KLEINFELDER
Bright People. Right Solutions.

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 Geotechnical Design Report
Camino Del Mar Bridge Replacement
Over San Dieguito River
Del Mar, California

Dear Mr. Thiele:

Kleinfelder is pleased to present this Preliminary Geotechnical Design Report (PGDR) for the Camino Del Mar Bridge Replacement project over San Dieguito River in Del Mar, California. This report presents the results of our geotechnical engineering investigation and provides preliminary geotechnical recommendations for the proposed replacement bridge project. This report is presented in conjunction with the Preliminary Foundation Report (PFR) 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 6

PRELIMINARY GEOTECHNICAL DESIGN 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
Engineering Manager for City of Del Mar
Michael Baker International
1050 Camino Del Mar
Del Mar, California 92014

PRELIMINARY GEOTECHNICAL DESIGN REPORT
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER
DEL MAR, CALIFORNIA

Prepared by:

Janna Bonfiglio, PE 89334
Project Engineer

Scot Rugg, CEG 1651
Senior Engineering Geologist

Reviewed by:

Karthik Radhakrishnan, GE 3046
Senior Program Manager

KLEINFELDER
550 West C Street, Suite 1200
San Diego, CA 92101
Phone: 619.831.4600
Fax: 619.232.1039

June 19, 2020
Kleinfelder Project No. 20180876.001A

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

FIGURES

APPENDICES

TABLES

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. This Preliminary Geotechnical Design Report (PGDR) was prepared to provide preliminary conclusions and recommendations for the type selection phase of the proposed project located along Camino Del Mar and over the San Dieguito River in Del Mar, California. This PGDR was prepared in accordance with Caltrans' Geotechnical Design Report Guidelines, dated January 2020, and covers the geotechnical aspects of the project outside of the bridge structure footprint. The PGDR is a companion report to a separately provided Preliminary Foundation Report (PFR) for the proposed project. The preliminary geotechnical recommendations for the bridge structure are provided in the PFR.

The purpose of this PGDR is to present the results of our Phase 0 geotechnical field investigation, evaluate the subsurface conditions at the site, determine potential geologic and seismic hazards, perform geotechnical engineering evaluations, and provide preliminary recommendations for the geotechnical aspects of the proposed project outside of the bridge structure footprint (approximately 150 feet away from the bridge limits).

This PGDR is not intended for final design of the project. Additional investigations and analyses will be required as recommended in Section 6 of this report for final design.

1.1 SCOPE OF WORK

The purposes of our geotechnical engineering services were to evaluate the soil and geologic conditions at the site and provide preliminary conclusions and recommendations for design and construction of the proposed improvements. 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 and approval 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;
- Preparation of this PGDR 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 on 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 and seismic 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 recommendations for embankment fill stability and settlement;
    - Pavement section recommendations;
    - Discussion of soil corrosivity properties affecting below-grade concrete and steel; and
    - Recommendations for further field investigations.
- Preparation of a Preliminary Foundation Report (PFR), inclusive of foundation design and construction recommendations, which is provided under a separate cover.

The recommendations contained within this report are subject to the limitations presented in Section 7.0 and are in conjunction with the PFR 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 approximate 596-foot-long 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 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 typical current tide elevations range from approximate elevation +0 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 south 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 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 existing abutments and piers are supported on timber piles. Outside of the 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 portion 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. Some of these abandoned timber piles can currently be observed to the west of the site.

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 are presented on Figure 2.

1.4 PROJECT DESCRIPTION

Based on 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 three 5-span and 6-span cast-in-place box girder bridge options as well as two 6-span precast concrete girder bridge options. The proposed bridge structure 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.

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 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 walls have 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. However, only the first approximate 80 to 100 feet of the approach fills from the abutments are proposed to be retained by retaining walls on both sides. Therefore, retaining walls are addressed as part of the PFR.

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. Temporary cuts are anticipated to be required outside of the bridge footprint for re-alignment of existing utilities at the site as well for remedial grading.

1.5 EXCEPTIONS TO POLICY

No exceptions to policy were taken for the preparation of this PGDR.

2 GEOTECHNICAL INVESTIGATION

Our preliminary geotechnical investigation (Phase 0 investigation) consisted of advancing two exploratory borings and three cone penetrometer tests (CPTs). 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 Camino Del Mar bridge. Laboratory testing and review of existing geotechnical and geologic information were also performed for our geotechnical investigation. 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

2.1.1 Previous Geotechnical Reports for Camino Del Mar Bridge

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 extending to reported depths of approximately 81½ feet and 155 feet below ground surface (bgs), respectively. Boring B-8 and CPT-12 were performed near the existing southern bridge abutment and extended to reported depths of approximately 95 feet and 196 feet bgs, respectively.

In general, Ninyo & Moore reported undocumented fill material overlying successive strata of alluvium and the Del Mar Formation. The fill reportedly extended to depths of up to approximately 12 to 14 feet bgs and generally consisted of brown and light gray, very loose to medium dense silty sand with trace amounts of shells, gravel, and asphalt fragments.

