	6	0.21	0.0	1.56	0.0	2.19	0.0	2.29
5	Poorly-Graded SAND with Silt (SP-SM)	13.5	15.5	2.0	57.0	323	NL	6.2	0.23	0.0	1.56	0.0	2.19	0.0	2.29
6	Poorly-Graded SAND with Silt (SP-SM)	15.5	17.5	2.0	39.0	186	NL	5	0.24	0.0	1.56	0.0	2.19	0.0	2.29
7	Poorly-Graded SAND with Silt (SP-SM)	17.5	19.5	2.0	26.0	114	1.21	3.9	0.26	2.0	1.56	1.9	2.19	2.2	2.29
8	Poorly-Graded SAND with Silt (SP-SM)	19.5	21.5	2.0	12.0	57	0.49	5.5	0.26	23.7	1.52	36.7	2.16	38.0	2.24
9	Poorly-Graded SAND with Silt (SP-SM)	21.5	25.0	3.5	11.0	54	0.46	5.5	0.27	28.1	1.04	40.2	1.42	42.4	1.48
10	Poorly-Graded SAND with Silt (SP-SM)	25.0	28.0	3.0	27.0	119	2	5.5	0.28	1.9	0.06	0.5	0.02	0.0	0.00

RESULTS

Lateral Spreading Displacement Estimate Results (D_H = Median)

	meters			feet		
	Low	Median	High	Low	Median	High
Youd et al. (2002):	**0.69**	**1.37**	**2.75**	**2.3**	**4.5**	**9.0**
Bardet et al. (1999):	**0.60**	**1.19**	**2.39**	**2.0**	**3.9**	**7.8**
Zhang et al. (2004):	**1.50**	**3.01**	**6.02**	**4.9**	**9.9**	**19.7**
Farris (2004) & Caltrans (2012):	**0.23**	**0.45**	**0.90**	**0.7**	**1.5**	**3.0**
Idriss & Boulanger (2008):		*See note below*			*See note below*	

CONCLUSIONS/INTERPRETATIONS:
Liquefiable layers accounted for in the upper 2H of the soil profile for lateral spreading calculations assuming mean grain size of 0.5mm.

Median "free-field" lateral spreading displacement for sloping condition at river channel range from approx. 1.5 to 4.5 feet (excluding Zhang due to unreasonably high value). However, the piles would resist lateral flow resulting in lower lateral displacements.

PROJECT: Camino Del Mar Bridge Replacement
PROJECT NO.: 20180876.001A
SUBJECT: Liquefaction-Induced lateral spreading estimate
Sloping ground condition at piers
Based on boring R-20-002

DATE: 4/20/2020
PERFORMED BY: JB
REVIEWED BY: ZZ

OBJECTIVE: Estimate liquefaction-induced lateral spreading displacements using the five methods indicated below based on subsurface information from boring R-20-002 (near South Abutment)

GIVENS: See Geometry in geologic Cross-Section A-A'
See Stratigraphy in Cross Section A-A' and Boring and CPT Logs
Factors of safety and CPT or SPT data are calculated in a separate liquefaction triggering analysis. See the calculation package in Appendix G.

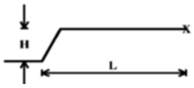
ASSUMPTIONS: Several simplyfying assumptions are necessary to perform this type of calculation: Simple geometry, average soil properties, site conditions fall within the range of characteristics of the model databases, etc.

METHODOLGY: Five methods are used in the calculation:

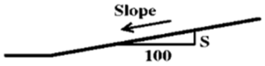
1. Youd et al. (1999, 2002) and Bartlet & Youd (1995).
2. Bardet et al. (1999) 6-parameter model.
3. Zhang et al. (2004).
4. Faris (2004) as implemented by Caltrans (2012)
5. Idriss & Boulanger (2008)

Earthquake and Geometry Input

Variable Name	Variable Description	Input Values (enter SI or English)		Model Range				
				Youd et al (2002)	Bardet et al (1999)	Zhang et al (2004)	Faris (2004) Caltrans (2012)	Idriss & Boulanger (2008)
Mw	Moment magnitude of design EQ	6.63		6<Mw<8	6.4<Mw<9.2	6.4<Mw<9.2	not provdied	not provdied
R (km)	Equivalent distance to seismic source	3.5 km	2.2 mi	0.5km<R<100km	0.2km<R<100km	na	na	na
H (m)	Height of free face	3.0 m	10.0 ft	na	na	na	na	na
L (m)	Distance from toe of free face to site	9.1 m	30.0 ft	na	na	na	na	na
T_{15} (m)	Thickness of liq. layer w/ $(N1)_{60}$<=15	2.6 m	8.5 ft	$1m<T_{15}<15m$	$0.2m<T_{15}<20m$	na	na	na
F_{15} (%)	Average fines content (-#200) in T_{15} layer	5.0 %		$0\%<F_{15}<50\%$	$0\%<F_{15}<70\%$	na	na	na
$(D_{50})_{15}$ (mm)	Mean grain size in T_{15} layer	0.50 mm		$0.07<(D_{50})_{15}$ <3mm	$0.04<(D_{50})_{15}<1.5mm$	na	na	na
S (%)	Ground slope	4.3 %		0.1%<S<6%	0.05%<S<6%	0%<S<6.4%	not provdied	not provdied
W (%)	Free face ratio, W = H/L(100%)	33.3 %		1%<W<20%	1.6%<W<56%	2.5%<W<25%	not provdied	not provdied
Z_{T15} (m)	Depth to top of liq. layer	3.0 m	10.0 ft	1m<W<10m	na	na	not provdied	not provdied
Geometry	Free face of sloping ground condition	Sloping Ground		"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"	"Free Face" or "Sloping Ground"



"Free Face" Case



"Sloping Ground" Case

Youd (2002) Intermediate Calculations

R_0	$=10^{(0.89M-5.64)}$	1.8 km	1.1 mi
R*	= Req + R_0	5.4 km	3.3 mi

Use Faris/Caltrans? Yes

Soil Profile Input use in Zhang et al. (2004) and Faris/Caltrans Only

Zhang et al., Faris/Caltrans and Idriss & Boulanger									Faris/Caltrans			Zhang et al.		I&B	
Layer No.	Soil Description	Depth to Top (feet)	Depth to Bot. (feet)	Layer Thickns (feet)	Use SPT or CPT $(N_1)_{60}$	q_{c1N}	FS_{liq}	FC (%)	CSR $_{M=7.5,\sigma_{vo'}=1}$	γ_{max} (%)	DPI (feet)	γ_{max} (%)	LDI (feet)	γ_{max} (%)	LDI (feet)
1	Poorly-Graded SAND (SP)	0.0	7.0	7.0	26.0	114	NL	3.3	0.19	0.0	3.94	0.0	4.05	0.0	3.83
2	Poorly-Graded SAND with Silt (SP-SM)	7.0	8.5	1.5	24.0	105	NL	5.6	0.19	0.0	3.94	0.0	4.05	0.0	3.83
3	Poorly-Graded SAND with Silt (SP-SM)	8.5	10.0	1.5	9.0	47	NL	5	0.20	0.0	3.94	0.0	4.05	0.0	3.83
4	Poorly-Graded SAND with Silt (SP-SM)	10.0	12.5	2.5	4.0	31	0.37	5.2	0.22	76.5	3.94	51.2	4.05	50.0	3.83
5	Poorly-Graded SAND with Silt (SP-SM)	12.5	14.0	1.5	8.0	44	0.46	5	0.23	40.3	2.03	51.2	2.77	50.0	2.58
6	Poorly-Graded SAND (SP)	14.0	16.5	2.5	11.0	54	0.5	3.2	0.24	26.1	1.43	40.2	2.00	42.4	1.83
7	Poorly-Graded SAND (SP)	16.5	18.5	2.0	41.0	199	NL	3	0.25	0.0	0.78	0.0	1.00	0.0	0.77
8	Poorly-Graded SAND(SP)	18.5	20.5	2.0	25.0	110	1.14	3	0.26	2.5	0.78	2.2	1.00	2.5	0.77
9	Silty SAND (SM)	20.5	22.5	2.0	9.0	47	0.52	16	0.27	36.3	0.73	47.8	0.96	35.7	0.71

RESULTS

Lateral Spreading Displacement Estimate Results (D_H = Median)

	meters			feet		
	Low	Median	High	Low	Median	High
Youd et al. (2002):	**0.87**	**1.74**	**3.47**	**2.8**	**5.7**	**11.4**
Bardet et al. (1999):	**0.74**	**1.48**	**2.96**	**2.4**	**4.9**	**9.7**
Zhang et al. (2004):	**2.78**	**5.56**	**11.12**	**9.1**	**18.2**	**36.5**
Farris (2004) & Caltrans (2012):	**0.61**	**1.22**	**2.44**	**2.0**	**4.0**	**8.0**
Idriss & Boulanger (2008):	*See note below*			*See note below*		

CONCLUSIONS/INTERPRETATIONS:
Liquefiable layers accounted for in the upper approximate 2H of the soil profile for lateral spreading calculations assuming mean grain size of 0.5mm.

Median "free-field" lateral spreading displacement for sloping condition within river channel range from approx.4 to 6 feet (excluding Zhang due to unreasonably high value). However, the piles would resist lateral flow resulting in lower lateral displacements.

https://kleinfelder1-my.sharepoint.com/personal/jbonfiglio_kleinfelder_com/Documents/Desktop/[LatSprd Printing.xlsx]Sheet1

G.4 SLOPE STABILITY AND SEISMIC SLOPE DISPLACEMENT ANALYSES

Soil Description	γ_{total} (pcf)	Static Slope Stability Strength Parameters					Seismic Slope Stability Strength Parameters					
		Model	Static		Rapid Drawdown		Model	Yield Acceleration		Post-Liquefaction		Residual Strength
			c' / Su (psf)	Φ'	c' (psf)	Φ'		c' / Su (psf)	Φ'	c' / Su (psf)	Φ'	Sr (psf)
Artificial Fill (af) (above water)	120	Mohr-Coulomb	50	33	50	33	Mohr-Coulomb	50	34	50	33	-
Recent Alluvial Deposits (Qa) (Sand)	120	Mohr-Coulomb	50	28	50	28	Undrained (Phi = 0)	-	-	-	-	450
Recent Alluvial Deposits (Qa) (Clay)	110	Undrained (Phi=0)	400	0	0	18	Undrained (Phi=0)	400	0	400	0	-
Young Alluvial Deposits (Qya) (Sand)	125	Mohr-Coulomb	50	32	50	32	Undrained (Phi = 0)	-	-	-	-	700
Young Estuarine Deposits (Qyes) (Clay)	115	Undrained (Phi=0)	750	0	0	22	Undrained (Phi=0)	750	0	750	0	-
Old Alluvial Deposits (Qoa) (Sand)	125	Mohr-Coulomb	50	34	50	34	Mohr-Coulomb	50	35	50	34	-
Old Alluvial Deposits (Qoa)	130	Mohr-Coulomb	50	36	50	36	Mohr-Coulomb	50	37	50	36	-
Old Alluvial Deposits (Qoa) (Gravelly Sand)	130	Mohr-Coulomb	0	36	0	36	Mohr-Coulomb	0	37	0	36	-
Del Mar Formation	135	Mohr-Coulomb	4000	0	0	36	Mohr-Coulomb	4000	0	4000	0	-

Minimum Factor of Safety Requirements:

Model Case	Minimum Factor of Safety
Static Short-Term	1.3
Static Long-Term	1.5
Post-Liquefaction	1.1
Rapid Drawdown	1.1

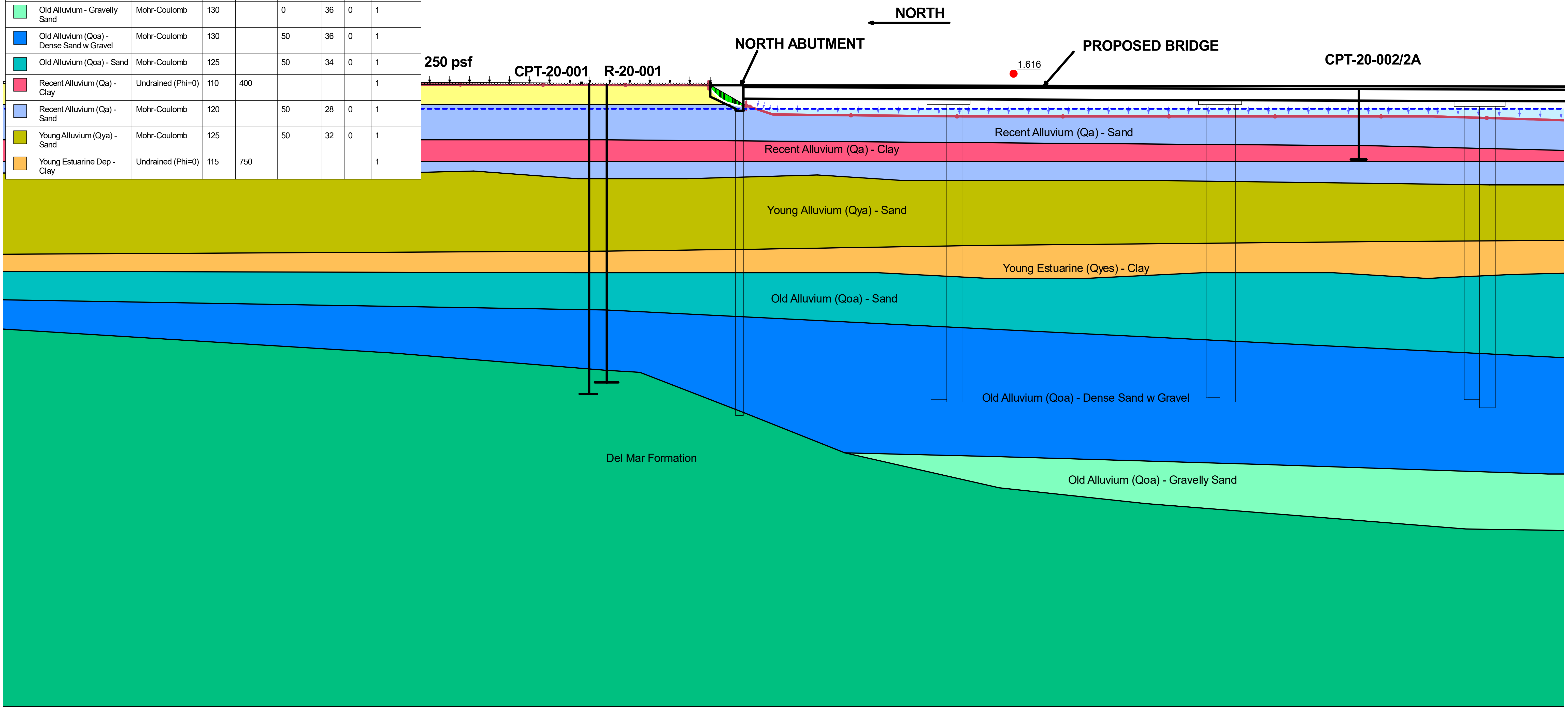
Static Slope Stability Results:

Case		Critical Factor of Safety
Static Short-Term	North Abutment	1.616
	South Abutment	1.515
Static Long-Term	North Abutment	1.861
	South Abutment	3.014
Post-Liquefaction	North Abutment	1.202
	South Abutment	1.363
Rapid Drawdown	North Abutment	1.534
	South Abutment	2.308

Seismic Slope Stability Results:

Case		Yield Coefficient	Median Seismic Slope Displacement
Yield Acceleration	North Abutment	0.049g	3.2 ft
	South Abutment	0.072g	2 ft

Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120		50	33	0	1
🟩	Del Mar Formation	Mohr-Coulomb	135		4,000	0	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130		0	36	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130		50	36	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125		50	34	0	1
🟥	Recent Alluvium (Qa) - Clay	Undrained (Phi=0)	110	400				1
🟦	Recent Alluvium (Qa) - Sand	Mohr-Coulomb	120		50	28	0	1
🟨	Young Alluvium (Qya) - Sand	Mohr-Coulomb	125		50	32	0	1
🟧	Young Estuarine Dep - Clay	Undrained (Phi=0)	115	750				1



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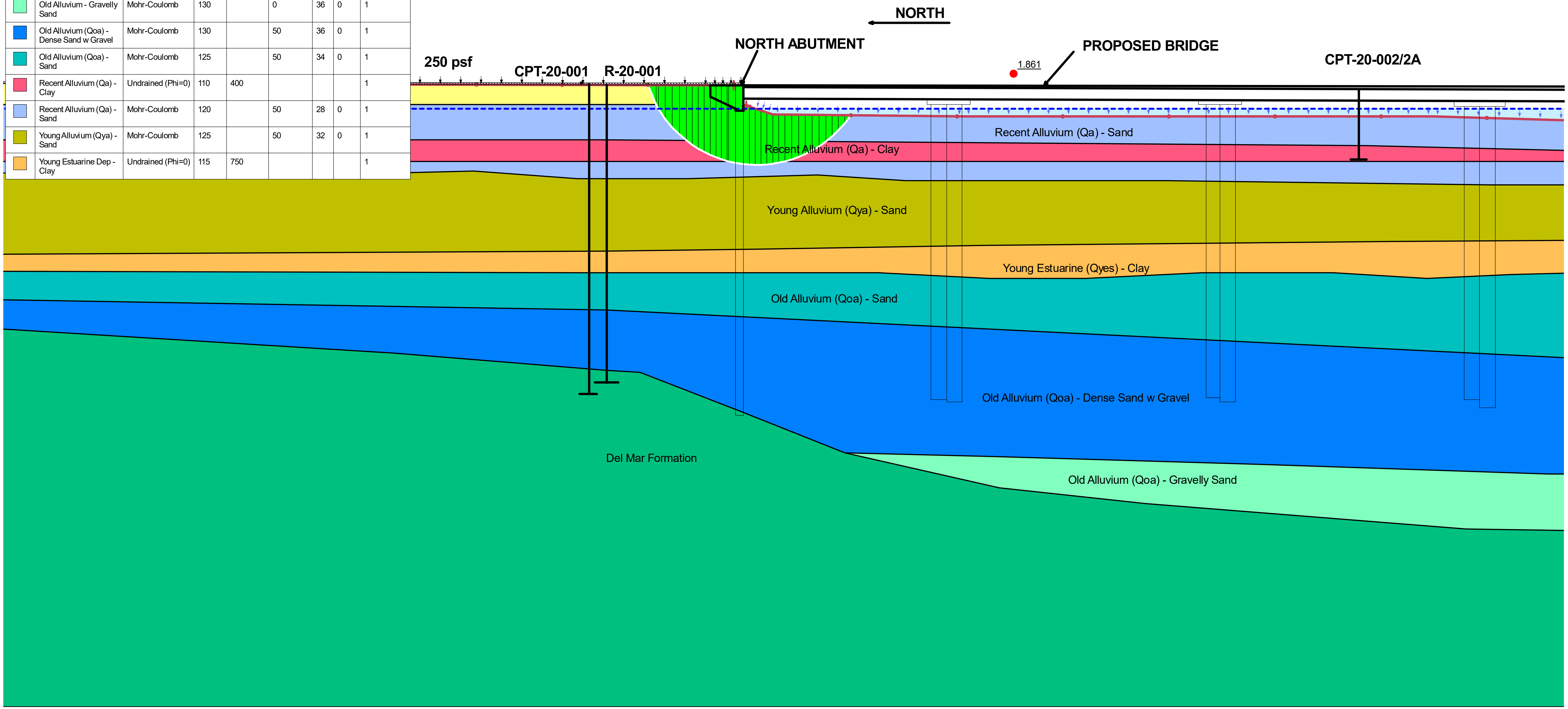
PROJECT NO.	20180876.001A
DRAWN:	04/2020
DRAWN BY:	JLB
CHECKED BY:	KR
FILE NAME:	-

SLOPE STABILITY ANALYSIS
NORTH ABUTMENT
SHORT-TERM STATIC ANALYSIS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
1

Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120		50	33	0	1
⬜	Concrete	High Strength	150					1
🟩	Del Mar Formation	Mohr-Coulomb	135		4,000	0	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130		0	36	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130		50	36	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125		50	34	0	1
🟥	Recent Alluvium (Qa) - Clay	Undrained (Phi=0)	110	400				1
🟦	Recent Alluvium (Qa) - Sand	Mohr-Coulomb	120		50	28	0	1
🟨	Young Alluvium (Qya) - Sand	Mohr-Coulomb	125		50	32	0	1
🟧	Young Estuarine Dep - Clay	Undrained (Phi=0)	115	750				1



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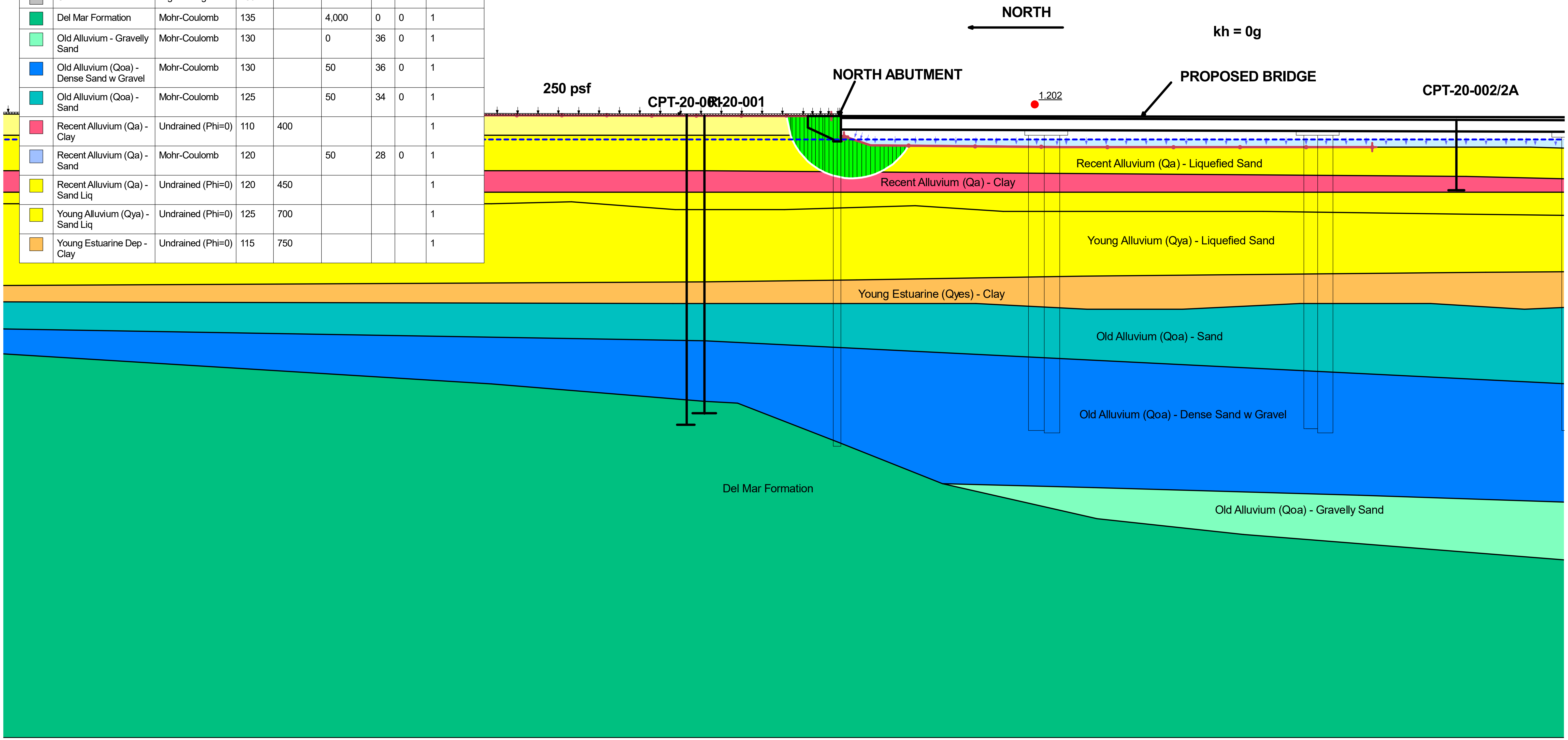
PROJECT NO. 20180876.001A
DRAWN: 04/2020
DRAWN BY: JLB
CHECKED BY: KR
FILE NAME: -

SLOPE STABILITY ANALYSIS
NORTH ABUTMENT
LONG-TERM STATIC ANALYSIS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
2



Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120		50	33	0	1
⬜	Concrete	High Strength	150					1
🟩	Del Mar Formation	Mohr-Coulomb	135		4,000	0	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130		0	36	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130		50	36	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125		50	34	0	1
🟥	Recent Alluvium (Qa) - Clay	Undrained (Phi=0)	110	400				1
🟦	Recent Alluvium (Qa) - Sand	Mohr-Coulomb	120		50	28	0	1
🟨	Recent Alluvium (Qa) - Sand Liq	Undrained (Phi=0)	120	450				1
🟨	Young Alluvium (Qya) - Sand Liq	Undrained (Phi=0)	125	700				1
🟧	Young Estuarine Dep - Clay	Undrained (Phi=0)	115	750				1

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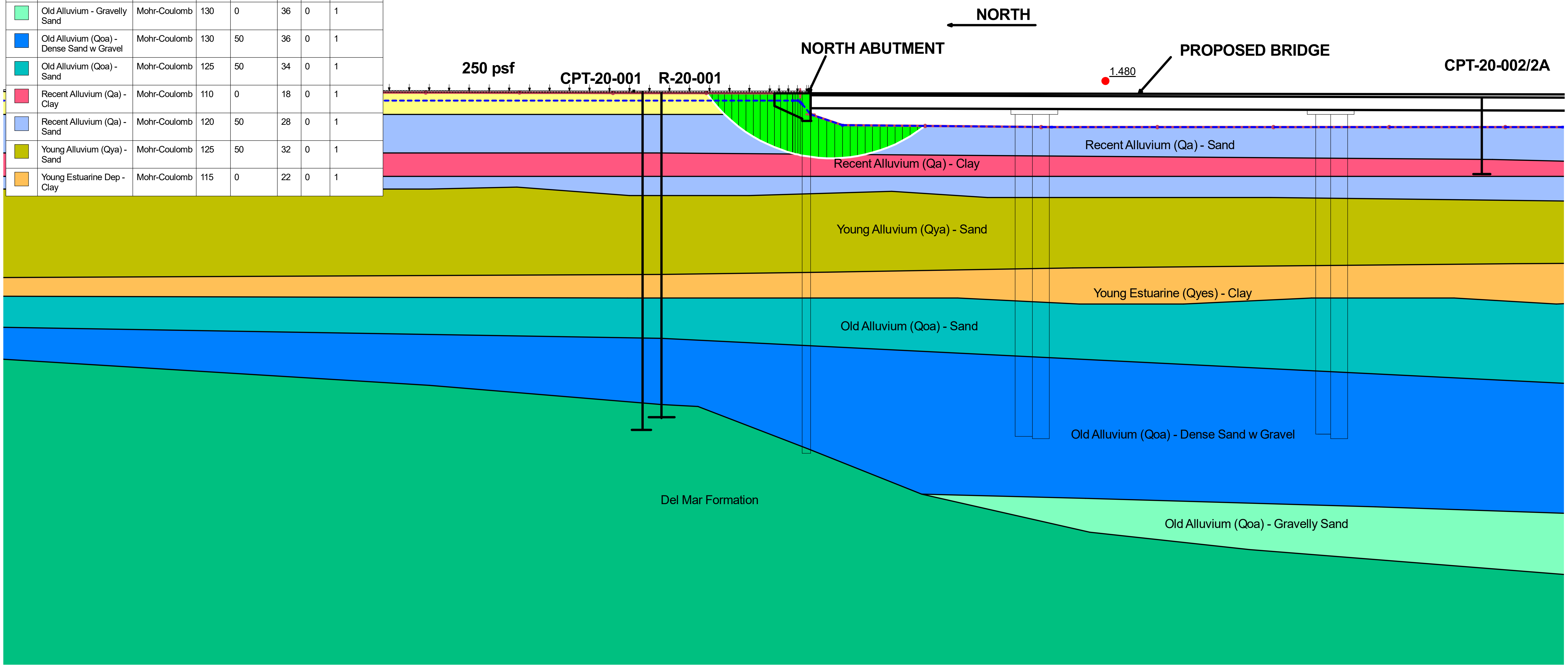
FILE NAME: -

SLOPE STABILITY ANALYSIS
NORTH ABUTMENT
POST-LIQUEFACTION ANALYSIS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
3

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120	50	33	0	1
⬜	Concrete	High Strength	150				1
🟩	Del Mar Formation	Mohr-Coulomb	135	0	36	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130	0	36	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130	50	36	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125	50	34	0	1
🟥	Recent Alluvium (Qa) - Clay	Mohr-Coulomb	110	0	18	0	1
🟦	Recent Alluvium (Qa) - Sand	Mohr-Coulomb	120	50	28	0	1
🟨	Young Alluvium (Qya) - Sand	Mohr-Coulomb	125	50	32	0	1
🟧	Young Estuarine Dep - Clay	Mohr-Coulomb	115	0	22	0	1



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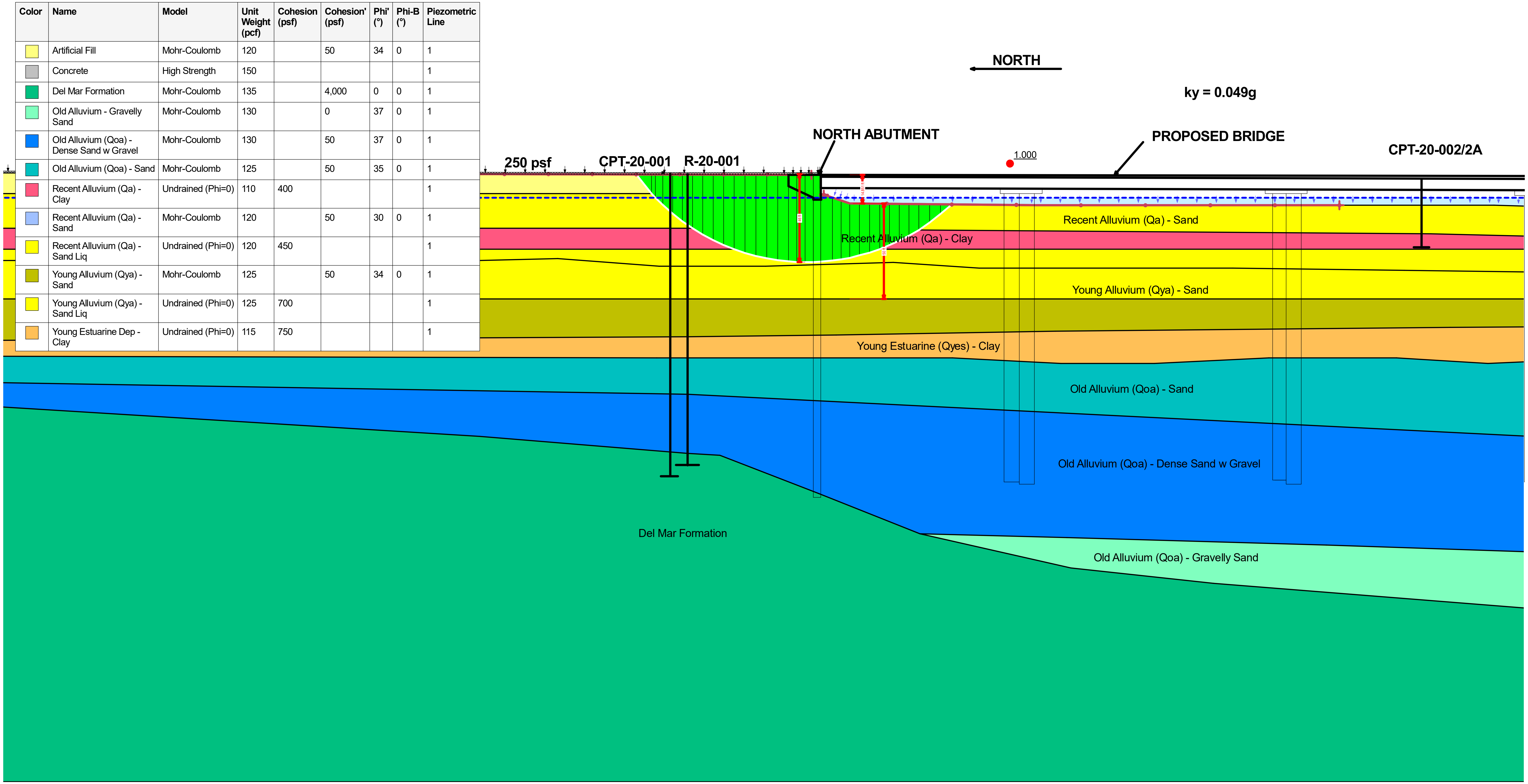
SLOPE STABILITY ANALYSIS NORTH ABUTMENT DESIGN FLOOD LEVEL RAPID DRAWDOWN

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
4



Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120		50	34	0	1
⬜	Concrete	High Strength	150					1
🟩	Del Mar Formation	Mohr-Coulomb	135		4,000	0	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130		0	37	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130		50	37	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125		50	35	0	1
🟥	Recent Alluvium (Qa) - Clay	Undrained (Phi=0)	110	400				1
🟦	Recent Alluvium (Qa) - Sand	Mohr-Coulomb	120		50	30	0	1
🟨	Recent Alluvium (Qa) - Sand Liq	Undrained (Phi=0)	120	450				1
🟨	Young Alluvium (Qya) - Sand	Mohr-Coulomb	125		50	34	0	1
🟨	Young Alluvium (Qya) - Sand Liq	Undrained (Phi=0)	125	700				1
🟧	Young Estuarine Dep - Clay	Undrained (Phi=0)	115	750				1

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SLOPE STABILITY ANALYSIS
NORTH ABUTMENT
PSEUDO-STATIC YIELD COEFFICIENT

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
5

North Abutment Seismic Slope Displacements

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.049		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.26	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.39	seconds	
Moment Magnitude (Mw)	6.6		
Spectral Acceleration (Sa(1.5Ts))	0.913	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	15.24	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	96.49	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

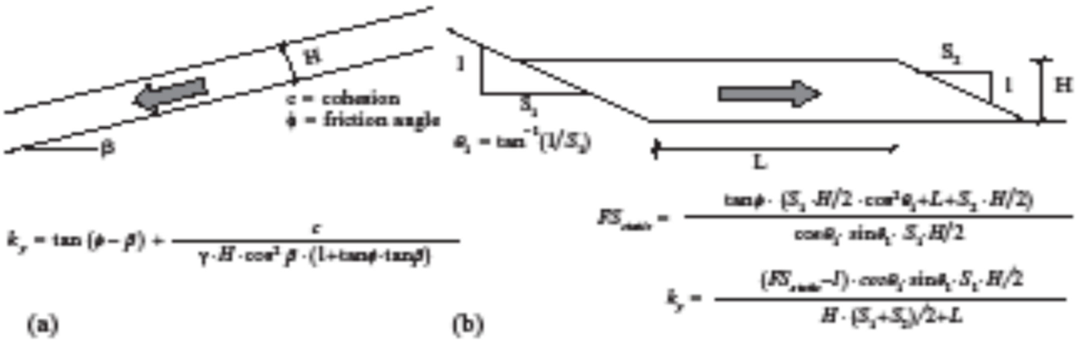
Results			
Probability of Negligible Displ. (P(D=0))	0.000		eq. (3)
D1	50.05	cm	calc. using eq. (7)
D2	96.49	cm	calc. using eq. (7)
D3	186.01	cm	calc. using eq. (7)
P(D>d_threshold)	0.997		eq. (7)

Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Median slope displacement = 96.5 cm = 38 inches = 3.2 ft

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, Vol 133, No. 4, pp. 381-392, April 2007



$$k_y = \tan(\phi - \beta) + \frac{c}{\gamma \cdot H \cdot \cos^2\beta \cdot (1 + \tan\phi \cdot \tan\beta)}$$

$$FS_{static} = \frac{\tan\phi \cdot (S_1 \cdot H/2 \cdot \cos^2\theta_1 + L + S_2 \cdot H/2)}{\cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}$$

$$k_y = \frac{(FS_{static} - 1) \cdot \cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}{H \cdot (S_1 + S_2)/2 + L}$$

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

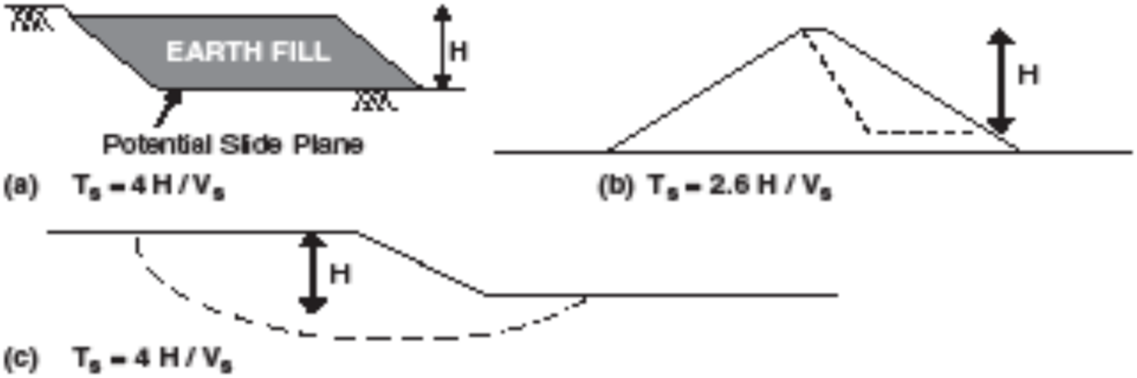
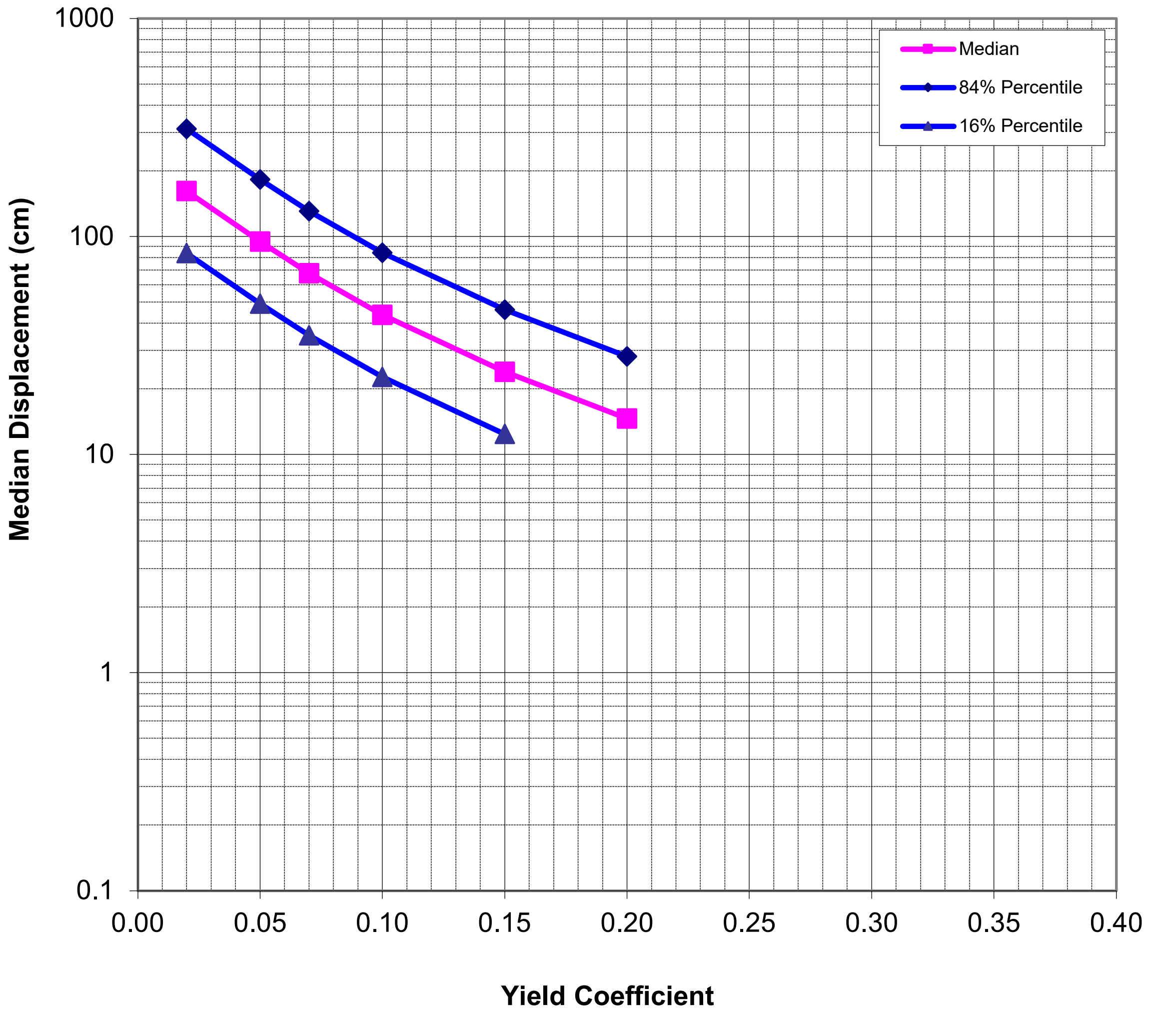


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

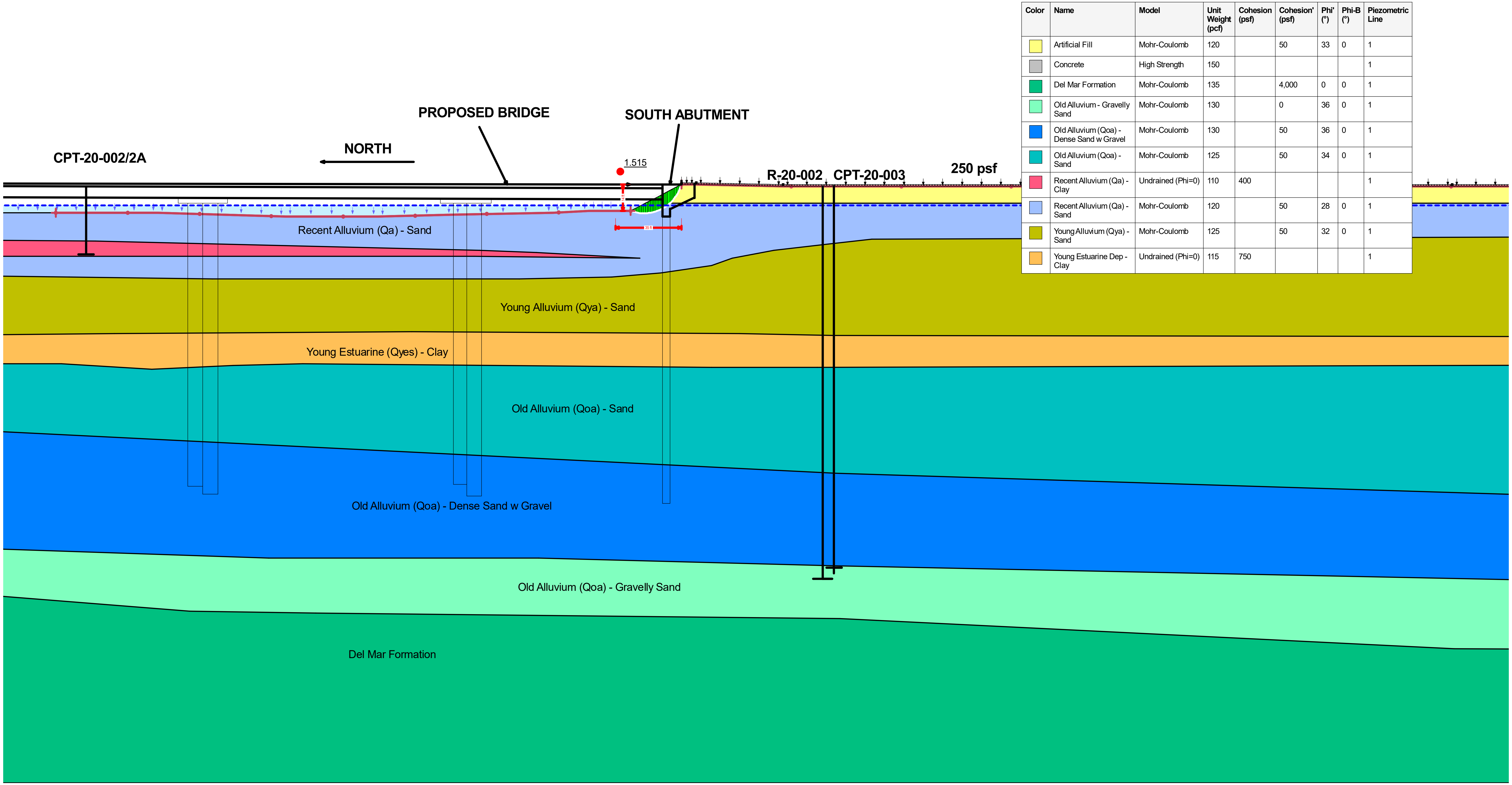
Figures from Bray, J.D. (2007) “Chapter 14: Simplified Seismic Slope Displacement Procedures,” Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, Vol. 6, Pitilakis, Kyriazis D., Ed., Springer, Vol. 6, pp. 327-353.

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	161.5	161.5	311.2	83.8
0.05	0.00	94.8	94.8	182.7	49.2
0.07	0.00	67.7	67.7	130.6	35.1
0.1	0.00	43.7	43.7	84.2	22.7
0.15	0.00	24.0	24.0	46.2	12.4
0.2	0.00	14.6	14.6	28.2	7.6
0.3	0.03	6.6	6.5	12.7	3.2
0.4	0.16	3.6	3.0	6.3	<1



Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, Vol 133, No. 4, pp. 381-392, April 2007



Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120		50	33	0	1
⬜	Concrete	High Strength	150					1
🟩	Del Mar Formation	Mohr-Coulomb	135		4,000	0	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130		0	36	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130		50	36	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125		50	34	0	1
🟥	Recent Alluvium (Qa) - Clay	Undrained (Phi=0)	110	400				1
🟦	Recent Alluvium (Qa) - Sand	Mohr-Coulomb	120		50	28	0	1
🟨	Young Alluvium (Qya) - Sand	Mohr-Coulomb	125		50	32	0	1
🟧	Young Estuarine Dep - Clay	Undrained (Phi=0)	115	750				1

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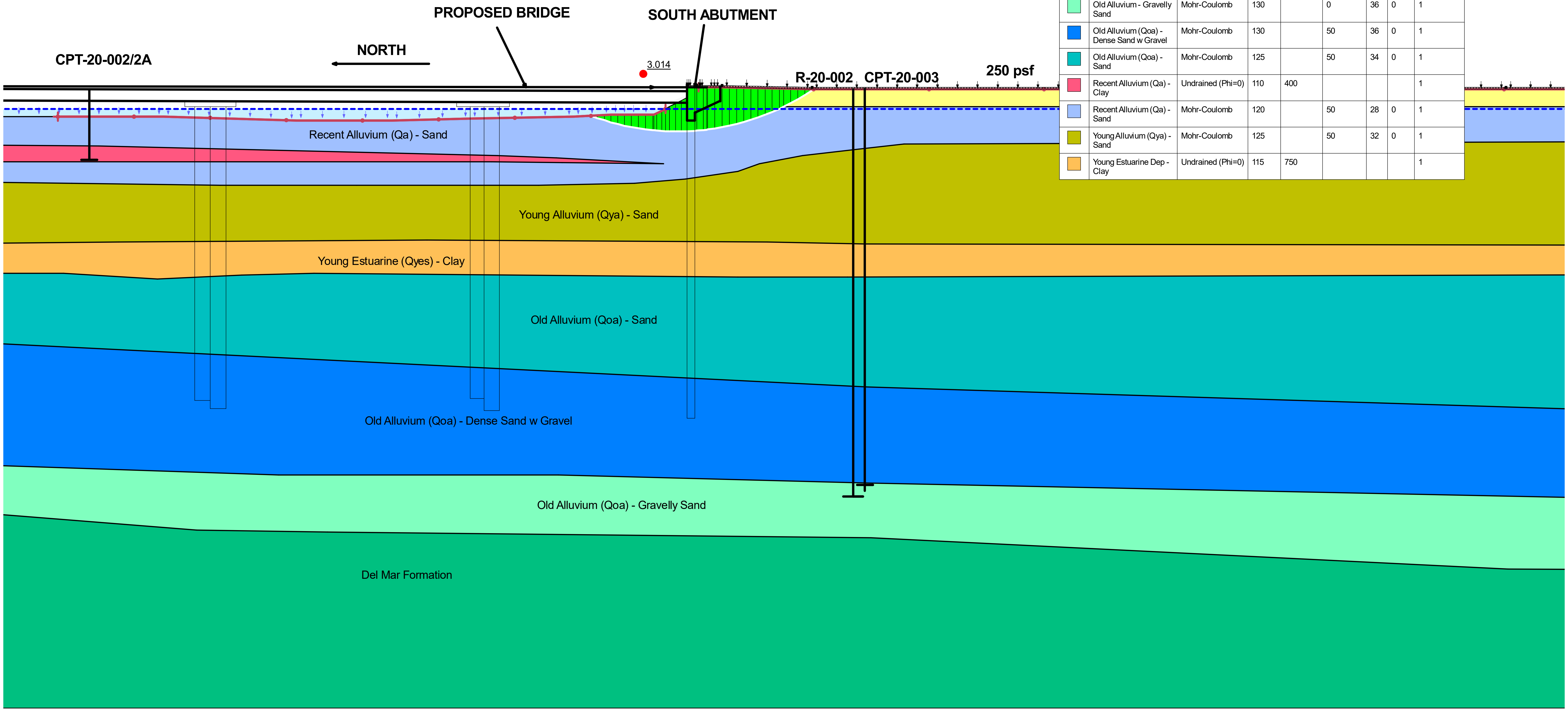
SLOPE STABILITY ANALYSIS SOUTH ABUTMENT SHORT-TERM STATIC ANALYSIS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
6



Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120		50	33	0	1
⬜	Concrete	High Strength	150					1
🟩	Del Mar Formation	Mohr-Coulomb	135		4,000	0	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130		0	36	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130		50	36	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125		50	34	0	1
🟥	Recent Alluvium (Qa) - Clay	Undrained (Phi=0)	110	400				1
🟦	Recent Alluvium (Qa) - Sand	Mohr-Coulomb	120		50	28	0	1
🟨	Young Alluvium (Qya) - Sand	Mohr-Coulomb	125		50	32	0	1
🟧	Young Estuarine Dep - Clay	Undrained (Phi=0)	115	750				1

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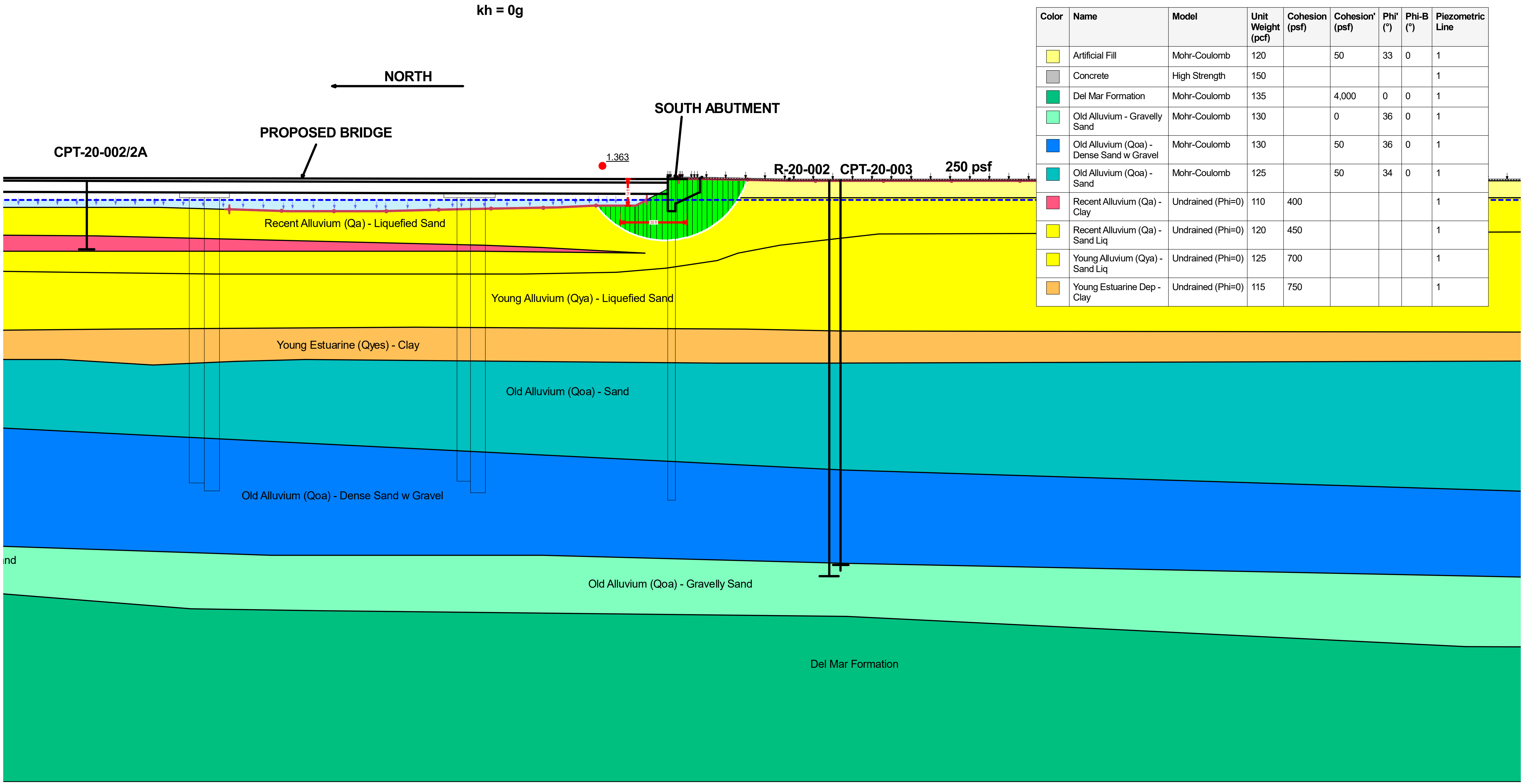
FILE NAME:	-

SLOPE STABILITY ANALYSIS SOUTH ABUTMENT LONG-TERM STATIC ANALYSIS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
7



Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120		50	33	0	1
⬜	Concrete	High Strength	150					1
🟩	Del Mar Formation	Mohr-Coulomb	135		4,000	0	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130		0	36	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130		50	36	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125		50	34	0	1
🟥	Recent Alluvium (Qa) - Clay	Undrained (Phi=0)	110	400				1
🟨	Recent Alluvium (Qa) - Sand Liq	Undrained (Phi=0)	120	450				1
🟨	Young Alluvium (Qya) - Sand Liq	Undrained (Phi=0)	125	700				1
🟧	Young Estuarine Dep - Clay	Undrained (Phi=0)	115	750				1

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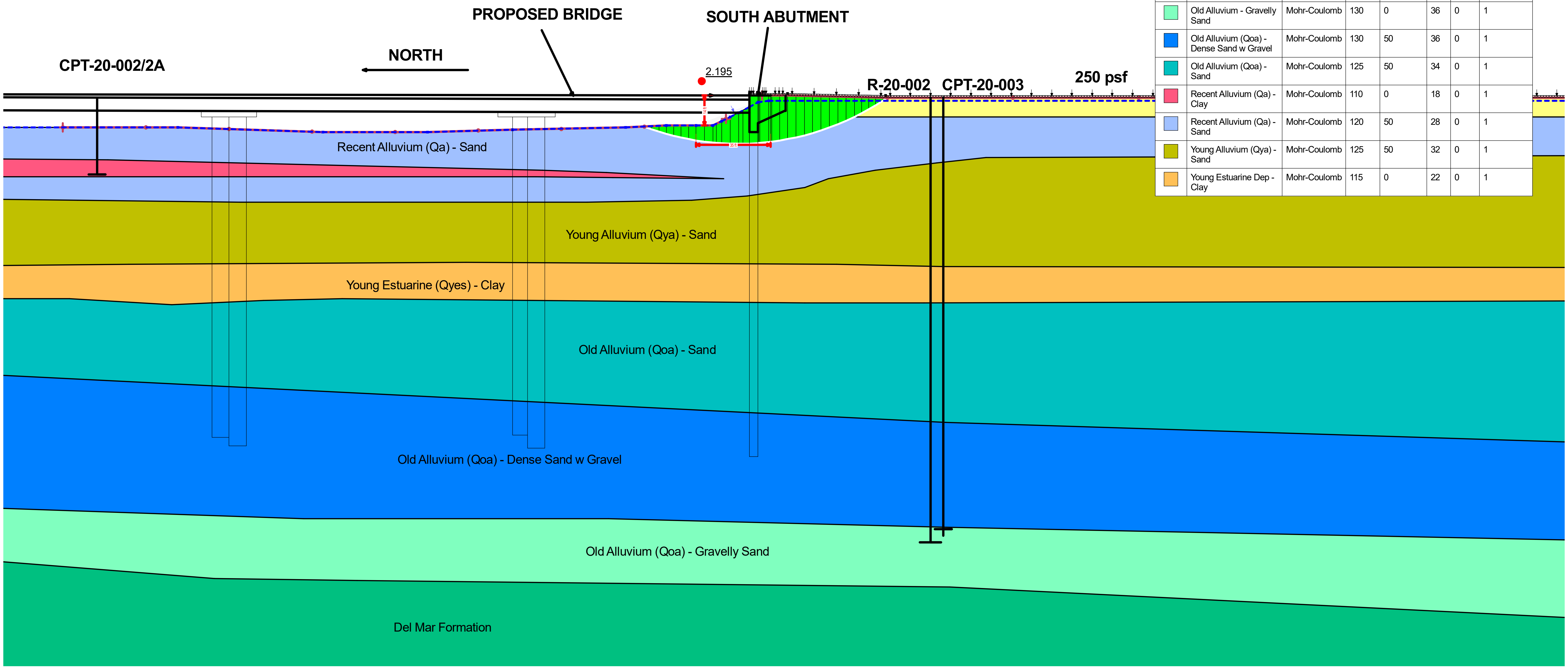
SLOPE STABILITY ANALYSIS
SOUTH ABUTMENT
POST-LIQUEFACTION ANALYSIS

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
8



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120	50	33	0	1
⬜	Concrete	High Strength	150				1
🟩	Del Mar Formation	Mohr-Coulomb	135	0	36	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130	0	36	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130	50	36	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125	50	34	0	1
🟥	Recent Alluvium (Qa) - Clay	Mohr-Coulomb	110	0	18	0	1
🟦	Recent Alluvium (Qa) - Sand	Mohr-Coulomb	120	50	28	0	1
🟨	Young Alluvium (Qya) - Sand	Mohr-Coulomb	125	50	32	0	1
🟧	Young Estuarine Dep - Clay	Mohr-Coulomb	115	0	22	0	1

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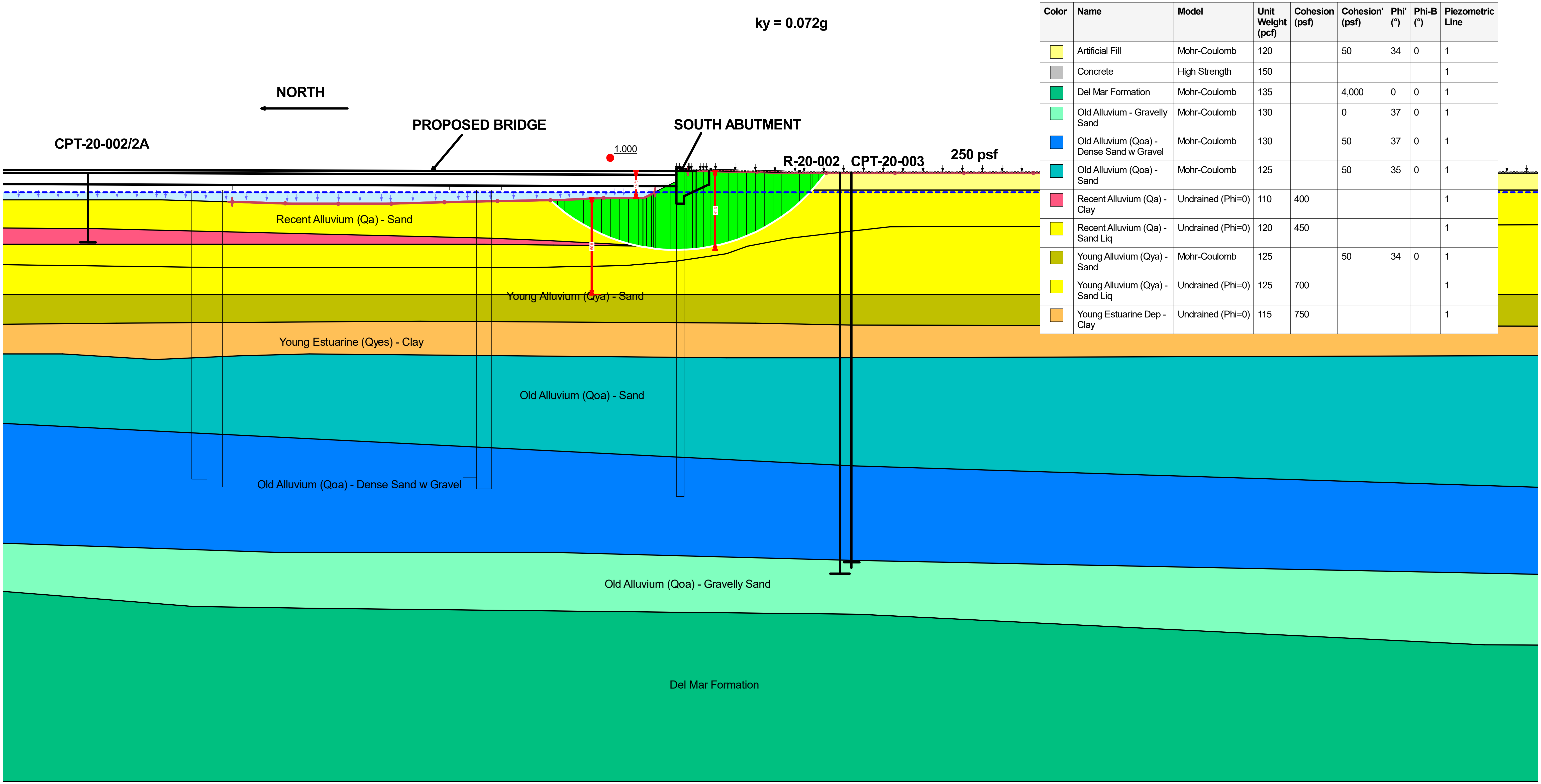
SLOPE STABILITY ANALYSIS
SOUTH ABUTMENT DESIGN FLOOD LEVEL
RAPID DRAWDOWN

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
9



Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
🟨	Artificial Fill	Mohr-Coulomb	120		50	34	0	1
⬜	Concrete	High Strength	150					1
🟩	Del Mar Formation	Mohr-Coulomb	135		4,000	0	0	1
🟩	Old Alluvium - Gravelly Sand	Mohr-Coulomb	130		0	37	0	1
🟦	Old Alluvium (Qoa) - Dense Sand w Gravel	Mohr-Coulomb	130		50	37	0	1
🟦	Old Alluvium (Qoa) - Sand	Mohr-Coulomb	125		50	35	0	1
🟥	Recent Alluvium (Qa) - Clay	Undrained (Phi=0)	110	400				1
🟨	Recent Alluvium (Qa) - Sand Liq	Undrained (Phi=0)	120	450				1
🟫	Young Alluvium (Qya) - Sand	Mohr-Coulomb	125		50	34	0	1
🟨	Young Alluvium (Qya) - Sand Liq	Undrained (Phi=0)	125	700				1
🟧	Young Estuarine Dep - Clay	Undrained (Phi=0)	115	750				1

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SLOPE STABILITY ANALYSIS
SOUTH ABUTMENT
PSEUDO-STATIC YIELD COEFFICIENT

CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE:
10

South Abutment Seismic Slope Displacements

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.072		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.23	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.35	seconds	
Moment Magnitude (Mw)	6.6		
Spectral Acceleration (Sa(1.5Ts))	0.9	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	15.24	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	61.49	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

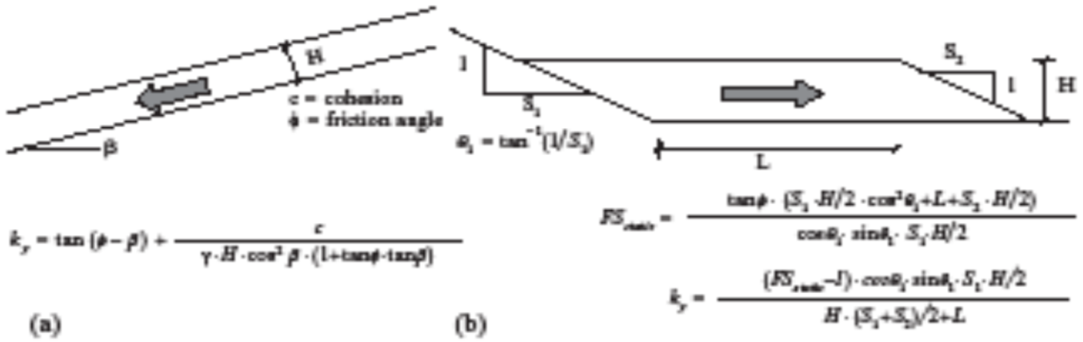
Results			
Probability of Negligible Displ. (P(D=0))	0.000		eq. (3)
D1	31.90	cm	calc. using eq. (7)
D2	61.49	cm	calc. using eq. (7)
D3	118.54	cm	calc. using eq. (7)
P(D>d_threshold)	0.983		eq. (7)

Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Median slope displacement = 61.5 cm = 24 inches = 2 ft

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, Vol 133, No. 4, pp. 381-392, April 2007

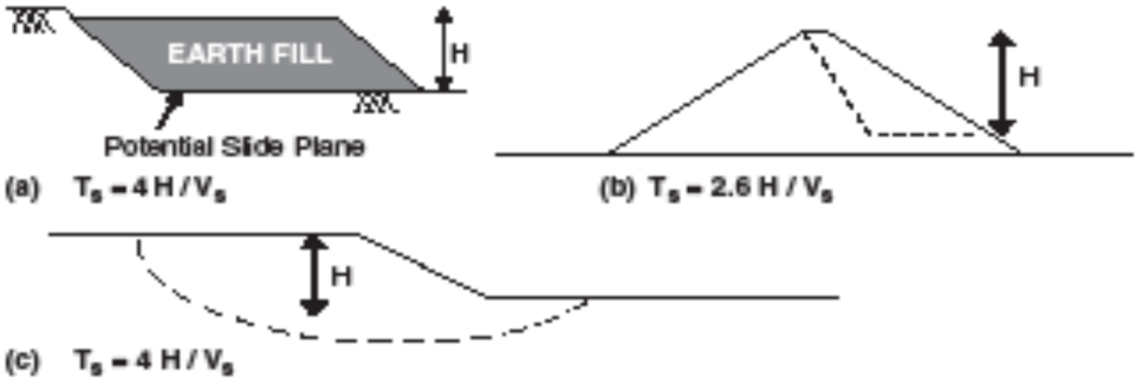


$$k_y = \tan(\phi - \beta) + \frac{c}{\gamma \cdot H \cdot \cos^2\beta \cdot (1 + \tan\phi \cdot \tan\beta)}$$

$$FS_{static} = \frac{\tan\phi \cdot (S_1 \cdot H/2 \cdot \cos^2\theta_1 + L + S_2 \cdot H/2)}{\cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}$$

$$k_y = \frac{(FS_{static} - 1) \cdot \cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}{H \cdot (S_1 + S_2)/2 + L}$$

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding



(a) $T_s = 4\,H / V_s$

(b) $T_s = 2.6\,H / V_s$

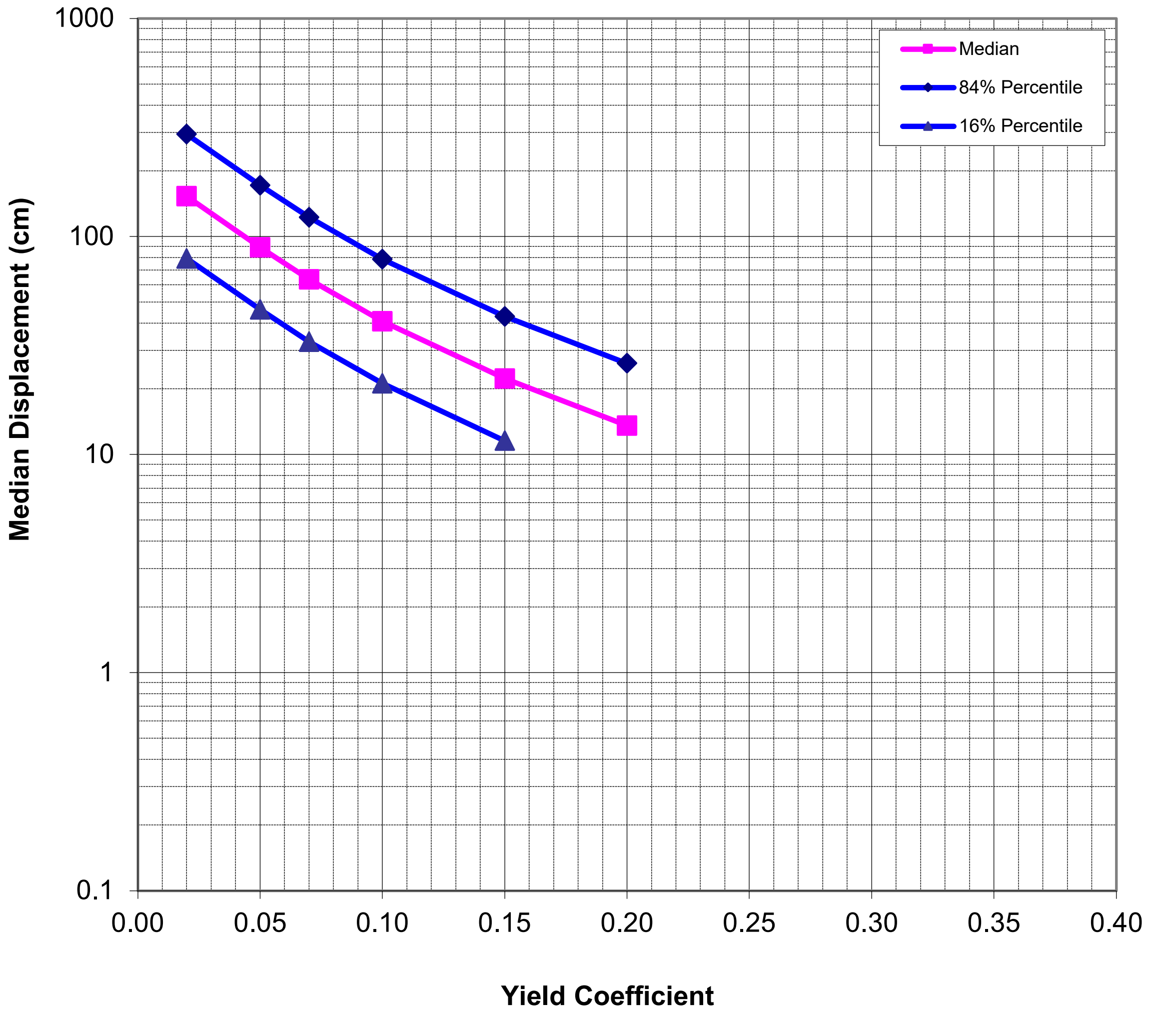
(c) $T_s = 4\,H / V_s$

Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray, J.D. (2007) “Chapter 14: Simplified Seismic Slope Displacement Procedures,” Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, Vol. 6, Pitilakis, Kyriazis D., Ed., Springer, Vol. 6, pp. 327-353.

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	152.8	152.8	294.6	79.3
0.05	0.00	89.1	89.1	171.7	46.2
0.07	0.00	63.5	63.5	122.4	32.9
0.1	0.00	40.8	40.8	78.7	21.2
0.15	0.00	22.3	22.3	43.0	11.6
0.2	0.00	13.6	13.6	26.2	7.0
0.3	0.03	6.2	6.0	11.7	3.0
0.4	0.18	3.3	2.7	5.8	<1



Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, Vol 133, No. 4, pp. 381-392, April 2007

G.5 AXIAL PILE CAPACITY ANALYSES

Strength Parameters for Axial Pile Capacity Analysis

NORTH ABUTMENT SUBSURFACE PARAMETERS USED IN AXIAL CAPACITY ANALYSES

Layer	Top Depth (ft)	Bottom Depth (ft)	Top Elev (ft)	Bottom Elev (ft)	Soil Model	Total Unit Weight	Friction Angle	Avg N Value	Top Beta, β	Bottom Beta, β	Adhesion factor, α	Undrained Shear Strength, Su (psf)	Steel Casing Reduction Factor[1]
1 - Artificial Fill	0	9	16	7	FHWA Sand	120	34	15	1.2	1.095	-	-	0
2 - Recent Alluvium (Qoa) (Sand)	9	28	7	-12	FHWA Sand	120	28	10	0.73	0.524	-	-	0
3 - Recent Alluvium (Qoa) (Clay)	28	39	-12	-23	FHWA Clay	110	-	-	-	-	0.55	400	0
4 - Recent Alluvium (Qoa) (Sand)	39	48	-23	-32	FHWA Sand	120	28	10	0.438	0.376	-	-	0
5 - Young Alluvium (Qya) (Sand)	48	85	-32	-69	FHWA Sand	125	32	30	0.565	0.255	-	-	0
6 - Young Estuarine (Qyes) (Clay)	85	95	-69	-79	FHWA Clay	115	-	-	-	-	0.55	750	0
7 - Old Alluvium (Qoa) (Sand)	95	116	-79	-100	FHWA Sand	125	34	25	0.25	0.25	-	-	1
8 - Old Alluvium (Qoa) (Sand w	116	146	-100	-130	FHWA Sand	130	36	35	0.25	0.25	-	-	1
9 - Del Mar Formation	146	266	-130	-250	FHWA Clay	135	-	-	-	-	0.51	4000	1

[1] Assumes casing extends to top of Old alluvial deposits for constructability. Geotechnical capacities are negelected for permanent steel smooth-wall casing per Caltrans Guildelines.

PIER SUBSURFACE PARAMETERS USED IN AXIAL CAPACITY ANALYSES

Layer	Top Depth (ft)	Bottom Depth (ft)	Top Elev (ft)	Bottom Elev (ft)	Top Depth from Pile Cut Off (ft)	Bottom Depth from Pile Cut Off (ft)	Soil Model	Total Unit Weight	Friction Angle*	Avg N Value	Top Beta, β	Bottom Beta, β	Adhesion factor, α	Undrained Shear Strength, Su (psf)	Steel Casing Reduction Factor[1]
1 - Recent Alluvium (Qoa) (Sand)	0	12	2	-10	-	-	FHWA Sand	120	28	10	1.00	0.69	-	-	0
2 - Recent Alluvium (Qoa) (Clay)	12	19	-10	-17	0	2	FHWA Clay	110	-	-	-	-	0.55	400	0
3 - Recent Alluvium (Qoa) (Sand)	19	35	-17	-33	2	18	FHWA Sand	120	28	10	0.61	0.47	-	-	0
4 - Young Alluvium (Qya) (Sand)	35	65	-33	-63	18	48	FHWA Sand	125	32	30	0.70	0.41	-	-	0
5 - Young Estuarine (Qyes) (Clay)	65	82	-63	-80	48	65	FHWA Clay	115	-	-	-	-	0.55	750	0
6 - Old Alluvium (Qoa) (Sand)	82	122	-80	-120	65	105	FHWA Sand	125	34	25	0.28	0.25	-	-	1
7 - Old Alluvium (Qoa) (Sand w	122	207	-120	-205	105	190	FHWA Sand	130	36	35	0.25	0.25	-	-	1
8 - Del Mar Formation	207	252	-205	-250	190	235	FHWA Clay	135	-	-	-	-	0.51	4000	1

[1] Assumes casing extends to top of Old alluvial deposits for constructability. Geotechnical capacities are negelected for permanent steel smooth-wall casing per Caltrans Guildelines.

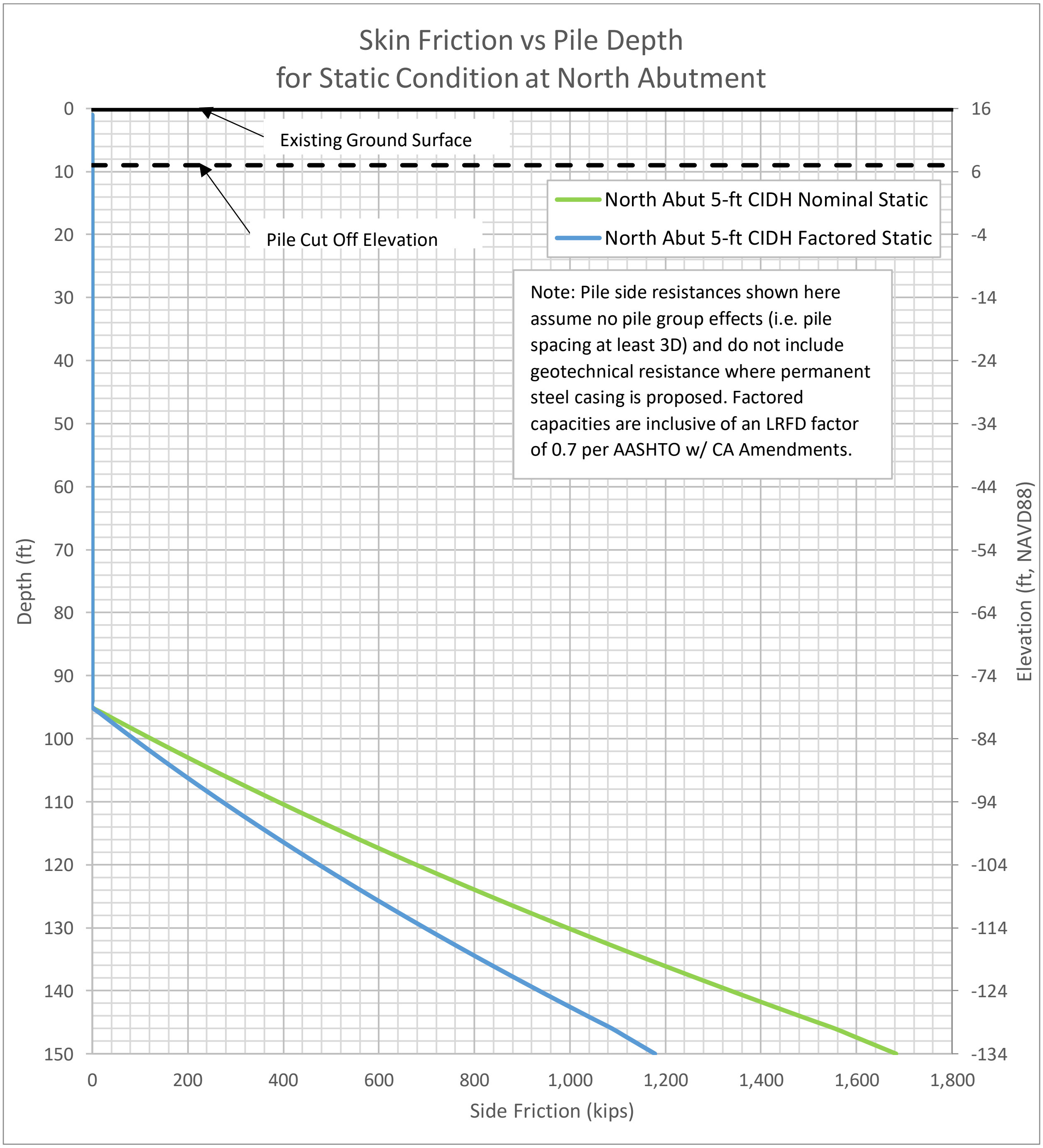
SOUTH ABUTMENT SUBSURFACE PARAMETERS USED IN AXIAL CAPACITY ANALYSES

Layer	Top Depth (ft)	Bottom Depth (ft)	Top Elev (ft)	Bottom Elev (ft)	Soil Model	Total Unit Weight	Friction Angle*	Avg N Value	Top Beta, β	Bottom Beta, β	Adhesion factor, α	Undrained Shear Strength, Su (psf)	Steel Casing Reduction Factor[1]
1 - Artificial Fill	0	9	16	7	FHWA Sand	120	34	15	1.20	1.10	-	-	0
2 - Recent Alluvium (Qoa) (Sand)	9	30	7	-14	FHWA Sand	120	28	10	0.73	0.51	-	-	0
3 - Young Alluvium (Qya) (Sand)	30	78	-14	-62	FHWA Sand	125	32	30	0.76	0.31	-	-	0
4 - Young Estuarine (Qyes) (Clay)	78	94	-62	-78	FHWA Clay	115	-	-	-	-	0.55	750	0
5 - Old Alluvium (Qoa) (Sand)	94	152	-78	-136	FHWA Sand	125	34	25	0.25	0.25	-	-	1
6 - Old Alluvium (Qoa) (Sand w	152	201	-136	-185	FHWA Sand	130	36	35	0.25	0.25	-	-	1
7 - Old Alluvial Deposits (Qoa) (Gravelly Sand)	201	231	-185	-215	FHWA Sand	130	36	50	0.25	0.25	-	-	1
8 - Del Mar Formation	231	266	-215	-250	FHWA Clay	135	-	-	-	-	0.51	4000	1

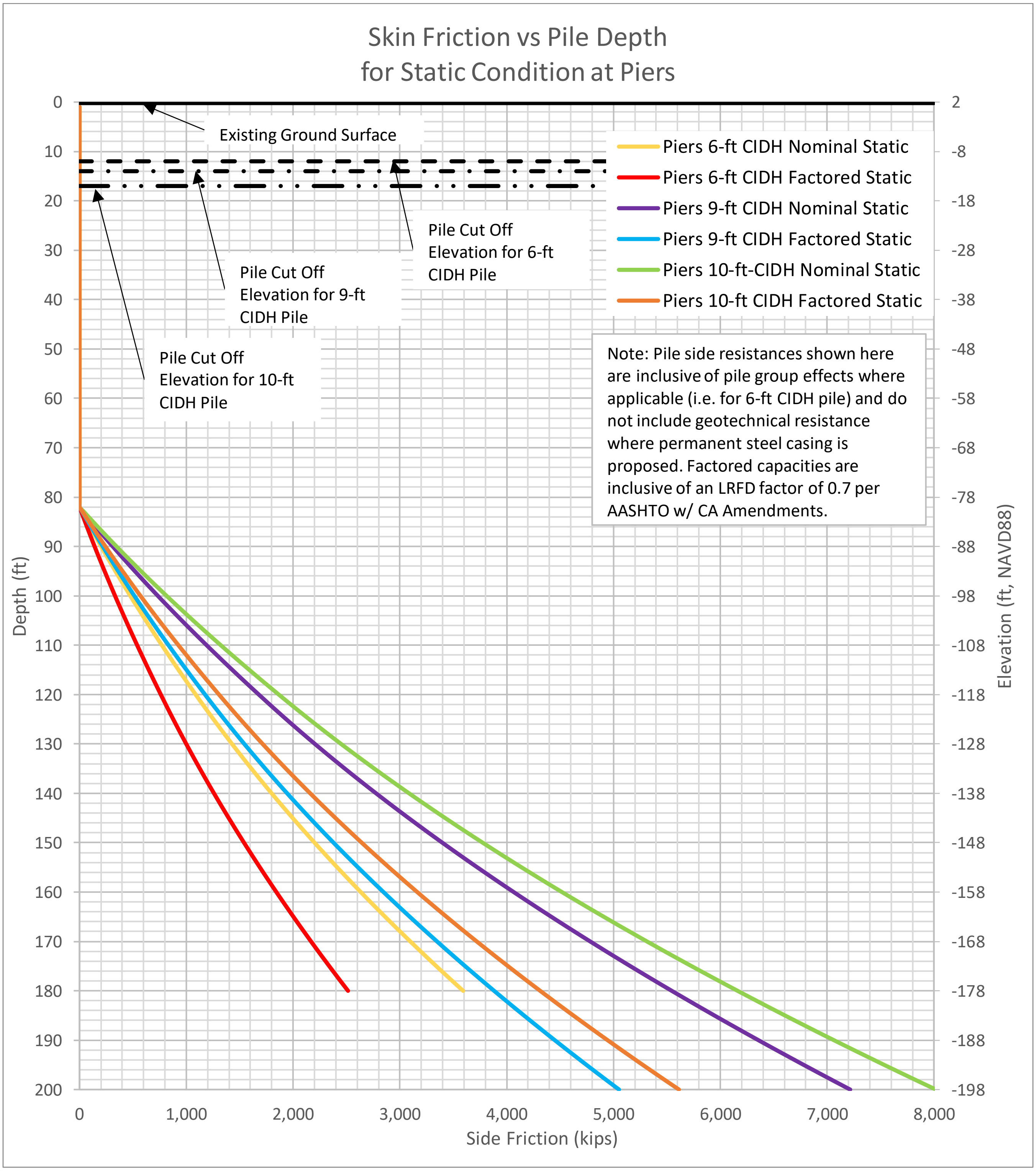
[1] Assumes casing extends to top of Old alluvial deposits for constructability. Geotechnical capacities are negelected for permanent steel smooth-wall casing per Caltrans Guildelines.

Axial Pile Capacity Charts

North Abutment Results

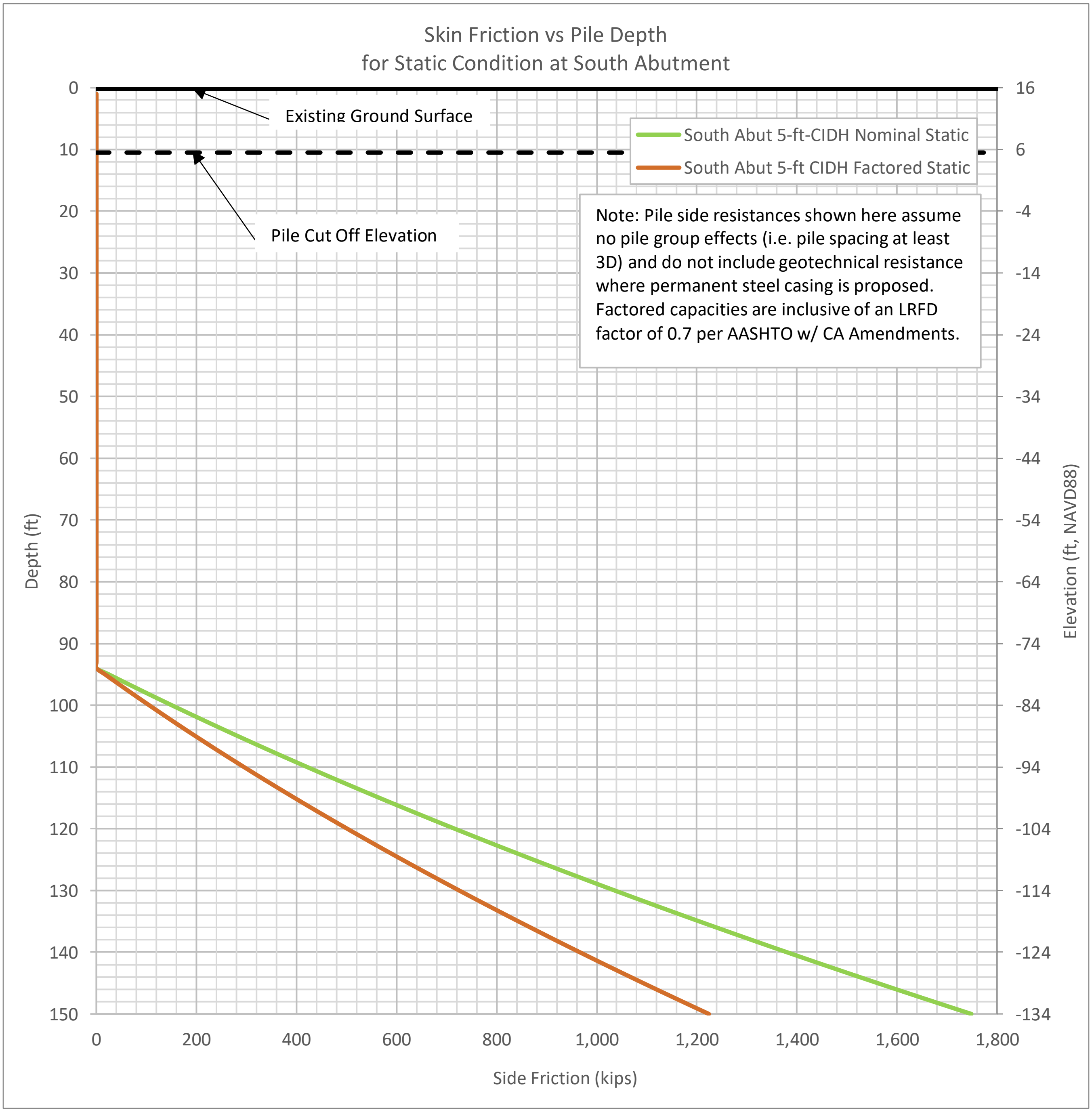


Pier Axial Capacity Results



Note: Pile side resistances shown here are inclusive of pile group effects where applicable (i.e. for 6-ft CIDH pile) and do not include geotechnical resistance where permanent steel casing is proposed. Factored capacities are inclusive of an LRFD factor of 0.7 per AASHTO w/ CA Amendments.

South Abutment Results



Note: Pile side resistances shown here assume no pile group effects (i.e. pile spacing at least 3D) and do not include geotechnical resistance where permanent steel casing is proposed. Factored capacities are inclusive of an LRFD factor of 0.7 per AASHTO w/ CA Amendments.

North Abutment
Static Axial Capacity
5-ft CIDH with 6-ft Casing

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=========================================================================

                 SHAFT for Windows, Version 2017.8.4

                  Serial Number :  253582343

               VERTICALLY LOADED DRILLED SHAFT ANALYSIS
                (c) Copyright ENSOFT, Inc., 1987-2017
                      All Rights Reserved

=========================================================================


Path to file locations      : C:\Users\JBonfiglio\OneDrive -
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
Name of input data file     : NorthAbut_Final_Rev.sf8d
Name of output file         : NorthAbut_Final_Rev.sf8o
Name of plot output file    : NorthAbut_Final_Rev.sf8p
Name of runtime file        : NorthAbut_Final_Rev.sf8r


-------------------------------------------------------------------------
                     Time and Date of Analysis
-------------------------------------------------------------------------

          Date:  May 18, 2020     Time:  09:08:59

Camino Del Mar Bridge Replacement - Phase 0


PROPOSED DEPTH =     150.0 FT
----------------


NUMBER OF LAYERS =    9
------------------


WATER TABLE DEPTH =      10.0 FT.
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SOIL INFORMATION
---------------

LAYER NO 1----SAND
```

```
  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.120E+01
  INTERNAL FRICTION ANGLE, DEG.                        = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.000E+00

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.110E+01
  INTERNAL FRICTION ANGLE, DEG.                        = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.900E+01

  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)              = 0.500E+00


LAYER NO 2----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.730E+00
  INTERNAL FRICTION ANGLE, DEG.                        = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.900E+01

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.524E+00
  INTERNAL FRICTION ANGLE, DEG.                        = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.280E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.700E+00
```

```
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)          = 0.500E+00


LAYER NO 3----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                  = 0.550E+00
  END BEARING COEFFICIENT-Nc                       = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT               = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST    = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                       = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT         = 0.100E+11
  DEPTH, FT                                        = 0.280E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                  = 0.550E+00
  END BEARING COEFFICIENT-Nc                       = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT               = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST    = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                       = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT         = 0.100E+11
  DEPTH, FT                                        = 0.390E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)           = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)          = 0.500E+00


LAYER NO 4----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                  = 0.438E+00
  INTERNAL FRICTION ANGLE, DEG.                    = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST    = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                       = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT         = 0.100E+11
  DEPTH, FT                                        = 0.390E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                  = 0.376E+00
  INTERNAL FRICTION ANGLE, DEG.                    = 0.280E+02
```

```
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.480E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 5----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.565E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.320E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.480E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.255E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.320E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.850E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 6----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.850E+02
```

```
  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                      = 0.550E+00
  END BEARING COEFFICIENT-Nc                           = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                   = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.950E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)              = 0.500E+00


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                        = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.950E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                        = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.116E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)              = 0.500E+00


LAYER NO 8----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                        = 0.360E+02
```

```
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.116E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.146E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 9----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.510E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.146E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.510E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.266E+03


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00
```

```
(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    5.000  FT.
 MAXIMUM SHAFT DIAMETER    =    5.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   95.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN






COMPUTATION RESULTS
-------------------




- CASE ANALYZED      :     1
  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1



DRILLED SHAFT INFORMATION

-------------------------

 DIAMETER OF STEM          =    5.000  FT.
 DIAMETER OF BASE          =    5.000  FT.
 END OF STEM TO BASE       =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   95.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 AREA OF ONE PERCENT STEEL =   28.278  SQ.IN.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
 VOLUME OF UNDERREAM       =    0.000  CU.YDS.
 SHAFT LENGTH              =  150.000  FT.


PREDICTED RESULTS
```

```
 ----------------

 QS      = ULTIMATE SIDE RESISTANCE;
 QB      = ULTIMATE BASE RESISTANCE;
 WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
 QU      = TOTAL ULTIMATE RESISTANCE;
 LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
           TO THE ULTIMATE SIDE RESISTANCE;
 LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
           TO THE ULTIMATE BASE RESISTANCE
 LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

 LENGTH  VOLUME     QS      QB       QU     LRFD QS  LRFD QB   LRFD QU
  (FT)   (CU.YDS) (TONS)   (TONS)  (TONS)   (TONS)   (TONS)    (TONS)
  96.0    69.82    12.08   174.58   186.66     8.46    87.29     95.75
  97.0    70.55    24.29   174.58   198.86    17.00    87.29    104.29
  98.0    71.28    36.61   174.58   211.19    25.63    87.29    112.92
  99.0    72.00    49.06   174.58   223.64    34.34    87.29    121.63
 100.0    72.73    61.64   174.58   236.21    43.15    87.29    130.43
 101.0    73.46    74.33   174.58   248.91    52.03    87.29    139.32
 102.0    74.19    87.15   174.58   261.73    61.01    87.29    148.30
 103.0    74.91   100.09   174.58   274.67    70.07    87.29    157.36
 104.0    75.64   113.16   174.58   287.74    79.21    87.29    166.50
 105.0    76.37   126.35   174.58   300.93    88.44    87.29    175.73
 106.0    77.10   139.66   174.58   314.24    97.76    87.29    185.05
 107.0    77.82   153.09   184.68   337.78   107.16    92.34    199.51
 108.0    78.55   166.65   195.71   362.36   116.65    97.86    214.51
 109.0    79.28   180.33   207.66   387.98   126.23   103.83    230.06
 110.0    80.00   194.13   220.52   414.65   135.89   110.26    246.15
 111.0    80.73   208.06   234.30   442.36   145.64   117.15    262.79
 112.0    81.46   222.11   245.33   467.43   155.47   122.66    278.14
 113.0    82.19   236.28   253.60   489.87   165.39   126.80    292.19
 114.0    82.91   250.57   259.11   509.68   175.40   129.55    304.95
 115.0    83.64   264.99   261.87   526.85   185.49   130.93    316.43
 116.0    84.37   279.53   261.87   541.40   195.67   130.93    326.60
 117.0    85.10   294.20   261.87   556.06   205.94   130.93    336.87
 118.0    85.82   309.00   261.87   570.86   216.30   130.93    347.23
 119.0    86.55   323.93   261.87   585.80   226.75   130.93    357.68
 120.0    87.28   339.00   261.87   600.86   237.30   130.93    368.23
 121.0    88.01   354.20   261.87   616.06   247.94   130.93    378.87
 122.0    88.73   369.53   261.87   631.39   258.67   130.93    389.60
 123.0    89.46   384.99   261.87   646.86   269.50   130.93    400.43
 124.0    90.19   400.59   261.87   662.46   280.41   130.93    411.35
 125.0    90.91   416.32   261.87   678.19   291.42   130.93    422.36
 126.0    91.64   432.18   261.87   694.05   302.53   130.93    433.46
 127.0    92.37   448.18   261.87   710.04   313.73   130.93    444.66
 128.0    93.10   464.31   261.87   726.17   325.01   130.93    455.95
 129.0    93.82   480.57   261.87   742.43   336.40   130.93    467.33
```

```
130.0    94.55   496.96   261.87   758.83   347.87   130.93    478.81
131.0    95.28   513.49   261.87   775.35   359.44   130.93    490.37
132.0    96.01   530.15   261.87   792.01   371.10   130.93    502.04
133.0    96.73   546.94   261.87   808.81   382.86   130.93    513.79
134.0    97.46   563.86   261.87   825.73   394.71   130.93    525.64
135.0    98.19   580.92   261.87   842.79   406.65   130.93    537.58
136.0    98.91   598.11   261.87   859.98   418.68   130.93    549.61
137.0    99.64   615.44   272.47   887.91   430.80   136.24    567.04
138.0   100.37   632.89   284.05   916.94   443.02   142.02    585.05
139.0   101.10   650.48   296.58   947.06   455.34   148.29    603.63
140.0   101.82   668.20   310.08   978.28   467.74   155.04    622.78
141.0   102.55   686.06   324.55  1010.60   480.24   162.27    642.51
142.0   103.28   704.04   336.12  1040.16   492.83   168.06    660.89
143.0   104.01   722.16   344.80  1066.96   505.51   172.40    677.91
144.0   104.73   740.41   350.58  1091.00   518.29   175.29    693.58
145.0   105.46   758.80   353.48  1112.27   531.16   176.74    707.90
146.0   106.19   777.32   353.48  1130.79   544.12   176.74    720.86
147.0   106.92   793.34   353.48  1146.82   555.34   176.74    732.08
148.0   107.64   809.36   353.48  1162.84   566.56   176.74    743.29
149.0   108.37   825.39   353.48  1178.86   577.77   176.74    754.51
150.0   109.10   841.41   353.48  1194.89   588.99   176.74    765.73




 AXIAL LOAD VS SETTLEMENT CURVES
 -------------------------------



    RESULT FROM TREND (AVERAGED) LINE

    TOP   LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
       TON              IN.              TON             IN.
   0.5856E-01      0.2734E-04      0.4654E-02      0.1000E-04
   0.2928E+00      0.1367E-03      0.2327E-01      0.5000E-04
   0.5856E+00      0.2734E-03      0.4654E-01      0.1000E-03
   0.2952E+02      0.1373E-01      0.2327E+01      0.5000E-02
   0.4428E+02      0.2060E-01      0.3491E+01      0.7500E-02
   0.5904E+02      0.2747E-01      0.4654E+01      0.1000E-01
   0.1476E+03      0.6867E-01      0.1164E+02      0.2500E-01
   0.2934E+03      0.1369E+00      0.2327E+02      0.5000E-01
   0.4065E+03      0.1957E+00      0.3491E+02      0.7500E-01
   0.5086E+03      0.2512E+00      0.4654E+02      0.1000E+00
   0.8122E+03      0.4949E+00      0.1164E+03      0.2500E+00
   0.9751E+03      0.7967E+00      0.1785E+03      0.5000E+00
   0.9990E+03      0.9048E+00      0.1979E+03      0.6000E+00
   0.1102E+04      0.1841E+01      0.3058E+03      0.1500E+01
   0.1137E+04      0.3354E+01      0.3429E+03      0.3000E+01
```

```
  RESULT FROM UPPER-BOUND LINE

  TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
     TON             IN.              TON             IN.
 0.8303E-01      0.3453E-04      0.6952E-02      0.1000E-04
 0.4152E+00      0.1727E-03      0.3476E-01      0.5000E-04
 0.8303E+00      0.3453E-03      0.6952E-01      0.1000E-03
 0.4198E+02      0.1739E-01      0.3476E+01      0.5000E-02
 0.6297E+02      0.2609E-01      0.5214E+01      0.7500E-02
 0.8396E+02      0.3479E-01      0.6952E+01      0.1000E-01
 0.2099E+03      0.8696E-01      0.1738E+02      0.2500E-01
 0.4130E+03      0.1723E+00      0.3476E+02      0.5000E-01
 0.5625E+03      0.2420E+00      0.5214E+02      0.7500E-01
 0.6880E+03      0.3048E+00      0.6952E+02      0.1000E+00
 0.9816E+03      0.5480E+00      0.1738E+03      0.2500E+00
 0.1083E+04      0.8324E+00      0.2463E+03      0.5000E+00
 0.1101E+04      0.9390E+00      0.2651E+03      0.6000E+00
 0.1172E+04      0.1864E+01      0.3393E+03      0.1500E+01
 0.1184E+04      0.3368E+01      0.3517E+03      0.3000E+01


  RESULT FROM LOWER-BOUND LINE

  TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
     TON             IN.              TON             IN.
 0.3588E-01      0.2062E-04      0.2357E-02      0.1000E-04
 0.1794E+00      0.1031E-03      0.1178E-01      0.5000E-04
 0.3588E+00      0.2062E-03      0.2356E-01      0.1000E-03
 0.1803E+02      0.1033E-01      0.1178E+01      0.5000E-02
 0.2705E+02      0.1550E-01      0.1767E+01      0.7500E-02
 0.3607E+02      0.2067E-01      0.2357E+01      0.1000E-01
 0.9017E+02      0.5167E-01      0.5891E+01      0.2500E-01
 0.1803E+03      0.1033E+00      0.1178E+02      0.5000E-01
 0.2579E+03      0.1514E+00      0.1767E+02      0.7500E-01
 0.3317E+03      0.1984E+00      0.2357E+02      0.1000E+00
 0.6349E+03      0.4394E+00      0.5891E+02      0.2500E+00
 0.8668E+03      0.7608E+00      0.1108E+03      0.5000E+00
 0.8961E+03      0.8705E+00      0.1308E+03      0.6000E+00
 0.1032E+04      0.1819E+01      0.2722E+03      0.1500E+01
 0.1087E+04      0.3338E+01      0.3323E+03      0.3000E+01
```

North Abutment
Seismic Axial Capacity
5-ft CIDH with 6-ft Casing

```
==========================================================================

                 SHAFT for Windows, Version 2017.8.4

                  Serial Number :  253582343

               VERTICALLY LOADED DRILLED SHAFT ANALYSIS
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==========================================================================


     Path to file locations      : C:\Users\JBonfiglio\OneDrive -
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
     Name of input data file     : NorthAbut_Seismic_Final_Rev.sf8d
     Name of output file         : NorthAbut_Seismic_Final_Rev.sf8o
     Name of plot output file    : NorthAbut_Seismic_Final_Rev.sf8p
     Name of runtime file        : NorthAbut_Seismic_Final_Rev.sf8r


--------------------------------------------------------------------------
                     Time and Date of Analysis
--------------------------------------------------------------------------

          Date:  May 18, 2020     Time:  11:16:38

     Camino Del Mar Bridge Replacement - Phase 0


     PROPOSED DEPTH =     150.0 FT
     ----------------


     NUMBER OF LAYERS =    9
     ------------------


     WATER TABLE DEPTH =      10.0 FT.
     -------------------




     SOIL INFORMATION
     ---------------

     LAYER NO 1----SAND
```

```
  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.120E+01
  INTERNAL FRICTION ANGLE, DEG.                      = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.000E+00

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.110E+01
  INTERNAL FRICTION ANGLE, DEG.                      = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.900E+01

  LRFD RESISTANCE FACTOR (SIDE FRICTION)             = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)            = 0.100E+01


LAYER NO 2----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.816E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.900E+01


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.280E+02
```

```
  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 3----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.280E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.390E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 4----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.390E+02


  AT THE BOTTOM
```

```
  STRENGTH REDUCTION FACTOR-ALPHA                      = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                           = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                   = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.480E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)              = 0.100E+01


LAYER NO 5----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                      = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                           = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                   = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.480E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                      = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                           = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                   = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.850E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)              = 0.100E+01


LAYER NO 6----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                      = 0.550E+00
  END BEARING COEFFICIENT-Nc                           = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                   = 0.750E+03
```

```
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.850E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.950E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.950E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.116E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 8----SAND
```

```
  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                     = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                       = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.116E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                     = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                       = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.146E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)              = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)             = 0.100E+01


LAYER NO 9----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                     = 0.510E+00
  END BEARING COEFFICIENT-Nc                          = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                  = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.146E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                     = 0.510E+00
  END BEARING COEFFICIENT-Nc                          = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                  = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.266E+03
```

```
  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01



(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    5.000  FT.
 MAXIMUM SHAFT DIAMETER    =    5.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   95.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN






COMPUTATION RESULTS
-------------------




- CASE ANALYZED      :     1
  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1



DRILLED SHAFT INFORMATION

-------------------------

 DIAMETER OF STEM          =    5.000  FT.
 DIAMETER OF BASE          =    5.000  FT.
 END OF STEM TO BASE       =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   95.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 AREA OF ONE PERCENT STEEL =   28.278  SQ.IN.
```

```
  ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
  VOLUME OF UNDERREAM       =    0.000  CU.YDS.
  SHAFT LENGTH              =  150.000  FT.


 PREDICTED RESULTS
 -----------------

 QS      = ULTIMATE SIDE RESISTANCE;
 QB      = ULTIMATE BASE RESISTANCE;
 WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
 QU      = TOTAL ULTIMATE RESISTANCE;
 LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
           TO THE ULTIMATE SIDE RESISTANCE;
 LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
           TO THE ULTIMATE BASE RESISTANCE
 LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

 LENGTH  VOLUME     QS      QB      QU     LRFD QS  LRFD QB   LRFD QU
  (FT)   (CU.YDS) (TONS)   (TONS)  (TONS)   (TONS)   (TONS)    (TONS)
  96.0    69.82    12.08   174.58  186.66    12.08   174.58    186.66
  97.0    70.55    24.29   174.58  198.86    24.29   174.58    198.86
  98.0    71.28    36.61   174.58  211.19    36.61   174.58    211.19
  99.0    72.00    49.06   174.58  223.64    49.06   174.58    223.64
 100.0    72.73    61.64   174.58  236.21    61.64   174.58    236.21
 101.0    73.46    74.33   174.58  248.91    74.33   174.58    248.91
 102.0    74.19    87.15   174.58  261.73    87.15   174.58    261.73
 103.0    74.91   100.09   174.58  274.67   100.09   174.58    274.67
 104.0    75.64   113.16   174.58  287.74   113.16   174.58    287.74
 105.0    76.37   126.35   174.58  300.93   126.35   174.58    300.93
 106.0    77.10   139.66   174.58  314.24   139.66   174.58    314.24
 107.0    77.82   153.09   184.68  337.78   153.09   184.68    337.78
 108.0    78.55   166.65   195.71  362.36   166.65   195.71    362.36
 109.0    79.28   180.33   207.66  387.98   180.33   207.66    387.98
 110.0    80.00   194.13   220.52  414.65   194.13   220.52    414.65
 111.0    80.73   208.06   234.30  442.36   208.06   234.30    442.36
 112.0    81.46   222.11   245.33  467.43   222.11   245.33    467.43
 113.0    82.19   236.28   253.60  489.87   236.28   253.60    489.87
 114.0    82.91   250.57   259.11  509.68   250.57   259.11    509.68
 115.0    83.64   264.99   261.87  526.85   264.99   261.87    526.85
 116.0    84.37   279.53   261.87  541.40   279.53   261.87    541.40
 117.0    85.10   294.20   261.87  556.06   294.20   261.87    556.06
 118.0    85.82   309.00   261.87  570.86   309.00   261.87    570.86
 119.0    86.55   323.93   261.87  585.80   323.93   261.87    585.80
 120.0    87.28   339.00   261.87  600.86   339.00   261.87    600.86
 121.0    88.01   354.20   261.87  616.06   354.20   261.87    616.06
 122.0    88.73   369.53   261.87  631.39   369.53   261.87    631.39
 123.0    89.46   384.99   261.87  646.86   384.99   261.87    646.86
```

```
124.0    90.19   400.59   261.87   662.46   400.59   261.87    662.46
125.0    90.91   416.32   261.87   678.19   416.32   261.87    678.19
126.0    91.64   432.18   261.87   694.05   432.18   261.87    694.05
127.0    92.37   448.18   261.87   710.04   448.18   261.87    710.04
128.0    93.10   464.31   261.87   726.17   464.31   261.87    726.17
129.0    93.82   480.57   261.87   742.43   480.57   261.87    742.43
130.0    94.55   496.96   261.87   758.83   496.96   261.87    758.83
131.0    95.28   513.49   261.87   775.35   513.49   261.87    775.35
132.0    96.01   530.15   261.87   792.01   530.15   261.87    792.01
133.0    96.73   546.94   261.87   808.81   546.94   261.87    808.81
134.0    97.46   563.86   261.87   825.73   563.86   261.87    825.73
135.0    98.19   580.92   261.87   842.79   580.92   261.87    842.79
136.0    98.91   598.11   261.87   859.98   598.11   261.87    859.98
137.0    99.64   615.44   272.47   887.91   615.44   272.47    887.91
138.0   100.37   632.89   284.05   916.94   632.89   284.05    916.94
139.0   101.10   650.48   296.58   947.06   650.48   296.58    947.06
140.0   101.82   668.20   310.08   978.28   668.20   310.08    978.28
141.0   102.55   686.06   324.55  1010.60   686.06   324.55   1010.60
142.0   103.28   704.04   336.12  1040.16   704.04   336.12   1040.16
143.0   104.01   722.16   344.80  1066.96   722.16   344.80   1066.96
144.0   104.73   740.41   350.58  1091.00   740.41   350.58   1091.00
145.0   105.46   758.80   353.48  1112.27   758.80   353.48   1112.27
146.0   106.19   777.32   353.48  1130.79   777.32   353.48   1130.79
147.0   106.92   793.34   353.48  1146.82   793.34   353.48   1146.82
148.0   107.64   809.36   353.48  1162.84   809.36   353.48   1162.84
149.0   108.37   825.39   353.48  1178.86   825.39   353.48   1178.86
150.0   109.10   841.41   353.48  1194.89   841.41   353.48   1194.89



 AXIAL LOAD VS SETTLEMENT CURVES
 -------------------------------


    RESULT FROM TREND (AVERAGED) LINE

    TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
       TON             IN.              TON            IN.
   0.5856E-01      0.2734E-04      0.4654E-02      0.1000E-04
   0.2928E+00      0.1367E-03      0.2327E-01      0.5000E-04
   0.5856E+00      0.2734E-03      0.4654E-01      0.1000E-03
   0.2952E+02      0.1373E-01      0.2327E+01      0.5000E-02
   0.4428E+02      0.2060E-01      0.3491E+01      0.7500E-02
   0.5904E+02      0.2747E-01      0.4654E+01      0.1000E-01
   0.1476E+03      0.6867E-01      0.1164E+02      0.2500E-01
   0.2934E+03      0.1369E+00      0.2327E+02      0.5000E-01
   0.4065E+03      0.1957E+00      0.3491E+02      0.7500E-01
```

```
0.5086E+03      0.2512E+00      0.4654E+02      0.1000E+00
0.8122E+03      0.4949E+00      0.1164E+03      0.2500E+00
0.9751E+03      0.7967E+00      0.1785E+03      0.5000E+00
0.9990E+03      0.9048E+00      0.1979E+03      0.6000E+00
0.1102E+04      0.1841E+01      0.3058E+03      0.1500E+01
0.1137E+04      0.3354E+01      0.3429E+03      0.3000E+01


 RESULT FROM UPPER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON             IN.              TON             IN.
0.8303E-01      0.3453E-04      0.6952E-02      0.1000E-04
0.4152E+00      0.1727E-03      0.3476E-01      0.5000E-04
0.8303E+00      0.3453E-03      0.6952E-01      0.1000E-03
0.4198E+02      0.1739E-01      0.3476E+01      0.5000E-02
0.6297E+02      0.2609E-01      0.5214E+01      0.7500E-02
0.8396E+02      0.3479E-01      0.6952E+01      0.1000E-01
0.2099E+03      0.8696E-01      0.1738E+02      0.2500E-01
0.4130E+03      0.1723E+00      0.3476E+02      0.5000E-01
0.5625E+03      0.2420E+00      0.5214E+02      0.7500E-01
0.6880E+03      0.3048E+00      0.6952E+02      0.1000E+00
0.9816E+03      0.5480E+00      0.1738E+03      0.2500E+00
0.1083E+04      0.8324E+00      0.2463E+03      0.5000E+00
0.1101E+04      0.9390E+00      0.2651E+03      0.6000E+00
0.1172E+04      0.1864E+01      0.3393E+03      0.1500E+01
0.1184E+04      0.3368E+01      0.3517E+03      0.3000E+01


 RESULT FROM LOWER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON             IN.              TON             IN.
0.3588E-01      0.2062E-04      0.2357E-02      0.1000E-04
0.1794E+00      0.1031E-03      0.1178E-01      0.5000E-04
0.3588E+00      0.2062E-03      0.2356E-01      0.1000E-03
0.1803E+02      0.1033E-01      0.1178E+01      0.5000E-02
0.2705E+02      0.1550E-01      0.1767E+01      0.7500E-02
0.3607E+02      0.2067E-01      0.2357E+01      0.1000E-01
0.9017E+02      0.5167E-01      0.5891E+01      0.2500E-01
0.1803E+03      0.1033E+00      0.1178E+02      0.5000E-01
0.2579E+03      0.1514E+00      0.1767E+02      0.7500E-01
0.3317E+03      0.1984E+00      0.2357E+02      0.1000E+00
0.6349E+03      0.4394E+00      0.5891E+02      0.2500E+00
0.8668E+03      0.7608E+00      0.1108E+03      0.5000E+00
0.8961E+03      0.8705E+00      0.1308E+03      0.6000E+00
0.1032E+04      0.1819E+01      0.2722E+03      0.1500E+01
0.1087E+04      0.3338E+01      0.3323E+03      0.3000E+01
```

South Abutment
Static Axial Capacity
5-ft CIDH with 6-ft Casing

```
==========================================================================

                 SHAFT for Windows, Version 2017.8.4

                  Serial Number :  253582343

               VERTICALLY LOADED DRILLED SHAFT ANALYSIS
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                       All Rights Reserved

==========================================================================


     Path to file locations      : C:\Users\JBonfiglio\OneDrive -
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
     Name of input data file     : SouthAbut_Final_Rev.sf8d
     Name of output file         : SouthAbut_Final_Rev.sf8o
     Name of plot output file    : SouthAbut_Final_Rev.sf8p
     Name of runtime file        : SouthAbut_Final_Rev.sf8r


     --------------------------------------------------------------------------
                          Time and Date of Analysis
     --------------------------------------------------------------------------

               Date:  May 18, 2020     Time:  09:27:10

     Camino Del Mar Bridge Replacement - Phase 0


     PROPOSED DEPTH =     150.0 FT
     ----------------


     NUMBER OF LAYERS =    8
     ------------------


     WATER TABLE DEPTH =      10.0 FT.
     -------------------




     SOIL INFORMATION
     ---------------

     LAYER NO 1----SAND
```

```
   AT THE TOP

   SIDE FRICTION PROCEDURE, BETA METHOD
   SKIN FRICTION COEFFICIENT- BETA                   = 0.120E+01
   INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.000E+00

   AT THE BOTTOM

   SIDE FRICTION PROCEDURE, BETA METHOD
   SKIN FRICTION COEFFICIENT- BETA                   = 0.110E+01
   INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.900E+01

   LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
   LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


 LAYER NO 2----SAND

   AT THE TOP

   SIDE FRICTION PROCEDURE, BETA METHOD
   SKIN FRICTION COEFFICIENT- BETA                   = 0.730E+00
   INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.900E+01

   AT THE BOTTOM

   SIDE FRICTION PROCEDURE, BETA METHOD
   SKIN FRICTION COEFFICIENT- BETA                   = 0.510E+00
   INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.300E+02

   LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
```

```
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 3----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.760E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.320E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.300E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.310E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.320E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.780E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 4----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.780E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
```

```
  SOIL UNIT WEIGHT, LB/CU FT                             = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT               = 0.100E+11
  DEPTH, FT                                              = 0.940E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)                 = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)                = 0.500E+00


LAYER NO 5----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                        = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                          = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST          = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                             = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT               = 0.100E+11
  DEPTH, FT                                              = 0.940E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                        = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                          = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST          = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                             = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT               = 0.100E+11
  DEPTH, FT                                              = 0.152E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)                 = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)                = 0.500E+00


LAYER NO 6----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                        = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                          = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST          = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                             = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT               = 0.100E+11
  DEPTH, FT                                              = 0.152E+03

  AT THE BOTTOM
```

```
  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.201E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.201E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.231E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 8----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.510E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
```

```
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.231E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.510E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.266E+03


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00



(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    5.000  FT.
 MAXIMUM SHAFT DIAMETER    =    5.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   94.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN





COMPUTATION RESULTS
-------------------



- CASE ANALYZED      :     1
  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1
```

```
 DRILLED SHAFT INFORMATION

 -------------------------

  DIAMETER OF STEM          =    5.000  FT.
  DIAMETER OF BASE          =    5.000  FT.
  END OF STEM TO BASE       =    0.000  FT.
  ANGLE OF BELL             =    0.000  DEG.
  IGNORED TOP PORTION       =   94.000  FT.
  IGNORED BOTTOM PORTION    =    0.000  FT.
  AREA OF ONE PERCENT STEEL =   28.278  SQ.IN.
  ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
  VOLUME OF UNDERREAM       =    0.000  CU.YDS.
  SHAFT LENGTH              =  150.000  FT.


 PREDICTED RESULTS
 -----------------

 QS      = ULTIMATE SIDE RESISTANCE;
 QB      = ULTIMATE BASE RESISTANCE;
 WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
 QU      = TOTAL ULTIMATE RESISTANCE;
 LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
           TO THE ULTIMATE SIDE RESISTANCE;
 LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
           TO THE ULTIMATE BASE RESISTANCE
 LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

 LENGTH  VOLUME     QS      QB       QU     LRFD QS  LRFD QB   LRFD QU
  (FT)   (CU.YDS) (TONS)   (TONS)   (TONS)   (TONS)   (TONS)    (TONS)
 95.0    69.09    12.23   174.58   186.81     8.56    87.29     95.85
 96.0    69.82    24.59   174.58   199.17    17.21    87.29    104.50
 97.0    70.55    37.07   174.58   211.65    25.95    87.29    113.24
 98.0    71.28    49.67   174.58   224.25    34.77    87.29    122.06
 99.0    72.00    62.40   174.58   236.97    43.68    87.29    130.97
100.0    72.73    75.25   174.58   249.82    52.67    87.29    139.96
101.0    73.46    88.22   174.58   262.79    61.75    87.29    149.04
102.0    74.19   101.31   174.58   275.89    70.92    87.29    158.21
103.0    74.91   114.53   174.58   289.11    80.17    87.29    167.46
104.0    75.64   127.87   174.58   302.45    89.51    87.29    176.80
105.0    76.37   141.33   174.58   315.91    98.93    87.29    186.22
106.0    77.10   154.92   174.58   329.49   108.44    87.29    195.73
107.0    77.82   168.63   174.58   343.20   118.04    87.29    205.33
108.0    78.55   182.46   174.58   357.03   127.72    87.29    215.01
```

```
109.0    79.28   196.41   174.58   370.99   137.49    87.29    224.78
110.0    80.00   210.49   174.58   385.07   147.34    87.29    234.63
111.0    80.73   224.69   174.58   399.27   157.28    87.29    244.57
112.0    81.46   239.01   174.58   413.59   167.31    87.29    254.60
113.0    82.19   253.46   174.58   428.04   177.42    87.29    264.71
114.0    82.91   268.03   174.58   442.61   187.62    87.29    274.91
115.0    83.64   282.72   174.58   457.30   197.90    87.29    285.19
116.0    84.37   297.54   174.58   472.11   208.28    87.29    295.56
117.0    85.10   312.47   174.58   487.05   218.73    87.29    306.02
118.0    85.82   327.54   174.58   502.11   229.27    87.29    316.56
119.0    86.55   342.72   174.58   517.30   239.90    87.29    327.19
120.0    87.28   358.03   174.58   532.60   250.62    87.29    337.91
121.0    88.01   373.46   174.58   548.03   261.42    87.29    348.71
122.0    88.73   389.01   174.58   563.59   272.31    87.29    359.59
123.0    89.46   404.68   174.58   579.26   283.28    87.29    370.57
124.0    90.19   420.48   174.58   595.06   294.34    87.29    381.63
125.0    90.91   436.40   174.58   610.98   305.48    87.29    392.77
126.0    91.64   452.45   174.58   627.03   316.71    87.29    404.00
127.0    92.37   468.62   174.58   643.19   328.03    87.29    415.32
128.0    93.10   484.91   174.58   659.48   339.43    87.29    426.72
129.0    93.82   501.32   174.58   675.90   350.92    87.29    438.21
130.0    94.55   517.86   174.58   692.43   362.50    87.29    449.79
131.0    95.28   534.51   174.58   709.09   374.16    87.29    461.45
132.0    96.01   551.30   174.58   725.87   385.91    87.29    473.20
133.0    96.73   568.20   174.58   742.78   397.74    87.29    485.03
134.0    97.46   585.23   174.58   759.81   409.66    87.29    496.95
135.0    98.19   602.38   174.58   776.96   421.67    87.29    508.95
136.0    98.91   619.65   174.58   794.23   433.76    87.29    521.05
137.0    99.64   637.05   174.58   811.63   445.94    87.29    533.22
138.0   100.37   654.57   174.58   829.15   458.20    87.29    545.49
139.0   101.10   672.21   174.58   846.79   470.55    87.29    557.84
140.0   101.82   689.98   174.58   864.56   482.98    87.29    570.27
141.0   102.55   707.87   174.58   882.44   495.51    87.29    582.80
142.0   103.28   725.88   174.58   900.46   508.11    87.29    595.40
143.0   104.01   744.01   184.68   928.70   520.81    92.34    613.15
144.0   104.73   762.27   195.71   957.98   533.59    97.86    631.44
145.0   105.46   780.65   207.66   988.30   546.45   103.83    650.28
146.0   106.19   799.15   220.52  1019.67   559.41   110.26    669.67
147.0   106.92   817.78   234.30  1052.08   572.45   117.15    689.60
148.0   107.64   836.53   245.33  1081.85   585.57   122.66    708.23
149.0   108.37   855.40   253.60  1109.00   598.78   126.80    725.58
150.0   109.10   874.39   259.11  1133.50   612.08   129.55    741.63




 AXIAL LOAD VS SETTLEMENT CURVES
 -------------------------------
```

```
 RESULT FROM TREND (AVERAGED) LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON            IN.               TON            IN.
0.5809E-01      0.2696E-04      0.3412E-02      0.1000E-04
0.2904E+00      0.1348E-03      0.1706E-01      0.5000E-04
0.5809E+00      0.2696E-03      0.3412E-01      0.1000E-03
0.2927E+02      0.1354E-01      0.1706E+01      0.5000E-02
0.4391E+02      0.2031E-01      0.2559E+01      0.7500E-02
0.5855E+02      0.2708E-01      0.3412E+01      0.1000E-01
0.1464E+03      0.6770E-01      0.8529E+01      0.2500E-01
0.2910E+03      0.1350E+00      0.1706E+02      0.5000E-01
0.4047E+03      0.1935E+00      0.2559E+02      0.7500E-01
0.5084E+03      0.2492E+00      0.3412E+02      0.1000E+00
0.8057E+03      0.4897E+00      0.8529E+02      0.2500E+00
0.9623E+03      0.7888E+00      0.1309E+03      0.5000E+00
0.9820E+03      0.8954E+00      0.1451E+03      0.6000E+00
0.1061E+04      0.1823E+01      0.2241E+03      0.1500E+01
0.1085E+04      0.3332E+01      0.2513E+03      0.3000E+01


 RESULT FROM UPPER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON            IN.               TON            IN.
0.8161E-01      0.3375E-04      0.5096E-02      0.1000E-04
0.4080E+00      0.1687E-03      0.2548E-01      0.5000E-04
0.8161E+00      0.3375E-03      0.5096E-01      0.1000E-03
0.4124E+02      0.1699E-01      0.2548E+01      0.5000E-02
0.6187E+02      0.2549E-01      0.3822E+01      0.7500E-02
0.8249E+02      0.3399E-01      0.5096E+01      0.1000E-01
0.2062E+03      0.8496E-01      0.1274E+02      0.2500E-01
0.4060E+03      0.1684E+00      0.2548E+02      0.5000E-01
0.5563E+03      0.2377E+00      0.3822E+02      0.7500E-01
0.6853E+03      0.3013E+00      0.5096E+02      0.1000E+00
0.9679E+03      0.5398E+00      0.1274E+03      0.2500E+00
0.1053E+04      0.8184E+00      0.1805E+03      0.5000E+00
0.1067E+04      0.9232E+00      0.1943E+03      0.6000E+00
0.1121E+04      0.1842E+01      0.2487E+03      0.1500E+01
0.1130E+04      0.3346E+01      0.2578E+03      0.3000E+01


 RESULT FROM LOWER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON            IN.               TON            IN.
```

```
0.3617E-01      0.2057E-04      0.1727E-02      0.1000E-04
0.1808E+00      0.1029E-03      0.8637E-02      0.5000E-04
0.3617E+00      0.2057E-03      0.1727E-01      0.1000E-03
0.1817E+02      0.1031E-01      0.8637E+00      0.5000E-02
0.2726E+02      0.1547E-01      0.1296E+01      0.7500E-02
0.3635E+02      0.2062E-01      0.1727E+01      0.1000E-01
0.9089E+02      0.5156E-01      0.4318E+01      0.2500E-01
0.1817E+03      0.1031E+00      0.8637E+01      0.5000E-01
0.2598E+03      0.1511E+00      0.1296E+02      0.7500E-01
0.3340E+03      0.1978E+00      0.1727E+02      0.1000E+00
0.6384E+03      0.4382E+00      0.4318E+02      0.2500E+00
0.8711E+03      0.7591E+00      0.8119E+02      0.5000E+00
0.8968E+03      0.8675E+00      0.9587E+02      0.6000E+00
0.1000E+04      0.1804E+01      0.1995E+03      0.1500E+01
0.1039E+04      0.3318E+01      0.2436E+03      0.3000E+01
```

South Abutment
Seismic Axial Capacity
5-ft CIDH with 6-ft Casing

```
==========================================================================

                 SHAFT for Windows, Version 2017.8.4

                  Serial Number :  253582343

               VERTICALLY LOADED DRILLED SHAFT ANALYSIS
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                       All Rights Reserved

==========================================================================


     Path to file locations      : C:\Users\JBonfiglio\OneDrive - 
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
     Name of input data file     : SouthAbut_Seismic_rev2.sf8d
     Name of output file         : SouthAbut_Seismic_rev2.sf8o
     Name of plot output file    : SouthAbut_Seismic_rev2.sf8p
     Name of runtime file        : SouthAbut_Seismic_rev2.sf8r


--------------------------------------------------------------------------
                      Time and Date of Analysis
--------------------------------------------------------------------------

          Date:  May 18, 2020     Time:  12:02:35

     Camino Del Mar Bridge Replacement - Phase 0


     PROPOSED DEPTH =     150.0 FT
     ----------------


     NUMBER OF LAYERS =    8
     ------------------


     WATER TABLE DEPTH =      10.0 FT.
     -------------------




     SOIL INFORMATION
     ---------------

     LAYER NO 1----SAND
```

```
  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.120E+01
  INTERNAL FRICTION ANGLE, DEG.                      = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.000E+00

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.110E+01
  INTERNAL FRICTION ANGLE, DEG.                      = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.900E+01

  LRFD RESISTANCE FACTOR (SIDE FRICTION)             = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)            = 0.100E+01


LAYER NO 2----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.816E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.900E+01


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.300E+02
```

```
  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 3----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.300E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.780E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 4----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.780E+02


  AT THE BOTTOM
```

```
   STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
   END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
   UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.940E+02


   LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
   LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


 LAYER NO 5----SAND

   AT THE TOP

   SIDE FRICTION PROCEDURE, BETA METHOD
   SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
   INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.940E+02

   AT THE BOTTOM

   SIDE FRICTION PROCEDURE, BETA METHOD
   SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
   INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.152E+03

   LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
   LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


 LAYER NO 6----SAND

   AT THE TOP

   SIDE FRICTION PROCEDURE, BETA METHOD
   SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
   INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
```

```
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.152E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.201E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.201E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.231E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 8----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.510E+00
```

```
  END BEARING COEFFICIENT-Nc                          = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                  = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.231E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                     = 0.510E+00
  END BEARING COEFFICIENT-Nc                          = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                  = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.266E+03


  LRFD RESISTANCE FACTOR (SIDE FRICTION)              = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)             = 0.100E+01


(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    5.000  FT.
 MAXIMUM SHAFT DIAMETER    =    5.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   94.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN




COMPUTATION RESULTS
-------------------
```

```
 - CASE ANALYZED      :     1
   VARIATION LENGTH   :     1
   VARIATION DIAMETER :     1



 DRILLED SHAFT INFORMATION

 -------------------------

  DIAMETER OF STEM          =    5.000  FT.
  DIAMETER OF BASE          =    5.000  FT.
  END OF STEM TO BASE       =    0.000  FT.
  ANGLE OF BELL             =    0.000  DEG.
  IGNORED TOP PORTION       =   94.000  FT.
  IGNORED BOTTOM PORTION    =    0.000  FT.
  AREA OF ONE PERCENT STEEL =   28.278  SQ.IN.
  ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
  VOLUME OF UNDERREAM       =    0.000  CU.YDS.
  SHAFT LENGTH              =  150.000  FT.


 PREDICTED RESULTS
 -----------------

 QS      = ULTIMATE SIDE RESISTANCE;
 QB      = ULTIMATE BASE RESISTANCE;
 WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
 QU      = TOTAL ULTIMATE RESISTANCE;
 LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
           TO THE ULTIMATE SIDE RESISTANCE;
 LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
           TO THE ULTIMATE BASE RESISTANCE
 LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

 LENGTH  VOLUME     QS      QB      QU     LRFD QS  LRFD QB   LRFD QU
  (FT)   (CU.YDS) (TONS)   (TONS)  (TONS)   (TONS)   (TONS)    (TONS)
 95.0    69.09    12.23   174.58   186.81    12.23   174.58    186.81
 96.0    69.82    24.59   174.58   199.17    24.59   174.58    199.17
 97.0    70.55    37.07   174.58   211.65    37.07   174.58    211.65
 98.0    71.28    49.67   174.58   224.25    49.67   174.58    224.25
 99.0    72.00    62.40   174.58   236.97    62.40   174.58    236.97
100.0    72.73    75.25   174.58   249.82    75.25   174.58    249.82
101.0    73.46    88.22   174.58   262.79    88.22   174.58    262.79
102.0    74.19   101.31   174.58   275.89   101.31   174.58    275.89
103.0    74.91   114.53   174.58   289.11   114.53   174.58    289.11
104.0    75.64   127.87   174.58   302.45   127.87   174.58    302.45
```

```
 105.0     76.37   141.33   174.58    315.91   141.33   174.58    315.91
 106.0     77.10   154.92   174.58    329.49   154.92   174.58    329.49
 107.0     77.82   168.63   174.58    343.20   168.63   174.58    343.20
 108.0     78.55   182.46   174.58    357.03   182.46   174.58    357.03
 109.0     79.28   196.41   174.58    370.99   196.41   174.58    370.99
 110.0     80.00   210.49   174.58    385.07   210.49   174.58    385.07
 111.0     80.73   224.69   174.58    399.27   224.69   174.58    399.27
 112.0     81.46   239.01   174.58    413.59   239.01   174.58    413.59
 113.0     82.19   253.46   174.58    428.04   253.46   174.58    428.04
 114.0     82.91   268.03   174.58    442.61   268.03   174.58    442.61
 115.0     83.64   282.72   174.58    457.30   282.72   174.58    457.30
 116.0     84.37   297.54   174.58    472.11   297.54   174.58    472.11
 117.0     85.10   312.47   174.58    487.05   312.47   174.58    487.05
 118.0     85.82   327.54   174.58    502.11   327.54   174.58    502.11
 119.0     86.55   342.72   174.58    517.30   342.72   174.58    517.30
 120.0     87.28   358.03   174.58    532.60   358.03   174.58    532.60
 121.0     88.01   373.46   174.58    548.03   373.46   174.58    548.03
 122.0     88.73   389.01   174.58    563.59   389.01   174.58    563.59
 123.0     89.46   404.68   174.58    579.26   404.68   174.58    579.26
 124.0     90.19   420.48   174.58    595.06   420.48   174.58    595.06
 125.0     90.91   436.40   174.58    610.98   436.40   174.58    610.98
 126.0     91.64   452.45   174.58    627.03   452.45   174.58    627.03
 127.0     92.37   468.62   174.58    643.19   468.62   174.58    643.19
 128.0     93.10   484.91   174.58    659.48   484.91   174.58    659.48
 129.0     93.82   501.32   174.58    675.90   501.32   174.58    675.90
 130.0     94.55   517.86   174.58    692.43   517.86   174.58    692.43
 131.0     95.28   534.51   174.58    709.09   534.51   174.58    709.09
 132.0     96.01   551.30   174.58    725.87   551.30   174.58    725.87
 133.0     96.73   568.20   174.58    742.78   568.20   174.58    742.78
 134.0     97.46   585.23   174.58    759.81   585.23   174.58    759.81
 135.0     98.19   602.38   174.58    776.96   602.38   174.58    776.96
 136.0     98.91   619.65   174.58    794.23   619.65   174.58    794.23
 137.0     99.64   637.05   174.58    811.63   637.05   174.58    811.63
 138.0    100.37   654.57   174.58    829.15   654.57   174.58    829.15
 139.0    101.10   672.21   174.58    846.79   672.21   174.58    846.79
 140.0    101.82   689.98   174.58    864.56   689.98   174.58    864.56
 141.0    102.55   707.87   174.58    882.44   707.87   174.58    882.44
 142.0    103.28   725.88   174.58    900.46   725.88   174.58    900.46
 143.0    104.01   744.01   184.68    928.70   744.01   184.68    928.70
 144.0    104.73   762.27   195.71    957.98   762.27   195.71    957.98
 145.0    105.46   780.65   207.66    988.30   780.65   207.66    988.30
 146.0    106.19   799.15   220.52   1019.67   799.15   220.52   1019.67
 147.0    106.92   817.78   234.30   1052.08   817.78   234.30   1052.08
 148.0    107.64   836.53   245.33   1081.85   836.53   245.33   1081.85
 149.0    108.37   855.40   253.60   1109.00   855.40   253.60   1109.00
 150.0    109.10   874.39   259.11   1133.50   874.39   259.11   1133.50
```

```
AXIAL LOAD VS SETTLEMENT CURVES
-------------------------------


   RESULT FROM TREND (AVERAGED) LINE

   TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
      TON            IN.               TON            IN.
  0.5809E-01      0.2696E-04      0.3412E-02      0.1000E-04
  0.2904E+00      0.1348E-03      0.1706E-01      0.5000E-04
  0.5809E+00      0.2696E-03      0.3412E-01      0.1000E-03
  0.2927E+02      0.1354E-01      0.1706E+01      0.5000E-02
  0.4391E+02      0.2031E-01      0.2559E+01      0.7500E-02
  0.5855E+02      0.2708E-01      0.3412E+01      0.1000E-01
  0.1464E+03      0.6770E-01      0.8529E+01      0.2500E-01
  0.2910E+03      0.1350E+00      0.1706E+02      0.5000E-01
  0.4047E+03      0.1935E+00      0.2559E+02      0.7500E-01
  0.5084E+03      0.2492E+00      0.3412E+02      0.1000E+00
  0.8057E+03      0.4897E+00      0.8529E+02      0.2500E+00
  0.9623E+03      0.7888E+00      0.1309E+03      0.5000E+00
  0.9820E+03      0.8954E+00      0.1451E+03      0.6000E+00
  0.1061E+04      0.1823E+01      0.2241E+03      0.1500E+01
  0.1085E+04      0.3332E+01      0.2513E+03      0.3000E+01


   RESULT FROM UPPER-BOUND LINE

   TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
      TON            IN.               TON            IN.
  0.8161E-01      0.3375E-04      0.5096E-02      0.1000E-04
  0.4080E+00      0.1687E-03      0.2548E-01      0.5000E-04
  0.8161E+00      0.3375E-03      0.5096E-01      0.1000E-03
  0.4124E+02      0.1699E-01      0.2548E+01      0.5000E-02
  0.6187E+02      0.2549E-01      0.3822E+01      0.7500E-02
  0.8249E+02      0.3399E-01      0.5096E+01      0.1000E-01
  0.2062E+03      0.8496E-01      0.1274E+02      0.2500E-01
  0.4060E+03      0.1684E+00      0.2548E+02      0.5000E-01
  0.5563E+03      0.2377E+00      0.3822E+02      0.7500E-01
  0.6853E+03      0.3013E+00      0.5096E+02      0.1000E+00
  0.9679E+03      0.5398E+00      0.1274E+03      0.2500E+00
  0.1053E+04      0.8184E+00      0.1805E+03      0.5000E+00
  0.1067E+04      0.9232E+00      0.1943E+03      0.6000E+00
  0.1121E+04      0.1842E+01      0.2487E+03      0.1500E+01
  0.1130E+04      0.3346E+01      0.2578E+03      0.3000E+01
```

```
 RESULT FROM LOWER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT      TIP LOAD      TIP  MOVEMENT
    TON            IN.              TON            IN.
0.3617E-01      0.2057E-04      0.1727E-02      0.1000E-04
0.1808E+00      0.1029E-03      0.8637E-02      0.5000E-04
0.3617E+00      0.2057E-03      0.1727E-01      0.1000E-03
0.1817E+02      0.1031E-01      0.8637E+00      0.5000E-02
0.2726E+02      0.1547E-01      0.1296E+01      0.7500E-02
0.3635E+02      0.2062E-01      0.1727E+01      0.1000E-01
0.9089E+02      0.5156E-01      0.4318E+01      0.2500E-01
0.1817E+03      0.1031E+00      0.8637E+01      0.5000E-01
0.2598E+03      0.1511E+00      0.1296E+02      0.7500E-01
0.3340E+03      0.1978E+00      0.1727E+02      0.1000E+00
0.6384E+03      0.4382E+00      0.4318E+02      0.2500E+00
0.8711E+03      0.7591E+00      0.8119E+02      0.5000E+00
0.8968E+03      0.8675E+00      0.9587E+02      0.6000E+00
0.1000E+04      0.1804E+01      0.1995E+03      0.1500E+01
0.1039E+04      0.3318E+01      0.2436E+03      0.3000E+01
```

Piers
Static Axial Capacity
6-ft CIDH with 7-ft Casing

```
     =========================================================================

                     SHAFT for Windows, Version 2017.8.4

                      Serial Number :  253582343

                   VERTICALLY LOADED DRILLED SHAFT ANALYSIS
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                           All Rights Reserved

     =========================================================================


     Path to file locations      : C:\Users\JBonfiglio\OneDrive -
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
     Name of input data file     : Piers_6ft_Final_Rev.sf8d
     Name of output file         : Piers_6ft_Final_Rev.sf8o
     Name of plot output file    : Piers_6ft_Final_Rev.sf8p
     Name of runtime file        : Piers_6ft_Final_Rev.sf8r


     -------------------------------------------------------------------------
                           Time and Date of Analysis
     -------------------------------------------------------------------------

               Date:  May 18, 2020     Time:  11:27:17

     Camino Del Mar Bridge Replacement - Phase 0


     TOTAL LOAD =    1100.0 TONS
     ------------


     NUMBER OF LAYERS =    8
     ------------------


     WATER TABLE DEPTH =       0.0 FT.
     -------------------




     SOIL INFORMATION
     ---------------

     LAYER NO 1----SAND
```

```
  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.100E+01
  INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.000E+00

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.690E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.120E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.680E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 2----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.840E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.120E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.190E+02
```

```
  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.680E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 3----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.610E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.190E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.470E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.350E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.680E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 4----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.700E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.320E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.350E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.410E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.320E+02
```

```
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.650E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.680E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 5----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.650E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.820E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.680E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 6----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.280E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.820E+02
```

```
  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                       = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                         = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST         = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                            = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT              = 0.100E+11
  DEPTH, FT                                             = 0.122E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)                = 0.680E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)               = 0.500E+00


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                       = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                         = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST         = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                            = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT              = 0.100E+11
  DEPTH, FT                                             = 0.122E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                       = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                         = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST         = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                            = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT              = 0.100E+11
  DEPTH, FT                                             = 0.207E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)                = 0.680E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)               = 0.500E+00


LAYER NO 8----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                       = 0.510E+00
  END BEARING COEFFICIENT-Nc                            = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                    = 0.400E+04
```

```
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.207E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.510E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.255E+03


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.680E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    6.000  FT.
 MAXIMUM SHAFT DIAMETER    =    6.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN




COMPUTATION RESULTS
-------------------



- CASE ANALYZED      :     1
```

```
  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1



DRILLED SHAFT INFORMATION

-------------------------

 DIAMETER OF STEM          =    6.000  FT.
 DIAMETER OF BASE          =    6.000  FT.
 END OF STEM TO BASE       =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 AREA OF ONE PERCENT STEEL =   40.720  SQ.IN.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
 VOLUME OF UNDERREAM       =    0.000  CU.YDS.


PREDICTED RESULTS
-----------------

QS      = ULTIMATE SIDE RESISTANCE;
QB      = ULTIMATE BASE RESISTANCE;
WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
QU      = TOTAL ULTIMATE RESISTANCE;
LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE SIDE RESISTANCE;
LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE BASE RESISTANCE
LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

LENGTH  VOLUME     QS      QB      QU     LRFD QS  LRFD QB   LRFD QU
 (FT)   (CU.YDS) (TONS)   (TONS)  (TONS)   (TONS)   (TONS)    (TONS)
83.0    86.93    12.50   209.49   221.99     8.50   104.75    113.25
84.0    87.98    25.13   209.49   234.63    17.09   104.75    121.84
85.0    89.02    37.90   209.49   247.39    25.77   104.75    130.52
86.0    90.07    50.79   209.49   260.28    34.54   104.75    139.28
87.0    91.12    63.81   209.49   273.30    43.39   104.75    148.14
88.0    92.17    76.95   209.49   286.45    52.33   104.75    157.08
89.0    93.21    90.23   209.49   299.72    61.35   104.75    166.10
90.0    94.26   103.63   209.49   313.12    70.47   104.75    175.21
91.0    95.31   117.15   209.49   326.64    79.66   104.75    184.41
92.0    96.35   130.79   209.49   340.29    88.94   104.75    193.69
93.0    97.40   144.56   209.49   354.06    98.30   104.75    203.05
94.0    98.45   158.46   209.49   367.95   107.75   104.75    212.50
95.0    99.50   172.47   209.49   381.96   117.28   104.75    222.02
```

```
  96.0   100.54   186.60   209.49   396.09   126.89   104.75    231.63
  97.0   101.59   200.85   209.49   410.35   136.58   104.75    241.33
  98.0   102.64   215.22   209.49   424.72   146.35   104.75    251.10
  99.0   103.69   229.71   209.49   439.20   156.20   104.75    260.95
 100.0   104.73   244.31   209.49   453.81   166.13   104.75    270.88
 101.0   105.78   259.03   209.49   468.53   176.14   104.75    280.89
 102.0   106.83   273.87   209.49   483.36   186.23   104.75    290.98
 103.0   107.88   288.82   209.49   498.31   196.40   104.75    301.14
 104.0   108.92   303.88   209.49   513.37   206.64   104.75    311.38
 105.0   109.97   319.05   209.49   528.55   216.96   104.75    321.70
 106.0   111.02   334.34   209.49   543.83   227.35   104.75    332.10
 107.0   112.06   349.73   209.49   559.23   237.82   104.75    342.57
 108.0   113.11   365.24   209.49   574.73   248.36   104.75    353.11
 109.0   114.16   380.85   209.49   590.35   258.98   104.75    363.73
 110.0   115.21   396.57   209.49   606.07   269.67   104.75    374.42
 111.0   116.25   412.40   219.36   631.76   280.43   109.68    390.11
 112.0   117.30   428.34   229.99   658.32   291.27   114.99    406.26
 113.0   118.35   444.38   241.37   685.75   302.18   120.69    422.86
 114.0   119.40   460.52   253.52   714.04   313.15   126.76    439.91
 115.0   120.44   476.77   266.42   743.19   324.20   133.21    457.41
 116.0   121.49   493.12   280.08   773.20   335.32   140.04    475.36
 117.0   122.54   509.57   291.47   801.04   346.51   145.73    492.24
 118.0   123.59   526.12   300.58   826.70   357.76   150.29    508.05
 119.0   124.63   542.77   307.41   850.18   369.08   153.70    522.79
 120.0   125.68   559.52   311.96   871.48   380.47   155.98    536.45
 121.0   126.73   576.37   314.24   890.61   391.93   157.12    549.05
 122.0   127.77   593.31   314.24   907.55   403.45   157.12    560.57
 123.0   128.82   610.41   314.24   924.65   415.08   157.12    572.20
 124.0   129.87   627.67   314.24   941.91   426.82   157.12    583.94
 125.0   130.92   645.09   314.24   959.33   438.66   157.12    595.78
 126.0   131.96   662.66   314.24   976.90   450.61   157.12    607.73
 127.0   133.01   680.40   314.24   994.64   462.67   157.12    619.79
 128.0   134.06   698.30   314.24  1012.54   474.84   157.12    631.96
 129.0   135.11   716.35   314.24  1030.59   487.12   157.12    644.24
 130.0   136.15   734.56   314.24  1048.80   499.50   157.12    656.62
 131.0   137.20   752.94   314.24  1067.18   512.00   157.12    669.12
 132.0   138.25   771.47   314.24  1085.71   524.60   157.12    681.72
 133.0   139.30   790.16   314.24  1104.40   537.31   157.12    694.43
 134.0   140.34   809.01   314.24  1123.25   550.13   157.12    707.25
 135.0   141.39   828.02   314.24  1142.26   563.06   157.12    720.18
 136.0   142.44   847.19   314.24  1161.43   576.09   157.12    733.21
 137.0   143.48   866.52   314.24  1180.76   589.23   157.12    746.35
 138.0   144.53   886.01   314.24  1200.25   602.49   157.12    759.61
 139.0   145.58   905.66   314.24  1219.90   615.85   157.12    772.97
 140.0   146.63   925.46   314.24  1239.70   629.32   157.12    786.44
 141.0   147.67   945.43   314.24  1259.67   642.89   157.12    800.01
 142.0   148.72   965.56   314.24  1279.80   656.58   157.12    813.70
 143.0   149.77   985.84   314.24  1300.08   670.37   157.12    827.49
```

```
144.0   150.82  1006.29   314.24  1320.52   684.27   157.12    841.39
145.0   151.86  1026.89   314.24  1341.13   698.28   157.12    855.40
146.0   152.91  1047.65   314.24  1361.89   712.40   157.12    869.52
147.0   153.96  1068.57   314.24  1382.81   726.63   157.12    883.75
148.0   155.01  1089.65   314.24  1403.89   740.97   157.12    898.08
149.0   156.05  1110.90   314.24  1425.13   755.41   157.12    912.53
150.0   157.10  1132.29   314.24  1446.53   769.96   157.12    927.08
151.0   158.15  1153.85   314.24  1468.09   784.62   157.12    941.74
152.0   159.19  1175.57   314.24  1489.81   799.39   157.12    956.51
153.0   160.24  1197.45   314.24  1511.69   814.27   157.12    971.39
154.0   161.29  1219.49   314.24  1533.73   829.25   157.12    986.37
155.0   162.34  1241.68   314.24  1555.92   844.34   157.12   1001.46
156.0   163.38  1264.04   314.24  1578.28   859.55   157.12   1016.67
157.0   164.43  1286.55   314.24  1600.79   874.86   157.12   1031.98
158.0   165.48  1309.23   314.24  1623.47   890.28   157.12   1047.40
159.0   166.53  1332.06   314.24  1646.30   905.80   157.12   1062.92
160.0   167.57  1355.05   314.24  1669.29   921.44   157.12   1078.56
161.0   168.62  1378.21   314.24  1692.45   937.18   157.12   1094.30
162.0   169.67  1401.52   314.24  1715.76   953.03   157.12   1110.15



 AXIAL LOAD VS SETTLEMENT CURVES
 -------------------------------


    RESULT FROM TREND (AVERAGED) LINE

    TOP   LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
       TON             IN.               TON             IN.
   0.8041E-01      0.2628E-04      0.3448E-02      0.1000E-04
   0.4021E+00      0.1314E-03      0.1724E-01      0.5000E-04
   0.8041E+00      0.2628E-03      0.3448E-01      0.1000E-03
   0.4043E+02      0.1318E-01      0.1724E+01      0.5000E-02
   0.6065E+02      0.1977E-01      0.2586E+01      0.7500E-02
   0.8086E+02      0.2637E-01      0.3448E+01      0.1000E-01
   0.2022E+03      0.6592E-01      0.8620E+01      0.2500E-01
   0.4034E+03      0.1317E+00      0.1724E+02      0.5000E-01
   0.5743E+03      0.1922E+00      0.2586E+02      0.7500E-01
   0.7171E+03      0.2466E+00      0.3448E+02      0.1000E+00
   0.1185E+04      0.4971E+00      0.8620E+02      0.2500E+00
   0.1462E+04      0.8086E+00      0.1443E+03      0.5000E+00
   0.1523E+04      0.1043E+01      0.1760E+03      0.7200E+00
   0.1616E+04      0.2148E+01      0.2718E+03      0.1800E+01
   0.1644E+04      0.3956E+01      0.3048E+03      0.3600E+01
```

```
 RESULT FROM UPPER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON            IN.               TON             IN.
0.1145E+00      0.3301E-04      0.5150E-02      0.1000E-04
0.5726E+00      0.1651E-03      0.2575E-01      0.5000E-04
0.1145E+01      0.3301E-03      0.5150E-01      0.1000E-03
0.5769E+02      0.1658E-01      0.2575E+01      0.5000E-02
0.8655E+02      0.2488E-01      0.3863E+01      0.7500E-02
0.1154E+03      0.3317E-01      0.5150E+01      0.1000E-01
0.2885E+03      0.8293E-01      0.1288E+02      0.2500E-01
0.5701E+03      0.1649E+00      0.2575E+02      0.5000E-01
0.7972E+03      0.2372E+00      0.3863E+02      0.7500E-01
0.9784E+03      0.2999E+00      0.5150E+02      0.1000E+00
0.1448E+04      0.5542E+00      0.1288E+03      0.2500E+00
0.1610E+04      0.8429E+00      0.2050E+03      0.5000E+00
0.1640E+04      0.1071E+01      0.2357E+03      0.7200E+00
0.1704E+04      0.2168E+01      0.3017E+03      0.1800E+01
0.1715E+04      0.3971E+01      0.3127E+03      0.3600E+01


 RESULT FROM LOWER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON            IN.               TON             IN.
0.4939E-01      0.2007E-04      0.1746E-02      0.1000E-04
0.2470E+00      0.1004E-03      0.8729E-02      0.5000E-04
0.4939E+00      0.2007E-03      0.1746E-01      0.1000E-03
0.2478E+02      0.1005E-01      0.8729E+00      0.5000E-02
0.3718E+02      0.1508E-01      0.1309E+01      0.7500E-02
0.4957E+02      0.2011E-01      0.1746E+01      0.1000E-01
0.1239E+03      0.5027E-01      0.4364E+01      0.2500E-01
0.2479E+03      0.1005E+00      0.8729E+01      0.5000E-01
0.3627E+03      0.1492E+00      0.1309E+02      0.7500E-01
0.4632E+03      0.1948E+00      0.1746E+02      0.1000E+00
0.9120E+03      0.4384E+00      0.4364E+02      0.2500E+00
0.1312E+04      0.7737E+00      0.8362E+02      0.5000E+00
0.1405E+04      0.1015E+01      0.1163E+03      0.7200E+00
0.1527E+04      0.2128E+01      0.2420E+03      0.1800E+01
0.1572E+04      0.3941E+01      0.2954E+03      0.3600E+01
```

Piers
Seismic Axial Capacity
6-ft CIDH with 7-ft Casing

```
     =========================================================================

                      SHAFT for Windows, Version 2017.8.4

                       Serial Number :  253582343

                    VERTICALLY LOADED DRILLED SHAFT ANALYSIS
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                           All Rights Reserved

     =========================================================================


     Path to file locations      : C:\Users\JBonfiglio\OneDrive -
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
     Name of input data file     : Piers_6ft_Seismic_Final_Rev.sf8d
     Name of output file         : Piers_6ft_Seismic_Final_Rev.sf8o
     Name of plot output file    : Piers_6ft_Seismic_Final_Rev.sf8p
     Name of runtime file        : Piers_6ft_Seismic_Final_Rev.sf8r


     -------------------------------------------------------------------------
                          Time and Date of Analysis
     -------------------------------------------------------------------------

               Date:  May 18, 2020     Time:  11:23:03

     Camino Del Mar Bridge Replacement - Phase 0


     PROPOSED DEPTH =     180.0 FT
     ----------------


     NUMBER OF LAYERS =    8
     ------------------


     WATER TABLE DEPTH =       0.0 FT.
     -------------------




     SOIL INFORMATION
     ---------------

     LAYER NO 1----CLAY
```

```
   AT THE TOP

   STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
   END BEARING COEFFICIENT-Nc                        = 0.600E+01  (*)
   UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.000E+00


   AT THE BOTTOM

   STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
   END BEARING COEFFICIENT-Nc                        = 0.840E+01  (*)
   UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.120E+02


   LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
   LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


 LAYER NO 2----CLAY

   AT THE TOP

   STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
   END BEARING COEFFICIENT-Nc                        = 0.840E+01  (*)
   UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
   DEPTH, FT                                         = 0.120E+02


   AT THE BOTTOM

   STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
   END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
   UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
   BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
   SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
   MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
```

```
  DEPTH, FT                                         = 0.190E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 3----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.190E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.350E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 4----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.350E+02
```

```
  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                       = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                            = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                    = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST         = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                            = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT              = 0.100E+11
  DEPTH, FT                                             = 0.650E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)                = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)               = 0.100E+01


LAYER NO 5----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                       = 0.550E+00
  END BEARING COEFFICIENT-Nc                            = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                    = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST         = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                            = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT              = 0.100E+11
  DEPTH, FT                                             = 0.650E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                       = 0.550E+00
  END BEARING COEFFICIENT-Nc                            = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                    = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST         = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                            = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT              = 0.100E+11
  DEPTH, FT                                             = 0.820E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)                = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)               = 0.100E+01


LAYER NO 6----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
```

```
  SKIN FRICTION COEFFICIENT- BETA                        = 0.280E+00
  INTERNAL FRICTION ANGLE, DEG.                          = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST          = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                             = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT               = 0.100E+11
  DEPTH, FT                                              = 0.820E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                        = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                          = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST          = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                             = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT               = 0.100E+11
  DEPTH, FT                                              = 0.122E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)                 = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)                = 0.100E+01


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                        = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                          = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST          = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                             = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT               = 0.100E+11
  DEPTH, FT                                              = 0.122E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                        = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                          = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST          = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                             = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT               = 0.100E+11
  DEPTH, FT                                              = 0.207E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)                 = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)                = 0.100E+01


LAYER NO 8----CLAY
```

```
  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                       = 0.510E+00
  END BEARING COEFFICIENT-Nc                            = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                    = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST         = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                            = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT              = 0.100E+11
  DEPTH, FT                                             = 0.207E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                       = 0.510E+00
  END BEARING COEFFICIENT-Nc                            = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                    = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST         = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                            = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT              = 0.100E+11
  DEPTH, FT                                             = 0.255E+03


  LRFD RESISTANCE FACTOR (SIDE FRICTION)                = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)               = 0.100E+01


(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    6.000  FT.
 MAXIMUM SHAFT DIAMETER    =    6.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
```

```
COMPUTATION RESULTS
-------------------



- CASE ANALYZED      :     1
  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1



DRILLED SHAFT INFORMATION

-------------------------

 DIAMETER OF STEM          =    6.000  FT.
 DIAMETER OF BASE          =    6.000  FT.
 END OF STEM TO BASE       =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 AREA OF ONE PERCENT STEEL =   40.720  SQ.IN.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
 VOLUME OF UNDERREAM       =    0.000  CU.YDS.
 SHAFT LENGTH              =  180.000  FT.


PREDICTED RESULTS
-----------------

QS      = ULTIMATE SIDE RESISTANCE;
QB      = ULTIMATE BASE RESISTANCE;
WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
QU      = TOTAL ULTIMATE RESISTANCE;
LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE SIDE RESISTANCE;
LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE BASE RESISTANCE
LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

LENGTH  VOLUME     QS      QB      QU     LRFD QS  LRFD QB   LRFD QU
 (FT)   (CU.YDS) (TONS)   (TONS)  (TONS)   (TONS)   (TONS)    (TONS)
83.0    86.93    12.50   209.49   221.99    12.50   209.49    221.99
84.0    87.98    25.13   209.49   234.63    25.13   209.49    234.63
85.0    89.02    37.90   209.49   247.39    37.90   209.49    247.39
86.0    90.07    50.79   209.49   260.28    50.79   209.49    260.28
87.0    91.12    63.81   209.49   273.30    63.81   209.49    273.30
88.0    92.17    76.95   209.49   286.45    76.95   209.49    286.45
```

```
 89.0    93.21    90.23   209.49   299.72    90.23   209.49    299.72
 90.0    94.26   103.63   209.49   313.12   103.63   209.49    313.12
 91.0    95.31   117.15   209.49   326.64   117.15   209.49    326.64
 92.0    96.35   130.79   209.49   340.29   130.79   209.49    340.29
 93.0    97.40   144.56   209.49   354.06   144.56   209.49    354.06
 94.0    98.45   158.46   209.49   367.95   158.46   209.49    367.95
 95.0    99.50   172.47   209.49   381.96   172.47   209.49    381.96
 96.0   100.54   186.60   209.49   396.09   186.60   209.49    396.09
 97.0   101.59   200.85   209.49   410.35   200.85   209.49    410.35
 98.0   102.64   215.22   209.49   424.72   215.22   209.49    424.72
 99.0   103.69   229.71   209.49   439.20   229.71   209.49    439.20
100.0   104.73   244.31   209.49   453.81   244.31   209.49    453.81
101.0   105.78   259.03   209.49   468.53   259.03   209.49    468.53
102.0   106.83   273.87   209.49   483.36   273.87   209.49    483.36
103.0   107.88   288.82   209.49   498.31   288.82   209.49    498.31
104.0   108.92   303.88   209.49   513.37   303.88   209.49    513.37
105.0   109.97   319.05   209.49   528.55   319.05   209.49    528.55
106.0   111.02   334.34   209.49   543.83   334.34   209.49    543.83
107.0   112.06   349.73   209.49   559.23   349.73   209.49    559.23
108.0   113.11   365.24   209.49   574.73   365.24   209.49    574.73
109.0   114.16   380.85   209.49   590.35   380.85   209.49    590.35
110.0   115.21   396.57   209.49   606.07   396.57   209.49    606.07
111.0   116.25   412.40   219.36   631.76   412.40   219.36    631.76
112.0   117.30   428.34   229.99   658.32   428.34   229.99    658.32
113.0   118.35   444.38   241.37   685.75   444.38   241.37    685.75
114.0   119.40   460.52   253.52   714.04   460.52   253.52    714.04
115.0   120.44   476.77   266.42   743.19   476.77   266.42    743.19
116.0   121.49   493.12   280.08   773.20   493.12   280.08    773.20
117.0   122.54   509.57   291.47   801.04   509.57   291.47    801.04
118.0   123.59   526.12   300.58   826.70   526.12   300.58    826.70
119.0   124.63   542.77   307.41   850.18   542.77   307.41    850.18
120.0   125.68   559.52   311.96   871.48   559.52   311.96    871.48
121.0   126.73   576.37   314.24   890.61   576.37   314.24    890.61
122.0   127.77   593.31   314.24   907.55   593.31   314.24    907.55
123.0   128.82   610.41   314.24   924.65   610.41   314.24    924.65
124.0   129.87   627.67   314.24   941.91   627.67   314.24    941.91
125.0   130.92   645.09   314.24   959.33   645.09   314.24    959.33
126.0   131.96   662.66   314.24   976.90   662.66   314.24    976.90
127.0   133.01   680.40   314.24   994.64   680.40   314.24    994.64
128.0   134.06   698.30   314.24  1012.54   698.30   314.24   1012.54
129.0   135.11   716.35   314.24  1030.59   716.35   314.24   1030.59
130.0   136.15   734.56   314.24  1048.80   734.56   314.24   1048.80
131.0   137.20   752.94   314.24  1067.18   752.94   314.24   1067.18
132.0   138.25   771.47   314.24  1085.71   771.47   314.24   1085.71
133.0   139.30   790.16   314.24  1104.40   790.16   314.24   1104.40
134.0   140.34   809.01   314.24  1123.25   809.01   314.24   1123.25
135.0   141.39   828.02   314.24  1142.26   828.02   314.24   1142.26
136.0   142.44   847.19   314.24  1161.43   847.19   314.24   1161.43
```

```
137.0   143.48   866.52   314.24  1180.76   866.52   314.24   1180.76
138.0   144.53   886.01   314.24  1200.25   886.01   314.24   1200.25
139.0   145.58   905.66   314.24  1219.90   905.66   314.24   1219.90
140.0   146.63   925.46   314.24  1239.70   925.46   314.24   1239.70
141.0   147.67   945.43   314.24  1259.67   945.43   314.24   1259.67
142.0   148.72   965.56   314.24  1279.80   965.56   314.24   1279.80
143.0   149.77   985.84   314.24  1300.08   985.84   314.24   1300.08
144.0   150.82  1006.29   314.24  1320.52  1006.29   314.24   1320.52
145.0   151.86  1026.89   314.24  1341.13  1026.89   314.24   1341.13
146.0   152.91  1047.65   314.24  1361.89  1047.65   314.24   1361.89
147.0   153.96  1068.57   314.24  1382.81  1068.57   314.24   1382.81
148.0   155.01  1089.65   314.24  1403.89  1089.65   314.24   1403.89
149.0   156.05  1110.90   314.24  1425.13  1110.90   314.24   1425.13
150.0   157.10  1132.29   314.24  1446.53  1132.29   314.24   1446.53
151.0   158.15  1153.85   314.24  1468.09  1153.85   314.24   1468.09
152.0   159.19  1175.57   314.24  1489.81  1175.57   314.24   1489.81
153.0   160.24  1197.45   314.24  1511.69  1197.45   314.24   1511.69
154.0   161.29  1219.49   314.24  1533.73  1219.49   314.24   1533.73
155.0   162.34  1241.68   314.24  1555.92  1241.68   314.24   1555.92
156.0   163.38  1264.04   314.24  1578.28  1264.04   314.24   1578.28
157.0   164.43  1286.55   314.24  1600.79  1286.55   314.24   1600.79
158.0   165.48  1309.23   314.24  1623.47  1309.23   314.24   1623.47
159.0   166.53  1332.06   314.24  1646.30  1332.06   314.24   1646.30
160.0   167.57  1355.05   314.24  1669.29  1355.05   314.24   1669.29
161.0   168.62  1378.21   314.24  1692.45  1378.21   314.24   1692.45
162.0   169.67  1401.52   314.24  1715.76  1401.52   314.24   1715.76
163.0   170.72  1424.99   314.24  1739.23  1424.99   314.24   1739.23
164.0   171.76  1448.62   314.24  1762.86  1448.62   314.24   1762.86
165.0   172.81  1472.41   314.24  1786.65  1472.41   314.24   1786.65
166.0   173.86  1496.36   314.24  1810.60  1496.36   314.24   1810.60
167.0   174.90  1520.46   314.24  1834.70  1520.46   314.24   1834.70
168.0   175.95  1544.73   314.24  1858.97  1544.73   314.24   1858.97
169.0   177.00  1569.16   314.24  1883.40  1569.16   314.24   1883.40
170.0   178.05  1593.74   314.24  1907.98  1593.74   314.24   1907.98
171.0   179.09  1618.49   314.24  1932.73  1618.49   314.24   1932.73
172.0   180.14  1643.39   314.24  1957.63  1643.39   314.24   1957.63
173.0   181.19  1668.46   314.24  1982.70  1668.46   314.24   1982.70
174.0   182.24  1693.68   314.24  2007.92  1693.68   314.24   2007.92
175.0   183.28  1719.06   314.24  2033.30  1719.06   314.24   2033.30
176.0   184.33  1744.60   314.24  2058.84  1744.60   314.24   2058.84
177.0   185.38  1770.31   314.24  2084.54  1770.31   314.24   2084.54
178.0   186.43  1796.17   314.24  2110.41  1796.17   314.24   2110.41
179.0   187.47  1822.19   314.24  2136.42  1822.19   314.24   2136.42
180.0   188.52  1848.36   314.24  2162.60  1848.36   314.24   2162.60
```

```
AXIAL LOAD VS SETTLEMENT CURVES
-------------------------------


   RESULT FROM TREND (AVERAGED) LINE

   TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
      TON             IN.              TON             IN.
  0.1101E+00      0.3400E-04      0.3448E-02      0.1000E-04
  0.5506E+00      0.1700E-03      0.1724E-01      0.5000E-04
  0.1101E+01      0.3400E-03      0.3448E-01      0.1000E-03
  0.5541E+02      0.1707E-01      0.1724E+01      0.5000E-02
  0.8313E+02      0.2561E-01      0.2586E+01      0.7500E-02
  0.1108E+03      0.3414E-01      0.3448E+01      0.1000E-01
  0.2771E+03      0.8536E-01      0.8620E+01      0.2500E-01
  0.5482E+03      0.1698E+00      0.1724E+02      0.5000E-01
  0.7760E+03      0.2458E+00      0.2586E+02      0.7500E-01
  0.9603E+03      0.3122E+00      0.3448E+02      0.1000E+00
  0.1537E+04      0.5977E+00      0.8620E+02      0.2500E+00
  0.1864E+04      0.9274E+00      0.1443E+03      0.5000E+00
  0.1931E+04      0.1165E+01      0.1760E+03      0.7200E+00
  0.2024E+04      0.2273E+01      0.2718E+03      0.1800E+01
  0.2052E+04      0.4081E+01      0.3048E+03      0.3600E+01


   RESULT FROM UPPER-BOUND LINE

   TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
      TON             IN.              TON             IN.
  0.1598E+00      0.4444E-04      0.5150E-02      0.1000E-04
  0.7989E+00      0.2222E-03      0.2575E-01      0.5000E-04
  0.1598E+01      0.4444E-03      0.5150E-01      0.1000E-03
  0.8059E+02      0.2235E-01      0.2575E+01      0.5000E-02
  0.1209E+03      0.3353E-01      0.3863E+01      0.7500E-02
  0.1612E+03      0.4471E-01      0.5150E+01      0.1000E-01
  0.4030E+03      0.1118E+00      0.1288E+02      0.2500E-01
  0.7837E+03      0.2200E+00      0.2575E+02      0.5000E-01
  0.1088E+04      0.3132E+00      0.3863E+02      0.7500E-01
  0.1312E+04      0.3895E+00      0.5150E+02      0.1000E+00
  0.1860E+04      0.6740E+00      0.1288E+03      0.2500E+00
  0.2033E+04      0.9702E+00      0.2050E+03      0.5000E+00
  0.2063E+04      0.1199E+01      0.2357E+03      0.7200E+00
  0.2129E+04      0.2299E+01      0.3017E+03      0.1800E+01
  0.2140E+04      0.4102E+01      0.3127E+03      0.3600E+01


   RESULT FROM LOWER-BOUND LINE
```

```
  TOP  LOAD     TOP MOVEMENT      TIP LOAD      TIP  MOVEMENT
     TON             IN.             TON             IN.
 0.6650E-01      0.2465E-04      0.1746E-02      0.1000E-04
 0.3325E+00      0.1233E-03      0.8729E-02      0.5000E-04
 0.6650E+00      0.2465E-03      0.1746E-01      0.1000E-03
 0.3338E+02      0.1235E-01      0.8729E+00      0.5000E-02
 0.5008E+02      0.1853E-01      0.1309E+01      0.7500E-02
 0.6678E+02      0.2471E-01      0.1746E+01      0.1000E-01
 0.1670E+03      0.6177E-01      0.4364E+01      0.2500E-01
 0.3336E+03      0.1235E+00      0.8729E+01      0.5000E-01
 0.4845E+03      0.1822E+00      0.1309E+02      0.7500E-01
 0.6183E+03      0.2369E+00      0.1746E+02      0.1000E+00
 0.1197E+04      0.5181E+00      0.4364E+02      0.2500E+00
 0.1691E+04      0.8837E+00      0.8362E+02      0.5000E+00
 0.1798E+04      0.1131E+01      0.1163E+03      0.7200E+00
 0.1919E+04      0.2247E+01      0.2420E+03      0.1800E+01
 0.1962E+04      0.4061E+01      0.2954E+03      0.3600E+01
```

Piers
Static Axial Capacity
9-ft CIDH with 10-ft Casing

```
     =========================================================================

                      SHAFT for Windows, Version 2017.8.4

                       Serial Number :  253582343

                    VERTICALLY LOADED DRILLED SHAFT ANALYSIS
                     (c) Copyright ENSOFT, Inc., 1987-2017
                           All Rights Reserved

     =========================================================================


     Path to file locations      : C:\Users\JBonfiglio\OneDrive -
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
     Name of input data file     : Piers_9ft_Final_Rev.sf8d
     Name of output file         : Piers_9ft_Final_Rev.sf8o
     Name of plot output file    : Piers_9ft_Final_Rev.sf8p
     Name of runtime file        : Piers_9ft_Final_Rev.sf8r


     -------------------------------------------------------------------------
                          Time and Date of Analysis
     -------------------------------------------------------------------------

               Date:  May 18, 2020     Time:  09:48:22

     Camino Del Mar Bridge Replacement - Phase 0


     PROPOSED DEPTH =     200.0 FT
     ----------------


     NUMBER OF LAYERS =    8
     ------------------


     WATER TABLE DEPTH =       0.0 FT.
     -------------------




     SOIL INFORMATION
     ---------------

     LAYER NO 1----SAND
```

```
  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.100E+01
  INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.000E+00

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.690E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.120E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 2----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.760E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.120E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.853E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.190E+02
```

```
  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 3----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.610E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.190E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.470E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.350E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 4----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.700E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.320E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.350E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.410E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.320E+02
```

```
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.650E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 5----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.650E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.820E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 6----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.280E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.820E+02
```

```
  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.122E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.122E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.207E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 8----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.510E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+04
```

```
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.207E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.510E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.255E+03


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00



(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    9.000  FT.
 MAXIMUM SHAFT DIAMETER    =    9.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN





COMPUTATION RESULTS
-------------------



- CASE ANALYZED      :     1
```

```
  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1



DRILLED SHAFT INFORMATION

-------------------------

 DIAMETER OF STEM          =    9.000  FT.
 DIAMETER OF BASE          =    9.000  FT.
 END OF STEM TO BASE       =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 AREA OF ONE PERCENT STEEL =   91.621  SQ.IN.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
 VOLUME OF UNDERREAM       =    0.000  CU.YDS.
 SHAFT LENGTH              =  200.000  FT.


PREDICTED RESULTS
-----------------

QS      = ULTIMATE SIDE RESISTANCE;
QB      = ULTIMATE BASE RESISTANCE;
WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
QU      = TOTAL ULTIMATE RESISTANCE;
LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE SIDE RESISTANCE;
LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE BASE RESISTANCE
LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

LENGTH  VOLUME     QS      QB      QU     LRFD QS  LRFD QB   LRFD QU
 (FT)   (CU.YDS) (TONS)   (TONS)  (TONS)   (TONS)   (TONS)    (TONS)
83.0   195.59    18.75   313.08   331.83    13.13   156.54    169.66
84.0   197.95    37.70   313.57   351.28    26.39   156.79    183.18
85.0   200.30    56.84   313.91   370.75    39.79   156.95    196.74
86.0   202.66    76.18   314.11   390.29    53.33   157.05    210.38
87.0   205.02    95.71   314.21   409.92    67.00   157.10    224.10
88.0   207.37   115.43   314.24   429.67    80.80   157.12    237.92
89.0   209.73   135.34   314.24   449.58    94.74   157.12    251.86
90.0   212.08   155.44   314.24   469.68   108.81   157.12    265.93
91.0   214.44   175.72   314.24   489.96   123.01   157.12    280.13
92.0   216.80   196.19   314.24   510.43   137.33   157.12    294.45
93.0   219.15   216.85   314.24   531.09   151.79   157.12    308.91
94.0   221.51   237.68   314.24   551.92   166.38   157.12    323.50
```

```
 95.0   223.87   258.70   314.24   572.94   181.09   157.12    338.21
 96.0   226.22   279.90   314.24   594.14   195.93   157.12    353.05
 97.0   228.58   301.28   314.24   615.52   210.89   157.12    368.01
 98.0   230.94   322.83   314.24   637.07   225.98   157.12    383.10
 99.0   233.29   344.56   314.24   658.80   241.20   157.12    398.32
100.0   235.65   366.47   314.24   680.71   256.53   157.12    413.65
101.0   238.01   388.55   314.24   702.79   271.99   157.12    429.11
102.0   240.36   410.80   314.24   725.04   287.56   157.12    444.68
103.0   242.72   433.23   314.24   747.47   303.26   157.12    460.38
104.0   245.08   455.82   314.24   770.06   319.07   157.12    476.19
105.0   247.43   478.58   323.72   802.30   335.01   161.86    496.86
106.0   249.79   501.51   333.69   835.20   351.06   166.85    517.90
107.0   252.15   524.60   344.17   868.77   367.22   172.08    539.30
108.0   254.50   547.86   355.14   903.00   383.50   177.57    561.07
109.0   256.86   571.28   366.61   937.89   399.90   183.31    583.20
110.0   259.21   594.86   378.58   973.44   416.40   189.29    605.69
111.0   261.57   618.60   391.05  1009.66   433.02   195.53    628.55
112.0   263.93   642.51   404.02  1046.53   449.75   202.01    651.76
113.0   266.28   666.56   417.49  1084.05   466.60   208.74    675.34
114.0   268.64   690.78   429.46  1120.24   483.55   214.73    698.28
115.0   271.00   715.15   439.94  1155.08   500.61   219.97    720.57
116.0   273.35   739.67   448.91  1188.59   517.77   224.46    742.23
117.0   275.71   764.35   456.40  1220.75   535.05   228.20    763.24
118.0   278.07   789.18   462.38  1251.56   552.42   231.19    783.61
119.0   280.42   814.15   466.87  1281.02   569.91   233.43    803.34
120.0   282.78   839.28   469.86  1309.14   587.50   234.93    822.43
121.0   285.14   864.55   471.36  1335.91   605.19   235.68    840.87
122.0   287.49   889.97   471.36  1361.33   622.98   235.68    858.66
123.0   289.85   915.62   471.36  1386.98   640.93   235.68    876.61
124.0   292.21   941.51   471.36  1412.86   659.05   235.68    894.73
125.0   294.56   967.63   471.36  1438.99   677.34   235.68    913.02
126.0   296.92   994.00   471.36  1465.36   695.80   235.68    931.48
127.0   299.28  1020.60   471.36  1491.96   714.42   235.68    950.10
128.0   301.63  1047.44   471.36  1518.80   733.21   235.68    968.89
129.0   303.99  1074.53   471.36  1545.88   752.17   235.68    987.85
130.0   306.35  1101.85   471.36  1573.21   771.29   235.68   1006.97
131.0   308.70  1129.41   471.36  1600.77   790.58   235.68   1026.26
132.0   311.06  1157.21   471.36  1628.56   810.04   235.68   1045.72
133.0   313.41  1185.24   471.36  1656.60   829.67   235.68   1065.35
134.0   315.77  1213.52   471.36  1684.88   849.46   235.68   1085.14
135.0   318.13  1242.04   471.36  1713.39   869.42   235.68   1105.10
136.0   320.48  1270.79   471.36  1742.15   889.55   235.68   1125.23
137.0   322.84  1299.78   471.36  1771.14   909.85   235.68   1145.53
138.0   325.20  1329.02   471.36  1800.37   930.31   235.68   1165.99
139.0   327.55  1358.49   471.36  1829.85   950.94   235.68   1186.62
140.0   329.91  1388.20   471.36  1859.56   971.74   235.68   1207.42
141.0   332.27  1418.15   471.36  1889.51   992.70   235.68   1228.38
142.0   334.62  1448.33   471.36  1919.69  1013.83   235.68   1249.51
```

```
143.0   336.98  1478.76   471.36  1950.12  1035.13   235.68   1270.81
144.0   339.34  1509.43   471.36  1980.79  1056.60   235.68   1292.28
145.0   341.69  1540.33   471.36  2011.69  1078.23   235.68   1313.91
146.0   344.05  1571.48   471.36  2042.84  1100.03   235.68   1335.71
147.0   346.41  1602.86   471.36  2074.22  1122.00   235.68   1357.68
148.0   348.76  1634.48   471.36  2105.84  1144.14   235.68   1379.82
149.0   351.12  1666.34   471.36  2137.70  1166.44   235.68   1402.12
150.0   353.48  1698.44   471.36  2169.80  1188.91   235.68   1424.59
151.0   355.83  1730.78   471.36  2202.14  1211.55   235.68   1447.23
152.0   358.19  1763.36   471.36  2234.72  1234.35   235.68   1470.03
153.0   360.54  1796.18   471.36  2267.53  1257.32   235.68   1493.00
154.0   362.90  1829.23   471.36  2300.59  1280.46   235.68   1516.14
155.0   365.26  1862.53   471.36  2333.88  1303.77   235.68   1539.45
156.0   367.61  1896.06   471.36  2367.42  1327.24   235.68   1562.92
157.0   369.97  1929.83   471.36  2401.19  1350.88   235.68   1586.56
158.0   372.33  1963.84   471.36  2435.20  1374.69   235.68   1610.37
159.0   374.68  1998.09   471.36  2469.45  1398.67   235.68   1634.34
160.0   377.04  2032.58   471.36  2503.94  1422.81   235.68   1658.49
161.0   379.40  2067.31   471.36  2538.67  1447.12   235.68   1682.80
162.0   381.75  2102.28   471.36  2573.64  1471.59   235.68   1707.27
163.0   384.11  2137.48   471.36  2608.84  1496.24   235.68   1731.92
164.0   386.47  2172.93   471.36  2644.29  1521.05   235.68   1756.73
165.0   388.82  2208.61   471.36  2679.97  1546.03   235.68   1781.71
166.0   391.18  2244.54   471.36  2715.89  1571.17   235.68   1806.85
167.0   393.54  2280.70   471.36  2752.06  1596.49   235.68   1832.17
168.0   395.89  2317.10   471.36  2788.46  1621.97   235.68   1857.65
169.0   398.25  2353.74   471.36  2825.10  1647.62   235.68   1883.30
170.0   400.61  2390.62   471.36  2861.98  1673.43   235.68   1909.11
171.0   402.96  2427.73   471.36  2899.09  1699.41   235.68   1935.09
172.0   405.32  2465.09   471.36  2936.45  1725.56   235.68   1961.24
173.0   407.67  2502.69   471.36  2974.05  1751.88   235.68   1987.56
174.0   410.03  2540.52   471.36  3011.88  1778.36   235.68   2014.04
175.0   412.39  2578.59   471.36  3049.95  1805.02   235.68   2040.70
176.0   414.74  2616.91   471.36  3088.27  1831.84   235.68   2067.51
177.0   417.10  2655.46   471.36  3126.82  1858.82   235.68   2094.50
178.0   419.46  2694.25   471.36  3165.61  1885.97   235.68   2121.65
179.0   421.81  2733.28   471.36  3204.64  1913.29   235.68   2148.97
180.0   424.17  2772.55   471.36  3243.91  1940.78   235.68   2176.46
181.0   426.53  2812.05   471.36  3283.41  1968.44   235.68   2204.12
182.0   428.88  2851.80   471.36  3323.16  1996.26   235.68   2231.94
183.0   431.24  2891.78   471.36  3363.14  2024.25   235.68   2259.93
184.0   433.60  2932.01   471.36  3403.37  2052.41   235.68   2288.09
185.0   435.95  2972.47   471.36  3443.83  2080.73   235.68   2316.41
186.0   438.31  3013.17   471.36  3484.53  2109.22   235.68   2344.90
187.0   440.67  3054.11   471.36  3525.47  2137.88   235.68   2373.56
188.0   443.02  3095.29   471.36  3566.65  2166.71   235.68   2402.39
189.0   445.38  3136.71   471.36  3608.07  2195.70   235.68   2431.38
190.0   447.73  3178.37   487.55  3665.92  2224.86   243.78   2468.64
```

```
191.0   450.09  3220.27   504.60  3724.87  2254.19   252.30   2506.49
192.0   452.45  3262.40   522.50  3784.90  2283.68   261.25   2544.93
193.0   454.80  3304.78   541.25  3846.02  2313.34   270.62   2583.97
194.0   457.16  3347.39   560.85  3908.24  2343.17   280.42   2623.60
195.0   459.52  3390.24   581.30  3971.55  2373.17   290.65   2663.82
196.0   461.87  3433.33   602.61  4035.95  2403.33   301.31   2704.64
197.0   464.23  3476.67   624.77  4101.44  2433.67   312.39   2746.05
198.0   466.59  3520.23   647.78  4168.02  2464.16   323.89   2788.06
199.0   468.94  3564.04   668.24  4232.28  2494.83   334.12   2828.95
200.0   471.30  3608.09   686.14  4294.23  2525.66   343.07   2868.73



 AXIAL LOAD VS SETTLEMENT CURVES
 -------------------------------


    RESULT FROM TREND (AVERAGED) LINE

    TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
       TON             IN.             TON            IN.
   0.1372E+00      0.2455E-04      0.5019E-02      0.1000E-04
   0.6862E+00      0.1228E-03      0.2509E-01      0.5000E-04
   0.1372E+01      0.2455E-03      0.5019E-01      0.1000E-03
   0.6884E+02      0.1230E-01      0.2509E+01      0.5000E-02
   0.1033E+03      0.1844E-01      0.3764E+01      0.7500E-02
   0.1377E+03      0.2459E-01      0.5019E+01      0.1000E-01
   0.3443E+03      0.6149E-01      0.1255E+02      0.2500E-01
   0.6886E+03      0.1230E+00      0.2509E+02      0.5000E-01
   0.1029E+04      0.1842E+00      0.3764E+02      0.7500E-01
   0.1336E+04      0.2428E+00      0.5019E+02      0.1000E+00
   0.2536E+04      0.5256E+00      0.1255E+03      0.2500E+00
   0.3356E+04      0.8706E+00      0.2509E+03      0.5000E+00
   0.3837E+04      0.1510E+01      0.3842E+03      0.1080E+01
   0.4041E+04      0.3160E+01      0.5935E+03      0.2700E+01
   0.4102E+04      0.5869E+01      0.6656E+03      0.5400E+01


    RESULT FROM UPPER-BOUND LINE

    TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
       TON             IN.             TON            IN.
   0.1957E+00      0.3054E-04      0.7497E-02      0.1000E-04
   0.9786E+00      0.1527E-03      0.3748E-01      0.5000E-04
   0.1957E+01      0.3054E-03      0.7497E-01      0.1000E-03
   0.9831E+02      0.1531E-01      0.3748E+01      0.5000E-02
   0.1475E+03      0.2297E-01      0.5623E+01      0.7500E-02
```

```
0.1967E+03      0.3063E-01      0.7497E+01      0.1000E-01
0.4917E+03      0.7658E-01      0.1874E+02      0.2500E-01
0.9833E+03      0.1532E+00      0.3748E+02      0.5000E-01
0.1455E+04      0.2284E+00      0.5623E+02      0.7500E-01
0.1867E+04      0.2985E+00      0.7497E+02      0.1000E+00
0.3271E+04      0.6074E+00      0.1874E+03      0.2500E+00
0.3907E+04      0.9359E+00      0.3748E+03      0.5000E+00
0.4113E+04      0.1544E+01      0.5146E+03      0.1080E+01
0.4255E+04      0.3185E+01      0.6587E+03      0.2700E+01
0.4279E+04      0.5889E+01      0.6827E+03      0.5400E+01


 RESULT FROM LOWER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON            IN.              TON            IN.
0.8432E-01      0.1902E-04      0.2541E-02      0.1000E-04
0.4216E+00      0.9511E-04      0.1271E-01      0.5000E-04
0.8432E+00      0.1902E-03      0.2541E-01      0.1000E-03
0.4224E+02      0.9517E-02      0.1271E+01      0.5000E-02
0.6337E+02      0.1428E-01      0.1906E+01      0.7500E-02
0.8450E+02      0.1904E-01      0.2541E+01      0.1000E-01
0.2113E+03      0.4760E-01      0.6353E+01      0.2500E-01
0.4226E+03      0.9520E-01      0.1271E+02      0.5000E-01
0.6339E+03      0.1428E+00      0.1906E+02      0.7500E-01
0.8365E+03      0.1897E+00      0.2541E+02      0.1000E+00
0.1782E+04      0.4425E+00      0.6353E+02      0.2500E+00
0.2782E+04      0.8032E+00      0.1271E+03      0.5000E+00
0.3558E+04      0.1475E+01      0.2539E+03      0.1080E+01
0.3826E+04      0.3134E+01      0.5283E+03      0.2700E+01
0.3922E+04      0.5849E+01      0.6450E+03      0.5400E+01
```

Piers
Seismic Axial Capacity
9-ft CIDH with 10-ft Casing

```
     =========================================================================

                      SHAFT for Windows, Version 2017.8.4

                       Serial Number :  253582343

                    VERTICALLY LOADED DRILLED SHAFT ANALYSIS
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                           All Rights Reserved

     =========================================================================


     Path to file locations      : C:\Users\JBonfiglio\OneDrive -
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
     Name of input data file     : Piers_9ft_Seismic_Final_Rev.sf8d
     Name of output file         : Piers_9ft_Seismic_Final_Rev.sf8o
     Name of plot output file    : Piers_9ft_Seismic_Final_Rev.sf8p
     Name of runtime file        : Piers_9ft_Seismic_Final_Rev.sf8r


     -------------------------------------------------------------------------
                          Time and Date of Analysis
     -------------------------------------------------------------------------

               Date:  May 18, 2020     Time:  11:52:53

     Camino Del Mar Bridge Replacement - Phase 0


     PROPOSED DEPTH =     200.0 FT
     ----------------


     NUMBER OF LAYERS =    8
     ------------------


     WATER TABLE DEPTH =       0.0 FT.
     -------------------




     SOIL INFORMATION
     ---------------

     LAYER NO 1----CLAY
```

```
  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.600E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.000E+00


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.760E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.120E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 2----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.760E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.120E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.853E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
```

```
  DEPTH, FT                                            = 0.190E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)              = 0.100E+01


LAYER NO 3----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                      = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                           = 0.853E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                   = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.190E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                      = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                           = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                   = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.350E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)              = 0.100E+01


LAYER NO 4----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                      = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                           = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                   = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.350E+02
```

```
  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                         = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                              = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                      = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST           = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                              = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT                = 0.100E+11
  DEPTH, FT                                               = 0.650E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)                  = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)                 = 0.100E+01


LAYER NO 5----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                         = 0.550E+00
  END BEARING COEFFICIENT-Nc                              = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                      = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST           = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                              = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT                = 0.100E+11
  DEPTH, FT                                               = 0.650E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                         = 0.550E+00
  END BEARING COEFFICIENT-Nc                              = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                      = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST           = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                              = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT                = 0.100E+11
  DEPTH, FT                                               = 0.820E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)                  = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)                 = 0.100E+01


LAYER NO 6----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
```

```
  SKIN FRICTION COEFFICIENT- BETA                   = 0.280E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.820E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.122E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.122E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.207E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 8----CLAY
```

```
  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                         = 0.510E+00
  END BEARING COEFFICIENT-Nc                              = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                      = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST           = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                              = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT                = 0.100E+11
  DEPTH, FT                                               = 0.207E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                         = 0.510E+00
  END BEARING COEFFICIENT-Nc                              = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                      = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST           = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                              = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT                = 0.100E+11
  DEPTH, FT                                               = 0.255E+03


  LRFD RESISTANCE FACTOR (SIDE FRICTION)                  = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)                 = 0.100E+01


(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    9.000  FT.
 MAXIMUM SHAFT DIAMETER    =    9.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
```

```
COMPUTATION RESULTS
-------------------



- CASE ANALYZED      :     1
  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1



DRILLED SHAFT INFORMATION

-------------------------

 DIAMETER OF STEM          =    9.000  FT.
 DIAMETER OF BASE          =    9.000  FT.
 END OF STEM TO BASE       =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 AREA OF ONE PERCENT STEEL =   91.621  SQ.IN.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
 VOLUME OF UNDERREAM       =    0.000  CU.YDS.
 SHAFT LENGTH              =  200.000  FT.


PREDICTED RESULTS
-----------------

QS      = ULTIMATE SIDE RESISTANCE;
QB      = ULTIMATE BASE RESISTANCE;
WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
QU      = TOTAL ULTIMATE RESISTANCE;
LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE SIDE RESISTANCE;
LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE BASE RESISTANCE
LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

LENGTH  VOLUME     QS      QB      QU     LRFD QS  LRFD QB   LRFD QU
 (FT)   (CU.YDS) (TONS)   (TONS)  (TONS)   (TONS)   (TONS)    (TONS)
83.0   195.59    18.75   313.08   331.83    18.75   313.08    331.83
84.0   197.95    37.70   313.57   351.28    37.70   313.57    351.28
85.0   200.30    56.84   313.91   370.75    56.84   313.91    370.75
86.0   202.66    76.18   314.11   390.29    76.18   314.11    390.29
87.0   205.02    95.71   314.21   409.92    95.71   314.21    409.92
88.0   207.37   115.43   314.24   429.67   115.43   314.24    429.67
```

```
  89.0   209.73   135.34   314.24    449.58   135.34   314.24    449.58
  90.0   212.08   155.44   314.24    469.68   155.44   314.24    469.68
  91.0   214.44   175.72   314.24    489.96   175.72   314.24    489.96
  92.0   216.80   196.19   314.24    510.43   196.19   314.24    510.43
  93.0   219.15   216.85   314.24    531.09   216.85   314.24    531.09
  94.0   221.51   237.68   314.24    551.92   237.68   314.24    551.92
  95.0   223.87   258.70   314.24    572.94   258.70   314.24    572.94
  96.0   226.22   279.90   314.24    594.14   279.90   314.24    594.14
  97.0   228.58   301.28   314.24    615.52   301.28   314.24    615.52
  98.0   230.94   322.83   314.24    637.07   322.83   314.24    637.07
  99.0   233.29   344.56   314.24    658.80   344.56   314.24    658.80
 100.0   235.65   366.47   314.24    680.71   366.47   314.24    680.71
 101.0   238.01   388.55   314.24    702.79   388.55   314.24    702.79
 102.0   240.36   410.80   314.24    725.04   410.80   314.24    725.04
 103.0   242.72   433.23   314.24    747.47   433.23   314.24    747.47
 104.0   245.08   455.82   314.24    770.06   455.82   314.24    770.06
 105.0   247.43   478.58   323.72    802.30   478.58   323.72    802.30
 106.0   249.79   501.51   333.69    835.20   501.51   333.69    835.20
 107.0   252.15   524.60   344.17    868.77   524.60   344.17    868.77
 108.0   254.50   547.86   355.14    903.00   547.86   355.14    903.00
 109.0   256.86   571.28   366.61    937.89   571.28   366.61    937.89
 110.0   259.21   594.86   378.58    973.44   594.86   378.58    973.44
 111.0   261.57   618.60   391.05   1009.66   618.60   391.05   1009.66
 112.0   263.93   642.51   404.02   1046.53   642.51   404.02   1046.53
 113.0   266.28   666.56   417.49   1084.05   666.56   417.49   1084.05
 114.0   268.64   690.78   429.46   1120.24   690.78   429.46   1120.24
 115.0   271.00   715.15   439.94   1155.08   715.15   439.94   1155.08
 116.0   273.35   739.67   448.91   1188.59   739.67   448.91   1188.59
 117.0   275.71   764.35   456.40   1220.75   764.35   456.40   1220.75
 118.0   278.07   789.18   462.38   1251.56   789.18   462.38   1251.56
 119.0   280.42   814.15   466.87   1281.02   814.15   466.87   1281.02
 120.0   282.78   839.28   469.86   1309.14   839.28   469.86   1309.14
 121.0   285.14   864.55   471.36   1335.91   864.55   471.36   1335.91
 122.0   287.49   889.97   471.36   1361.33   889.97   471.36   1361.33
 123.0   289.85   915.62   471.36   1386.98   915.62   471.36   1386.98
 124.0   292.21   941.51   471.36   1412.86   941.51   471.36   1412.86
 125.0   294.56   967.63   471.36   1438.99   967.63   471.36   1438.99
 126.0   296.92   994.00   471.36   1465.36   994.00   471.36   1465.36
 127.0   299.28  1020.60   471.36   1491.96  1020.60   471.36   1491.96
 128.0   301.63  1047.44   471.36   1518.80  1047.44   471.36   1518.80
 129.0   303.99  1074.53   471.36   1545.88  1074.53   471.36   1545.88
 130.0   306.35  1101.85   471.36   1573.21  1101.85   471.36   1573.21
 131.0   308.70  1129.41   471.36   1600.77  1129.41   471.36   1600.77
 132.0   311.06  1157.21   471.36   1628.56  1157.21   471.36   1628.56
 133.0   313.41  1185.24   471.36   1656.60  1185.24   471.36   1656.60
 134.0   315.77  1213.52   471.36   1684.88  1213.52   471.36   1684.88
 135.0   318.13  1242.04   471.36   1713.39  1242.04   471.36   1713.39
 136.0   320.48  1270.79   471.36   1742.15  1270.79   471.36   1742.15
```

```
137.0   322.84  1299.78   471.36  1771.14  1299.78   471.36   1771.14
138.0   325.20  1329.02   471.36  1800.37  1329.02   471.36   1800.37
139.0   327.55  1358.49   471.36  1829.85  1358.49   471.36   1829.85
140.0   329.91  1388.20   471.36  1859.56  1388.20   471.36   1859.56
141.0   332.27  1418.15   471.36  1889.51  1418.15   471.36   1889.51
142.0   334.62  1448.33   471.36  1919.69  1448.33   471.36   1919.69
143.0   336.98  1478.76   471.36  1950.12  1478.76   471.36   1950.12
144.0   339.34  1509.43   471.36  1980.79  1509.43   471.36   1980.79
145.0   341.69  1540.33   471.36  2011.69  1540.33   471.36   2011.69
146.0   344.05  1571.48   471.36  2042.84  1571.48   471.36   2042.84
147.0   346.41  1602.86   471.36  2074.22  1602.86   471.36   2074.22
148.0   348.76  1634.48   471.36  2105.84  1634.48   471.36   2105.84
149.0   351.12  1666.34   471.36  2137.70  1666.34   471.36   2137.70
150.0   353.48  1698.44   471.36  2169.80  1698.44   471.36   2169.80
151.0   355.83  1730.78   471.36  2202.14  1730.78   471.36   2202.14
152.0   358.19  1763.36   471.36  2234.72  1763.36   471.36   2234.72
153.0   360.54  1796.18   471.36  2267.53  1796.18   471.36   2267.53
154.0   362.90  1829.23   471.36  2300.59  1829.23   471.36   2300.59
155.0   365.26  1862.53   471.36  2333.88  1862.53   471.36   2333.88
156.0   367.61  1896.06   471.36  2367.42  1896.06   471.36   2367.42
157.0   369.97  1929.83   471.36  2401.19  1929.83   471.36   2401.19
158.0   372.33  1963.84   471.36  2435.20  1963.84   471.36   2435.20
159.0   374.68  1998.09   471.36  2469.45  1998.09   471.36   2469.45
160.0   377.04  2032.58   471.36  2503.94  2032.58   471.36   2503.94
161.0   379.40  2067.31   471.36  2538.67  2067.31   471.36   2538.67
162.0   381.75  2102.28   471.36  2573.64  2102.28   471.36   2573.64
163.0   384.11  2137.48   471.36  2608.84  2137.48   471.36   2608.84
164.0   386.47  2172.93   471.36  2644.29  2172.93   471.36   2644.29
165.0   388.82  2208.61   471.36  2679.97  2208.61   471.36   2679.97
166.0   391.18  2244.54   471.36  2715.89  2244.54   471.36   2715.89
167.0   393.54  2280.70   471.36  2752.06  2280.70   471.36   2752.06
168.0   395.89  2317.10   471.36  2788.46  2317.10   471.36   2788.46
169.0   398.25  2353.74   471.36  2825.10  2353.74   471.36   2825.10
170.0   400.61  2390.62   471.36  2861.98  2390.62   471.36   2861.98
171.0   402.96  2427.73   471.36  2899.09  2427.73   471.36   2899.09
172.0   405.32  2465.09   471.36  2936.45  2465.09   471.36   2936.45
173.0   407.67  2502.69   471.36  2974.05  2502.69   471.36   2974.05
174.0   410.03  2540.52   471.36  3011.88  2540.52   471.36   3011.88
175.0   412.39  2578.59   471.36  3049.95  2578.59   471.36   3049.95
176.0   414.74  2616.91   471.36  3088.27  2616.91   471.36   3088.27
177.0   417.10  2655.46   471.36  3126.82  2655.46   471.36   3126.82
178.0   419.46  2694.25   471.36  3165.61  2694.25   471.36   3165.61
179.0   421.81  2733.28   471.36  3204.64  2733.28   471.36   3204.64
180.0   424.17  2772.55   471.36  3243.91  2772.55   471.36   3243.91
181.0   426.53  2812.05   471.36  3283.41  2812.05   471.36   3283.41
182.0   428.88  2851.80   471.36  3323.16  2851.80   471.36   3323.16
183.0   431.24  2891.78   471.36  3363.14  2891.78   471.36   3363.14
184.0   433.60  2932.01   471.36  3403.37  2932.01   471.36   3403.37
```

```
185.0   435.95  2972.47   471.36  3443.83  2972.47   471.36   3443.83
186.0   438.31  3013.17   471.36  3484.53  3013.17   471.36   3484.53
187.0   440.67  3054.11   471.36  3525.47  3054.11   471.36   3525.47
188.0   443.02  3095.29   471.36  3566.65  3095.29   471.36   3566.65
189.0   445.38  3136.71   471.36  3608.07  3136.71   471.36   3608.07
190.0   447.73  3178.37   487.55  3665.92  3178.37   487.55   3665.92
191.0   450.09  3220.27   504.60  3724.87  3220.27   504.60   3724.87
192.0   452.45  3262.40   522.50  3784.90  3262.40   522.50   3784.90
193.0   454.80  3304.78   541.25  3846.02  3304.78   541.25   3846.02
194.0   457.16  3347.39   560.85  3908.24  3347.39   560.85   3908.24
195.0   459.52  3390.24   581.30  3971.55  3390.24   581.30   3971.55
196.0   461.87  3433.33   602.61  4035.95  3433.33   602.61   4035.95
197.0   464.23  3476.67   624.77  4101.44  3476.67   624.77   4101.44
198.0   466.59  3520.23   647.78  4168.02  3520.23   647.78   4168.02
199.0   468.94  3564.04   668.24  4232.28  3564.04   668.24   4232.28
200.0   471.30  3608.09   686.14  4294.23  3608.09   686.14   4294.23



 AXIAL LOAD VS SETTLEMENT CURVES
 -------------------------------


    RESULT FROM TREND (AVERAGED) LINE

    TOP   LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
       TON             IN.               TON            IN.
   0.1372E+00       0.2455E-04      0.5019E-02      0.1000E-04
   0.6862E+00       0.1228E-03      0.2509E-01      0.5000E-04
   0.1372E+01       0.2455E-03      0.5019E-01      0.1000E-03
   0.6884E+02       0.1230E-01      0.2509E+01      0.5000E-02
   0.1033E+03       0.1844E-01      0.3764E+01      0.7500E-02
   0.1377E+03       0.2459E-01      0.5019E+01      0.1000E-01
   0.3443E+03       0.6149E-01      0.1255E+02      0.2500E-01
   0.6886E+03       0.1230E+00      0.2509E+02      0.5000E-01
   0.1029E+04       0.1842E+00      0.3764E+02      0.7500E-01
   0.1336E+04       0.2428E+00      0.5019E+02      0.1000E+00
   0.2536E+04       0.5256E+00      0.1255E+03      0.2500E+00
   0.3356E+04       0.8706E+00      0.2509E+03      0.5000E+00
   0.3837E+04       0.1510E+01      0.3842E+03      0.1080E+01
   0.4041E+04       0.3160E+01      0.5935E+03      0.2700E+01
   0.4102E+04       0.5869E+01      0.6656E+03      0.5400E+01


    RESULT FROM UPPER-BOUND LINE

    TOP   LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
```

```
    TON             IN.              TON            IN.
0.1957E+00      0.3054E-04      0.7497E-02      0.1000E-04
0.9786E+00      0.1527E-03      0.3748E-01      0.5000E-04
0.1957E+01      0.3054E-03      0.7497E-01      0.1000E-03
0.9831E+02      0.1531E-01      0.3748E+01      0.5000E-02
0.1475E+03      0.2297E-01      0.5623E+01      0.7500E-02
0.1967E+03      0.3063E-01      0.7497E+01      0.1000E-01
0.4917E+03      0.7658E-01      0.1874E+02      0.2500E-01
0.9833E+03      0.1532E+00      0.3748E+02      0.5000E-01
0.1455E+04      0.2284E+00      0.5623E+02      0.7500E-01
0.1867E+04      0.2985E+00      0.7497E+02      0.1000E+00
0.3271E+04      0.6074E+00      0.1874E+03      0.2500E+00
0.3907E+04      0.9359E+00      0.3748E+03      0.5000E+00
0.4113E+04      0.1544E+01      0.5146E+03      0.1080E+01
0.4255E+04      0.3185E+01      0.6587E+03      0.2700E+01
0.4279E+04      0.5889E+01      0.6827E+03      0.5400E+01


 RESULT FROM LOWER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON             IN.              TON            IN.
0.8432E-01      0.1902E-04      0.2541E-02      0.1000E-04
0.4216E+00      0.9511E-04      0.1271E-01      0.5000E-04
0.8432E+00      0.1902E-03      0.2541E-01      0.1000E-03
0.4224E+02      0.9517E-02      0.1271E+01      0.5000E-02
0.6337E+02      0.1428E-01      0.1906E+01      0.7500E-02
0.8450E+02      0.1904E-01      0.2541E+01      0.1000E-01
0.2113E+03      0.4760E-01      0.6353E+01      0.2500E-01
0.4226E+03      0.9520E-01      0.1271E+02      0.5000E-01
0.6339E+03      0.1428E+00      0.1906E+02      0.7500E-01
0.8365E+03      0.1897E+00      0.2541E+02      0.1000E+00
0.1782E+04      0.4425E+00      0.6353E+02      0.2500E+00
0.2782E+04      0.8032E+00      0.1271E+03      0.5000E+00
0.3558E+04      0.1475E+01      0.2539E+03      0.1080E+01
0.3826E+04      0.3134E+01      0.5283E+03      0.2700E+01
0.3922E+04      0.5849E+01      0.6450E+03      0.5400E+01
```

Piers
Static Axial Capacity
10-ft CIDH with 11-ft Casing

```
==========================================================================

                 SHAFT for Windows, Version 2017.8.4

                  Serial Number :  253582343

               VERTICALLY LOADED DRILLED SHAFT ANALYSIS
                (c) Copyright ENSOFT, Inc., 1987-2017
                       All Rights Reserved

==========================================================================


Path to file locations      : C:\Users\JBonfiglio\OneDrive -
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
Name of input data file     : Piers_10ft_Final_Rev.sf8d
Name of output file         : Piers_10ft_Final_Rev.sf8o
Name of plot output file    : Piers_10ft_Final_Rev.sf8p
Name of runtime file        : Piers_10ft_Final_Rev.sf8r


--------------------------------------------------------------------------
                     Time and Date of Analysis
--------------------------------------------------------------------------

          Date:  May 18, 2020     Time:  09:56:42

Camino Del Mar Bridge Replacement - Phase 0


PROPOSED DEPTH =     200.0 FT
----------------


NUMBER OF LAYERS =    8
------------------


WATER TABLE DEPTH =       0.0 FT.
-------------------




SOIL INFORMATION
---------------

LAYER NO 1----SAND
```

```
  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.100E+01
  INTERNAL FRICTION ANGLE, DEG.                      = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.000E+00

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.690E+00
  INTERNAL FRICTION ANGLE, DEG.                      = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.120E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)             = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)            = 0.500E+00


LAYER NO 2----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00
  END BEARING COEFFICIENT-Nc                         = 0.744E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.120E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00
  END BEARING COEFFICIENT-Nc                         = 0.828E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.190E+02
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  LRFD RESISTANCE FACTOR (SIDE FRICTION)             = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)            = 0.500E+00


LAYER NO 3----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.610E+00
  INTERNAL FRICTION ANGLE, DEG.                      = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.190E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.470E+00
  INTERNAL FRICTION ANGLE, DEG.                      = 0.280E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.350E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)             = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)            = 0.500E+00


LAYER NO 4----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.700E+00
  INTERNAL FRICTION ANGLE, DEG.                      = 0.320E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.350E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.410E+00
  INTERNAL FRICTION ANGLE, DEG.                      = 0.320E+02
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```
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.650E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 5----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.650E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.820E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00


LAYER NO 6----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.280E+00
  INTERNAL FRICTION ANGLE, DEG.                     = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.820E+02
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```
  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                     = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                       = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.122E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)              = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)             = 0.500E+00


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                     = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                       = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.122E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                     = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                       = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.207E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)              = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)             = 0.500E+00


LAYER NO 8----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                     = 0.510E+00
  END BEARING COEFFICIENT-Nc                          = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                  = 0.400E+04
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```
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.207E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.510E+00
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.255E+03


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.500E+00



(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =   10.000  FT.
 MAXIMUM SHAFT DIAMETER    =   10.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN





COMPUTATION RESULTS
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- CASE ANALYZED      :     1
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  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1



DRILLED SHAFT INFORMATION

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 DIAMETER OF STEM          =   10.000  FT.
 DIAMETER OF BASE          =   10.000  FT.
 END OF STEM TO BASE       =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 AREA OF ONE PERCENT STEEL =  113.112  SQ.IN.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
 VOLUME OF UNDERREAM       =    0.000  CU.YDS.
 SHAFT LENGTH              =  200.000  FT.


PREDICTED RESULTS
-----------------

QS      = ULTIMATE SIDE RESISTANCE;
QB      = ULTIMATE BASE RESISTANCE;
WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
QU      = TOTAL ULTIMATE RESISTANCE;
LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE SIDE RESISTANCE;
LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE BASE RESISTANCE
LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

LENGTH  VOLUME     QS      QB      QU     LRFD QS  LRFD QB   LRFD QU
 (FT)   (CU.YDS) (TONS)   (TONS)  (TONS)   (TONS)   (TONS)    (TONS)
83.0   241.47    20.84   332.14   352.98    14.59   166.07    180.66
84.0   244.38    41.89   334.83   376.72    29.32   167.42    196.74
85.0   247.29    63.16   337.29   400.45    44.21   168.65    212.86
86.0   250.20    84.65   339.52   424.17    59.25   169.76    229.01
87.0   253.11   106.35   341.51   447.85    74.44   170.75    245.20
88.0   256.01   128.26   343.25   471.50    89.78   171.62    261.40
89.0   258.92   150.38   344.72   495.10   105.26   172.36    277.63
90.0   261.83   172.71   345.93   518.64   120.90   172.97    293.86
91.0   264.74   195.25   346.90   542.15   136.67   173.45    310.12
92.0   267.65   217.99   347.65   565.64   152.59   173.83    326.42
93.0   270.56   240.94   348.21   589.16   168.66   174.11    342.77
94.0   273.47   264.09   348.62   612.71   184.86   174.31    359.17
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  95.0   276.38   287.45   348.89   636.33   201.21   174.44    375.66
  96.0   279.29   311.00   349.05   660.05   217.70   174.52    392.22
  97.0   282.20   334.75   349.13   683.88   234.33   174.56    408.89
  98.0   285.11   358.70   349.15   707.86   251.09   174.58    425.67
  99.0   288.02   382.85   349.15   732.00   267.99   174.58    442.57
 100.0   290.93   407.19   349.15   756.34   285.03   174.58    459.61
 101.0   293.84   431.72   349.15   780.88   302.21   174.58    476.78
 102.0   296.74   456.45   349.15   805.60   319.51   174.58    494.09
 103.0   299.65   481.36   358.56   839.92   336.95   179.28    516.23
 104.0   302.56   506.47   368.40   874.87   354.53   184.20    538.73
 105.0   305.47   531.76   378.70   910.45   372.23   189.35    561.58
 106.0   308.38   557.23   389.44   946.67   390.06   194.72    584.78
 107.0   311.29   582.89   400.63   983.52   408.02   200.32    608.34
 108.0   314.20   608.73   412.27  1021.00   426.11   206.14    632.25
 109.0   317.11   634.75   424.36  1059.11   444.33   212.18    656.51
 110.0   320.02   660.96   436.89  1097.85   462.67   218.45    681.12
 111.0   322.93   687.34   449.87  1137.21   481.14   224.94    706.07
 112.0   325.84   713.89   463.30  1177.20   499.73   231.65    731.38
 113.0   328.75   740.63   475.39  1216.01   518.44   237.69    756.13
 114.0   331.66   767.53   486.13  1253.66   537.27   243.07    780.34
 115.0   334.56   794.61   495.53  1290.14   556.23   247.77    803.99
 116.0   337.47   821.86   503.59  1325.45   575.30   251.79    827.10
 117.0   340.38   849.28   510.30  1359.58   594.49   255.15    849.65
 118.0   343.29   876.86   515.67  1392.54   613.81   257.84    871.64
 119.0   346.20   904.62   519.70  1424.32   633.23   259.85    893.08
 120.0   349.11   932.53   522.39  1454.92   652.77   261.19    913.97
 121.0   352.02   960.61   523.73  1484.35   672.43   261.87    934.30
 122.0   354.93   988.86   523.73  1512.59   692.20   261.87    954.07
 123.0   357.84  1017.35   523.73  1541.09   712.15   261.87    974.01
 124.0   360.75  1046.12   523.73  1569.85   732.28   261.87    994.15
 125.0   363.66  1075.15   523.73  1598.88   752.60   261.87   1014.47
 126.0   366.57  1104.44   523.73  1628.17   773.11   261.87   1034.97
 127.0   369.48  1134.00   523.73  1657.73   793.80   261.87   1055.67
 128.0   372.39  1163.83   523.73  1687.56   814.68   261.87   1076.54
 129.0   375.29  1193.92   523.73  1717.65   835.74   261.87   1097.61
 130.0   378.20  1224.27   523.73  1748.01   856.99   261.87   1118.86
 131.0   381.11  1254.90   523.73  1778.63   878.43   261.87   1140.29
 132.0   384.02  1285.78   523.73  1809.52   900.05   261.87   1161.91
 133.0   386.93  1316.94   523.73  1840.67   921.86   261.87   1183.72
 134.0   389.84  1348.35   523.73  1872.09   943.85   261.87   1205.71
 135.0   392.75  1380.04   523.73  1903.77   966.03   261.87   1227.89
 136.0   395.66  1411.99   523.73  1935.72   988.39   261.87   1250.26
 137.0   398.57  1444.20   523.73  1967.94  1010.94   261.87   1272.81
 138.0   401.48  1476.68   523.73  2000.42  1033.68   261.87   1295.54
 139.0   404.39  1509.43   523.73  2033.16  1056.60   261.87   1318.47
 140.0   407.30  1542.44   523.73  2066.17  1079.71   261.87   1341.57
 141.0   410.21  1575.72   523.73  2099.45  1103.00   261.87   1364.87
 142.0   413.11  1609.26   523.73  2132.99  1126.48   261.87   1388.35
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143.0   416.02  1643.07   523.73  2166.80  1150.15   261.87   1412.01
144.0   418.93  1677.14   523.73  2200.87  1174.00   261.87   1435.87
145.0   421.84  1711.48   523.73  2235.21  1198.04   261.87   1459.90
146.0   424.75  1746.09   523.73  2269.82  1222.26   261.87   1484.13
147.0   427.66  1780.96   523.73  2304.69  1246.67   261.87   1508.53
148.0   430.57  1816.09   523.73  2339.82  1271.26   261.87   1533.13
149.0   433.48  1851.49   523.73  2375.22  1296.04   261.87   1557.91
150.0   436.39  1887.16   523.73  2410.89  1321.01   261.87   1582.88
151.0   439.30  1923.09   523.73  2446.82  1346.16   261.87   1608.03
152.0   442.21  1959.29   523.73  2483.02  1371.50   261.87   1633.37
153.0   445.12  1995.75   523.73  2519.48  1397.03   261.87   1658.89
154.0   448.03  2032.48   523.73  2556.21  1422.73   261.87   1684.60
155.0   450.94  2069.47   523.73  2593.20  1448.63   261.87   1710.50
156.0   453.84  2106.73   523.73  2630.46  1474.71   261.87   1736.58
157.0   456.75  2144.26   523.73  2667.99  1500.98   261.87   1762.85
158.0   459.66  2182.05   523.73  2705.78  1527.43   261.87   1789.30
159.0   462.57  2220.10   523.73  2743.84  1554.07   261.87   1815.94
160.0   465.48  2258.42   523.73  2782.16  1580.90   261.87   1842.76
161.0   468.39  2297.01   523.73  2820.74  1607.91   261.87   1869.77
162.0   471.30  2335.86   523.73  2859.60  1635.10   261.87   1896.97
163.0   474.21  2374.98   523.73  2898.71  1662.49   261.87   1924.35
164.0   477.12  2414.36   523.73  2938.10  1690.06   261.87   1951.92
165.0   480.03  2454.01   523.73  2977.75  1717.81   261.87   1979.68
166.0   482.94  2493.93   523.73  3017.66  1745.75   261.87   2007.62
167.0   485.85  2534.11   523.73  3057.84  1773.88   261.87   2035.74
168.0   488.76  2574.55   523.73  3098.29  1802.19   261.87   2064.05
169.0   491.66  2615.26   523.73  3139.00  1830.69   261.87   2092.55
170.0   494.57  2656.24   523.73  3179.97  1859.37   261.87   2121.23
171.0   497.48  2697.48   523.73  3221.21  1888.24   261.87   2150.10
172.0   500.39  2738.99   523.73  3262.72  1917.29   261.87   2179.16
173.0   503.30  2780.76   523.73  3304.50  1946.53   261.87   2208.40
174.0   506.21  2822.80   523.73  3346.53  1975.96   261.87   2237.83
175.0   509.12  2865.11   523.73  3388.84  2005.57   261.87   2267.44
176.0   512.03  2907.67   523.73  3431.41  2035.37   261.87   2297.24
177.0   514.94  2950.51   523.73  3474.24  2065.36   261.87   2327.22
178.0   517.85  2993.61   523.73  3517.34  2095.53   261.87   2357.39
179.0   520.76  3036.98   523.73  3560.71  2125.88   261.87   2387.75
180.0   523.67  3080.61   523.73  3604.34  2156.42   261.87   2418.29
181.0   526.58  3124.50   523.73  3648.24  2187.15   261.87   2449.02
182.0   529.49  3168.67   523.73  3692.40  2218.07   261.87   2479.93
183.0   532.39  3213.09   523.73  3736.83  2249.17   261.87   2511.03
184.0   535.30  3257.79   523.73  3781.52  2280.45   261.87   2542.32
185.0   538.21  3302.75   523.73  3826.48  2311.92   261.87   2573.79
186.0   541.12  3347.97   523.73  3871.70  2343.58   261.87   2605.45
187.0   544.03  3393.46   523.73  3917.19  2375.42   261.87   2637.29
188.0   546.94  3439.22   542.53  3981.74  2407.45   271.26   2678.71
189.0   549.85  3485.24   562.22  4047.45  2439.67   281.11   2720.77
190.0   552.76  3531.52   582.80  4114.32  2472.07   291.40   2763.47
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191.0   555.67  3578.07   604.28  4182.36  2504.65   302.14   2806.79
192.0   558.58  3624.89   626.66  4251.55  2537.42   313.33   2850.75
193.0   561.49  3671.97   649.93  4321.90  2570.38   324.96   2895.35
194.0   564.40  3719.32   674.09  4393.41  2603.53   337.05   2940.57
195.0   567.31  3766.94   699.15  4466.09  2636.86   349.58   2986.43
196.0   570.21  3814.82   725.10  4539.92  2670.37   362.55   3032.92
197.0   573.12  3862.96   751.95  4614.92  2704.07   375.98   3080.05
198.0   576.03  3911.37   776.12  4687.49  2737.96   388.06   3126.02
199.0   578.94  3960.05   797.60  4757.65  2772.03   398.80   3170.83
200.0   581.85  4008.99   816.39  4825.38  2806.29   408.20   3214.49



 AXIAL LOAD VS SETTLEMENT CURVES
 -------------------------------


    RESULT FROM TREND (AVERAGED) LINE

    TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
       TON             IN.              TON             IN.
   0.1337E+00      0.2156E-04      0.5375E-02      0.1000E-04
   0.6687E+00      0.1078E-03      0.2687E-01      0.5000E-04
   0.1337E+01      0.2156E-03      0.5375E-01      0.1000E-03
   0.6704E+02      0.1079E-01      0.2687E+01      0.5000E-02
   0.1006E+03      0.1619E-01      0.4031E+01      0.7500E-02
   0.1341E+03      0.2159E-01      0.5375E+01      0.1000E-01
   0.3353E+03      0.5398E-01      0.1344E+02      0.2500E-01
   0.6706E+03      0.1080E+00      0.2687E+02      0.5000E-01
   0.1006E+04      0.1620E+00      0.4031E+02      0.7500E-01
   0.1327E+04      0.2151E+00      0.5375E+02      0.1000E+00
   0.2651E+04      0.4837E+00      0.1344E+03      0.2500E+00
   0.3577E+04      0.8200E+00      0.2687E+03      0.5000E+00
   0.4292E+04      0.1590E+01      0.4572E+03      0.1200E+01
   0.4536E+04      0.3419E+01      0.7062E+03      0.3000E+01
   0.4610E+04      0.6428E+01      0.7919E+03      0.6000E+01


    RESULT FROM UPPER-BOUND LINE

    TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
       TON             IN.              TON             IN.
   0.1890E+00      0.2621E-04      0.8028E-02      0.1000E-04
   0.9449E+00      0.1311E-03      0.4014E-01      0.5000E-04
   0.1890E+01      0.2621E-03      0.8028E-01      0.1000E-03
   0.9484E+02      0.1313E-01      0.4014E+01      0.5000E-02
   0.1423E+03      0.1970E-01      0.6021E+01      0.7500E-02
```

```
0.1897E+03      0.2626E-01      0.8028E+01      0.1000E-01
0.4743E+03      0.6567E-01      0.2007E+02      0.2500E-01
0.9486E+03      0.1313E+00      0.4014E+02      0.5000E-01
0.1421E+04      0.1969E+00      0.6021E+02      0.7500E-01
0.1852E+04      0.2600E+00      0.8028E+02      0.1000E+00
0.3477E+04      0.5576E+00      0.2007E+03      0.2500E+00
0.4255E+04      0.8841E+00      0.4014E+03      0.5000E+00
0.4611E+04      0.1623E+01      0.6123E+03      0.1200E+01
0.4780E+04      0.3443E+01      0.7837E+03      0.3000E+01
0.4808E+04      0.6446E+01      0.8123E+03      0.6000E+01


 RESULT FROM LOWER-BOUND LINE

 TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
    TON            IN.              TON             IN.
0.8292E-01      0.1722E-04      0.2721E-02      0.1000E-04
0.4146E+00      0.8610E-04      0.1361E-01      0.5000E-04
0.8292E+00      0.1722E-03      0.2721E-01      0.1000E-03
0.4152E+02      0.8614E-02      0.1361E+01      0.5000E-02
0.6229E+02      0.1292E-01      0.2041E+01      0.7500E-02
0.8306E+02      0.1723E-01      0.2721E+01      0.1000E-01
0.2077E+03      0.4308E-01      0.6803E+01      0.2500E-01
0.4154E+03      0.8616E-01      0.1361E+02      0.5000E-01
0.6231E+03      0.1292E+00      0.2041E+02      0.7500E-01
0.8294E+03      0.1723E+00      0.2721E+02      0.1000E+00
0.1816E+04      0.4093E+00      0.6803E+02      0.2500E+00
0.2878E+04      0.7543E+00      0.1361E+03      0.5000E+00
0.3971E+04      0.1557E+01      0.3021E+03      0.1200E+01
0.4293E+04      0.3396E+01      0.6286E+03      0.3000E+01
0.4408E+04      0.6410E+01      0.7674E+03      0.6000E+01
```

Piers
Seismic Axial Capacity
10-ft CIDH with 11-ft Casing

```
=========================================================================

                 SHAFT for Windows, Version 2017.8.4

                  Serial Number :  253582343

               VERTICALLY LOADED DRILLED SHAFT ANALYSIS
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                       All Rights Reserved

=========================================================================


Path to file locations      : C:\Users\JBonfiglio\OneDrive -
Kleinfelder\Desktop\Camino Del Mar Desktop\Pile Capacity\SHAFT Final Rev2\
Name of input data file     : Piers_10ft_Seismic_rev2.sf8d
Name of output file         : Piers_10ft_Seismic_rev2.sf8o
Name of plot output file    : Piers_10ft_Seismic_rev2.sf8p
Name of runtime file        : Piers_10ft_Seismic_rev2.sf8r


-------------------------------------------------------------------------
                     Time and Date of Analysis
-------------------------------------------------------------------------

          Date:  May 18, 2020     Time:  11:57:48

Camino Del Mar Bridge Replacement - Phase 0


PROPOSED DEPTH =     200.0 FT
----------------


NUMBER OF LAYERS =    8
------------------


WATER TABLE DEPTH =       0.0 FT.
-------------------




SOIL INFORMATION
---------------

LAYER NO 1----CLAY
```

```
  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00
  END BEARING COEFFICIENT-Nc                         = 0.600E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.000E+00


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00
  END BEARING COEFFICIENT-Nc                         = 0.744E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.120E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)             = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)            = 0.100E+01


LAYER NO 2----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00
  END BEARING COEFFICIENT-Nc                         = 0.744E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.120E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00
  END BEARING COEFFICIENT-Nc                         = 0.828E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.110E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
```

```
  DEPTH, FT                                         = 0.190E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 3----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.828E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.190E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.450E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.120E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.350E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E+01


LAYER NO 4----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.350E+02
```

```
  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                     = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                          = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                  = 0.700E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.650E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)              = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)             = 0.100E+01


LAYER NO 5----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                     = 0.550E+00
  END BEARING COEFFICIENT-Nc                          = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                  = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.650E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                     = 0.550E+00
  END BEARING COEFFICIENT-Nc                          = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                  = 0.750E+03
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.115E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.820E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)              = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)             = 0.100E+01


LAYER NO 6----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
```

```
  SKIN FRICTION COEFFICIENT- BETA                      = 0.280E+00
  INTERNAL FRICTION ANGLE, DEG.                        = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.820E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                        = 0.340E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.122E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)              = 0.100E+01


LAYER NO 7----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                        = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.122E+03

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                      = 0.250E+00
  INTERNAL FRICTION ANGLE, DEG.                        = 0.360E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST        = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                           = 0.130E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT             = 0.100E+11
  DEPTH, FT                                            = 0.207E+03

  LRFD RESISTANCE FACTOR (SIDE FRICTION)               = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)              = 0.100E+01


LAYER NO 8----CLAY
```

```
  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                          = 0.510E+00
  END BEARING COEFFICIENT-Nc                               = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                       = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST            = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                               = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT                 = 0.100E+11
  DEPTH, FT                                                = 0.207E+03


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                          = 0.510E+00
  END BEARING COEFFICIENT-Nc                               = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                       = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST            = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                               = 0.135E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT                 = 0.100E+11
  DEPTH, FT                                                = 0.255E+03


  LRFD RESISTANCE FACTOR (SIDE FRICTION)                   = 0.100E+01
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)                  = 0.100E+01


(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =   10.000  FT.
 MAXIMUM SHAFT DIAMETER    =   10.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
```

```
COMPUTATION RESULTS
-------------------



- CASE ANALYZED      :     1
  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1



DRILLED SHAFT INFORMATION

-------------------------

 DIAMETER OF STEM          =   10.000  FT.
 DIAMETER OF BASE          =   10.000  FT.
 END OF STEM TO BASE       =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =   82.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 AREA OF ONE PERCENT STEEL =  113.112  SQ.IN.
 ELASTIC MODULUS, Ec       = 0.360E+07 LB/SQ IN
 VOLUME OF UNDERREAM       =    0.000  CU.YDS.
 SHAFT LENGTH              =  200.000  FT.


PREDICTED RESULTS
-----------------

QS      = ULTIMATE SIDE RESISTANCE;
QB      = ULTIMATE BASE RESISTANCE;
WT      = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
QU      = TOTAL ULTIMATE RESISTANCE;
LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE SIDE RESISTANCE;
LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR
          TO THE ULTIMATE BASE RESISTANCE
LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

LENGTH  VOLUME     QS      QB      QU     LRFD QS  LRFD QB   LRFD QU
 (FT)   (CU.YDS) (TONS)   (TONS)  (TONS)   (TONS)   (TONS)    (TONS)
83.0   241.47    20.84   332.14   352.98    20.84   332.14    352.98
84.0   244.38    41.89   334.83   376.72    41.89   334.83    376.72
85.0   247.29    63.16   337.29   400.45    63.16   337.29    400.45
86.0   250.20    84.65   339.52   424.17    84.65   339.52    424.17
87.0   253.11   106.35   341.51   447.85   106.35   341.51    447.85
88.0   256.01   128.26   343.25   471.50   128.26   343.25    471.50
```

```
  89.0   258.92   150.38   344.72   495.10   150.38   344.72    495.10
  90.0   261.83   172.71   345.93   518.64   172.71   345.93    518.64
  91.0   264.74   195.25   346.90   542.15   195.25   346.90    542.15
  92.0   267.65   217.99   347.65   565.64   217.99   347.65    565.64
  93.0   270.56   240.94   348.21   589.16   240.94   348.21    589.16
  94.0   273.47   264.09   348.62   612.71   264.09   348.62    612.71
  95.0   276.38   287.45   348.89   636.33   287.45   348.89    636.33
  96.0   279.29   311.00   349.05   660.05   311.00   349.05    660.05
  97.0   282.20   334.75   349.13   683.88   334.75   349.13    683.88
  98.0   285.11   358.70   349.15   707.86   358.70   349.15    707.86
  99.0   288.02   382.85   349.15   732.00   382.85   349.15    732.00
 100.0   290.93   407.19   349.15   756.34   407.19   349.15    756.34
 101.0   293.84   431.72   349.15   780.88   431.72   349.15    780.88
 102.0   296.74   456.45   349.15   805.60   456.45   349.15    805.60
 103.0   299.65   481.36   358.56   839.92   481.36   358.56    839.92
 104.0   302.56   506.47   368.40   874.87   506.47   368.40    874.87
 105.0   305.47   531.76   378.70   910.45   531.76   378.70    910.45
 106.0   308.38   557.23   389.44   946.67   557.23   389.44    946.67
 107.0   311.29   582.89   400.63   983.52   582.89   400.63    983.52
 108.0   314.20   608.73   412.27  1021.00   608.73   412.27   1021.00
 109.0   317.11   634.75   424.36  1059.11   634.75   424.36   1059.11
 110.0   320.02   660.96   436.89  1097.85   660.96   436.89   1097.85
 111.0   322.93   687.34   449.87  1137.21   687.34   449.87   1137.21
 112.0   325.84   713.89   463.30  1177.20   713.89   463.30   1177.20
 113.0   328.75   740.63   475.39  1216.01   740.63   475.39   1216.01
 114.0   331.66   767.53   486.13  1253.66   767.53   486.13   1253.66
 115.0   334.56   794.61   495.53  1290.14   794.61   495.53   1290.14
 116.0   337.47   821.86   503.59  1325.45   821.86   503.59   1325.45
 117.0   340.38   849.28   510.30  1359.58   849.28   510.30   1359.58
 118.0   343.29   876.86   515.67  1392.54   876.86   515.67   1392.54
 119.0   346.20   904.62   519.70  1424.32   904.62   519.70   1424.32
 120.0   349.11   932.53   522.39  1454.92   932.53   522.39   1454.92
 121.0   352.02   960.61   523.73  1484.35   960.61   523.73   1484.35
 122.0   354.93   988.86   523.73  1512.59   988.86   523.73   1512.59
 123.0   357.84  1017.35   523.73  1541.09  1017.35   523.73   1541.09
 124.0   360.75  1046.12   523.73  1569.85  1046.12   523.73   1569.85
 125.0   363.66  1075.15   523.73  1598.88  1075.15   523.73   1598.88
 126.0   366.57  1104.44   523.73  1628.17  1104.44   523.73   1628.17
 127.0   369.48  1134.00   523.73  1657.73  1134.00   523.73   1657.73
 128.0   372.39  1163.83   523.73  1687.56  1163.83   523.73   1687.56
 129.0   375.29  1193.92   523.73  1717.65  1193.92   523.73   1717.65
 130.0   378.20  1224.27   523.73  1748.01  1224.27   523.73   1748.01
 131.0   381.11  1254.90   523.73  1778.63  1254.90   523.73   1778.63
 132.0   384.02  1285.78   523.73  1809.52  1285.78   523.73   1809.52
 133.0   386.93  1316.94   523.73  1840.67  1316.94   523.73   1840.67
 134.0   389.84  1348.35   523.73  1872.09  1348.35   523.73   1872.09
 135.0   392.75  1380.04   523.73  1903.77  1380.04   523.73   1903.77
 136.0   395.66  1411.99   523.73  1935.72  1411.99   523.73   1935.72
```

```
137.0   398.57  1444.20   523.73  1967.94  1444.20   523.73   1967.94
138.0   401.48  1476.68   523.73  2000.42  1476.68   523.73   2000.42
139.0   404.39  1509.43   523.73  2033.16  1509.43   523.73   2033.16
140.0   407.30  1542.44   523.73  2066.17  1542.44   523.73   2066.17
141.0   410.21  1575.72   523.73  2099.45  1575.72   523.73   2099.45
142.0   413.11  1609.26   523.73  2132.99  1609.26   523.73   2132.99
143.0   416.02  1643.07   523.73  2166.80  1643.07   523.73   2166.80
144.0   418.93  1677.14   523.73  2200.87  1677.14   523.73   2200.87
145.0   421.84  1711.48   523.73  2235.21  1711.48   523.73   2235.21
146.0   424.75  1746.09   523.73  2269.82  1746.09   523.73   2269.82
147.0   427.66  1780.96   523.73  2304.69  1780.96   523.73   2304.69
148.0   430.57  1816.09   523.73  2339.82  1816.09   523.73   2339.82
149.0   433.48  1851.49   523.73  2375.22  1851.49   523.73   2375.22
150.0   436.39  1887.16   523.73  2410.89  1887.16   523.73   2410.89
151.0   439.30  1923.09   523.73  2446.82  1923.09   523.73   2446.82
152.0   442.21  1959.29   523.73  2483.02  1959.29   523.73   2483.02
153.0   445.12  1995.75   523.73  2519.48  1995.75   523.73   2519.48
154.0   448.03  2032.48   523.73  2556.21  2032.48   523.73   2556.21
155.0   450.94  2069.47   523.73  2593.20  2069.47   523.73   2593.20
156.0   453.84  2106.73   523.73  2630.46  2106.73   523.73   2630.46
157.0   456.75  2144.26   523.73  2667.99  2144.26   523.73   2667.99
158.0   459.66  2182.05   523.73  2705.78  2182.05   523.73   2705.78
159.0   462.57  2220.10   523.73  2743.84  2220.10   523.73   2743.84
160.0   465.48  2258.42   523.73  2782.16  2258.42   523.73   2782.16
161.0   468.39  2297.01   523.73  2820.74  2297.01   523.73   2820.74
162.0   471.30  2335.86   523.73  2859.60  2335.86   523.73   2859.60
163.0   474.21  2374.98   523.73  2898.71  2374.98   523.73   2898.71
164.0   477.12  2414.36   523.73  2938.10  2414.36   523.73   2938.10
165.0   480.03  2454.01   523.73  2977.75  2454.01   523.73   2977.75
166.0   482.94  2493.93   523.73  3017.66  2493.93   523.73   3017.66
167.0   485.85  2534.11   523.73  3057.84  2534.11   523.73   3057.84
168.0   488.76  2574.55   523.73  3098.29  2574.55   523.73   3098.29
169.0   491.66  2615.26   523.73  3139.00  2615.26   523.73   3139.00
170.0   494.57  2656.24   523.73  3179.97  2656.24   523.73   3179.97
171.0   497.48  2697.48   523.73  3221.21  2697.48   523.73   3221.21
172.0   500.39  2738.99   523.73  3262.72  2738.99   523.73   3262.72
173.0   503.30  2780.76   523.73  3304.50  2780.76   523.73   3304.50
174.0   506.21  2822.80   523.73  3346.53  2822.80   523.73   3346.53
175.0   509.12  2865.11   523.73  3388.84  2865.11   523.73   3388.84
176.0   512.03  2907.67   523.73  3431.41  2907.67   523.73   3431.41
177.0   514.94  2950.51   523.73  3474.24  2950.51   523.73   3474.24
178.0   517.85  2993.61   523.73  3517.34  2993.61   523.73   3517.34
179.0   520.76  3036.98   523.73  3560.71  3036.98   523.73   3560.71
180.0   523.67  3080.61   523.73  3604.34  3080.61   523.73   3604.34
181.0   526.58  3124.50   523.73  3648.24  3124.50   523.73   3648.24
182.0   529.49  3168.67   523.73  3692.40  3168.67   523.73   3692.40
183.0   532.39  3213.09   523.73  3736.83  3213.09   523.73   3736.83
184.0   535.30  3257.79   523.73  3781.52  3257.79   523.73   3781.52
```

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185.0   538.21  3302.75   523.73  3826.48  3302.75   523.73   3826.48
186.0   541.12  3347.97   523.73  3871.70  3347.97   523.73   3871.70
187.0   544.03  3393.46   523.73  3917.19  3393.46   523.73   3917.19
188.0   546.94  3439.22   542.53  3981.74  3439.22   542.53   3981.74
189.0   549.85  3485.24   562.22  4047.45  3485.24   562.22   4047.45
190.0   552.76  3531.52   582.80  4114.32  3531.52   582.80   4114.32
191.0   555.67  3578.07   604.28  4182.36  3578.07   604.28   4182.36
192.0   558.58  3624.89   626.66  4251.55  3624.89   626.66   4251.55
193.0   561.49  3671.97   649.93  4321.90  3671.97   649.93   4321.90
194.0   564.40  3719.32   674.09  4393.41  3719.32   674.09   4393.41
195.0   567.31  3766.94   699.15  4466.09  3766.94   699.15   4466.09
196.0   570.21  3814.82   725.10  4539.92  3814.82   725.10   4539.92
197.0   573.12  3862.96   751.95  4614.92  3862.96   751.95   4614.92
198.0   576.03  3911.37   776.12  4687.49  3911.37   776.12   4687.49
199.0   578.94  3960.05   797.60  4757.65  3960.05   797.60   4757.65
200.0   581.85  4008.99   816.39  4825.38  4008.99   816.39   4825.38



 AXIAL LOAD VS SETTLEMENT CURVES
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    RESULT FROM TREND (AVERAGED) LINE

    TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
       TON             IN.              TON             IN.
   0.1337E+00      0.2156E-04      0.5375E-02      0.1000E-04
   0.6687E+00      0.1078E-03      0.2687E-01      0.5000E-04
   0.1337E+01      0.2156E-03      0.5375E-01      0.1000E-03
   0.6704E+02      0.1079E-01      0.2687E+01      0.5000E-02
   0.1006E+03      0.1619E-01      0.4031E+01      0.7500E-02
   0.1341E+03      0.2159E-01      0.5375E+01      0.1000E-01
   0.3353E+03      0.5398E-01      0.1344E+02      0.2500E-01
   0.6706E+03      0.1080E+00      0.2687E+02      0.5000E-01
   0.1006E+04      0.1620E+00      0.4031E+02      0.7500E-01
   0.1327E+04      0.2151E+00      0.5375E+02      0.1000E+00
   0.2651E+04      0.4837E+00      0.1344E+03      0.2500E+00
   0.3577E+04      0.8200E+00      0.2687E+03      0.5000E+00
   0.4292E+04      0.1590E+01      0.4572E+03      0.1200E+01
   0.4536E+04      0.3419E+01      0.7062E+03      0.3000E+01
   0.4610E+04      0.6428E+01      0.7919E+03      0.6000E+01


    RESULT FROM UPPER-BOUND LINE

    TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
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     TON              IN.              TON            IN.
 0.1890E+00      0.2621E-04      0.8028E-02      0.1000E-04
 0.9449E+00      0.1311E-03      0.4014E-01      0.5000E-04
 0.1890E+01      0.2621E-03      0.8028E-01      0.1000E-03
 0.9484E+02      0.1313E-01      0.4014E+01      0.5000E-02
 0.1423E+03      0.1970E-01      0.6021E+01      0.7500E-02
 0.1897E+03      0.2626E-01      0.8028E+01      0.1000E-01
 0.4743E+03      0.6567E-01      0.2007E+02      0.2500E-01
 0.9486E+03      0.1313E+00      0.4014E+02      0.5000E-01
 0.1421E+04      0.1969E+00      0.6021E+02      0.7500E-01
 0.1852E+04      0.2600E+00      0.8028E+02      0.1000E+00
 0.3477E+04      0.5576E+00      0.2007E+03      0.2500E+00
 0.4255E+04      0.8841E+00      0.4014E+03      0.5000E+00
 0.4611E+04      0.1623E+01      0.6123E+03      0.1200E+01
 0.4780E+04      0.3443E+01      0.7837E+03      0.3000E+01
 0.4808E+04      0.6446E+01      0.8123E+03      0.6000E+01


  RESULT FROM LOWER-BOUND LINE

  TOP  LOAD     TOP MOVEMENT       TIP LOAD      TIP  MOVEMENT
     TON              IN.              TON            IN.
 0.8292E-01      0.1722E-04      0.2721E-02      0.1000E-04
 0.4146E+00      0.8610E-04      0.1361E-01      0.5000E-04
 0.8292E+00      0.1722E-03      0.2721E-01      0.1000E-03
 0.4152E+02      0.8614E-02      0.1361E+01      0.5000E-02
 0.6229E+02      0.1292E-01      0.2041E+01      0.7500E-02
 0.8306E+02      0.1723E-01      0.2721E+01      0.1000E-01
 0.2077E+03      0.4308E-01      0.6803E+01      0.2500E-01
 0.4154E+03      0.8616E-01      0.1361E+02      0.5000E-01
 0.6231E+03      0.1292E+00      0.2041E+02      0.7500E-01
 0.8294E+03      0.1723E+00      0.2721E+02      0.1000E+00
 0.1816E+04      0.4093E+00      0.6803E+02      0.2500E+00
 0.2878E+04      0.7543E+00      0.1361E+03      0.5000E+00
 0.3971E+04      0.1557E+01      0.3021E+03      0.1200E+01
 0.4293E+04      0.3396E+01      0.6286E+03      0.3000E+01
 0.4408E+04      0.6410E+01      0.7674E+03      0.6000E+01
```

APPENDIX H

GEOTECHNICAL BUSINESS COUNCIL INSERT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

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