Alluvium consisting 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 was reported below the fill in all of the Ninyo & Moore borings and CPTs. The alluvium extended to the termination depth of the borings (up to 95 feet bgs) and to the termination depth of CPT-12 (approximately 195 feet bgs) located at the southern end of the bridge. Ninyo & Moore stated 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 CPT-11 located at the northern end of the bridge. The contact was shown at approximately 145 feet bgs on the cross-section. 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 presented in Section 3.2 of this report, we anticipate that the Del Mar Formation was encountered at approximately 145 feet bgs at the CPT-11 location as shown on the Ninyo & Moore cross-section.

Groundwater was encountered in the Ninyo & Moore borings at depths of approximately 12½ feet and 14 feet bgs, or at approximate elevations +1½ feet and +3 feet, and surface water was observed within the San Dieguito River.

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, exploration plan, geologic cross section, and laboratory test results, along with the Division of Highway LOTBs, are provided in Appendix E.

2.1.2 Nearby Geotechnical Reports and LOTBs

Previous geotechnical reports and other available geotechnical information for projects located in the site vicinity were also reviewed and include the following:

- “San Dieguito River Bridge Replacement, Double Track and Del Mar Fairgrounds Special Events Platform (Milepost 242 to Milepost 244) 90% Design, Draft Foundation Report, Bridge 243.0,” prepared by Leighton Consulting, Inc., dated May 5, 2017.
- “Foundation Investigation for Jimmy Durante Bridge, Del Mar, California,” prepared by Robert Prater Associates, dated May 1980 (as available online on GeoDOG).
- Various Foundation Reports, LOTBs, and geologic and seismic letters regarding various stages of construction and widening of the I-5 bridge over San Dieguito River spanning from 1962 to 2004, as available online on GeoDOG.

These available geotechnical documents provide further information regarding the geologic conditions in the site vicinity. These reports were completed by others for the bridges spanning across the San Dieguito River up channel from our site and include the heavy rail bridge located 550 feet to the east of the site, the Jimmy Durante Blvd bridge located 2,500 feet to the east of the site, and the I-5 bridge located 5,700 feet to the east.

The quantity and quality of information provided in the previous reports varies. The Leighton Consulting investigation performed for the heavy rail bridge located just east of the site reports that a relatively thin layer of young flood plain deposits overlies deep alluvium in the borings and CPTs performed near the river crossing. The borings performed near the river crossing terminated in the alluvium at a depth of 76½ feet bgs. The CPTs performed near the crossing reportedly refused on the Del Mar Formation at depths ranging from 140½ feet to 222 feet bgs.

The Robert Prater Associates foundation investigation report reviewed for the Jimmy Durante Boulevard bridge replacement reports the results of three exploratory borings, two performed at the ends of the existing bridge at the time of the report, and one performed within the middle of the bridge over the San Dieguito River. The boring logs provided in this report indicate artificial fill materials extending to depths of 12 and 13 feet at the ends of the bridge overlying natural soils consisting of loose to very dense sandy silt, silty sand, and poorly-graded sand. Boring EB-1, performed at the northern end of the bridge, terminated in these natural overburden soils at a depth of 53 feet bgs. These sandy materials were reportedly underlain by formational claystone at a depth of approximately 41 feet in Boring EB-2, performed at the southern end of the Jimmy Durante bridge, and formational sandstone at a depth of approximately 48 feet in boring EB-3, performed in the center of the bridge.

The LOTBs for the I-5 bridge widening project in 1991 near the San Dieguito River crossing reports deep estuary deposits overlying the Del Mar Formation at a depth of approximately 140 feet bgs.

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. Prior to drilling the borings, a public utility mark-out was performed and nearby utilities were located within the City of Del Mar right-of-way (ROW) using geophysical surveys performed by Southwest Geophysics. Additionally, the surface pavement was cored by Cut N Core and the first 5 to 6 feet of each borehole were advanced by manual hand auger to further clear for underground utilities.

A field engineer from our office logged the subsurface conditions encountered in the boreholes and collected soil samples for further evaluation and laboratory testing. Selected bulk and relatively intact samples were retrieved from the boreholes at selected sampling depths, sealed, and transported to our laboratory for further evaluation. The intact samples were retrieved using either a Standard Penetration Test (SPT) split-spoon sampler or a California sampler. The number of blows necessary to drive the samplers 18 inches, using a 140-pound automatic hammer dropped from a height of 30 inches, were recorded by our field engineer. Graphic notations on the borehole logs indicate which sampler type was utilized at each sampling location.

Upon completion, the boreholes were backfilled with bentonite and patched at the surface with asphalt concrete (AC). Soil cuttings were stored in 55-gallon steel drums and, upon completion of laboratory waste characterization testing, were disposed of offsite.

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 described further in Section 3.2 of this report.

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 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 were advanced to depths of approximately 158 feet and 200 feet, or to approximate elevations of -142 feet and -184 feet, respectively. 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. 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.

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 American Society of Testing and Materials (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, soil survey results, 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 (Qop6) 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 existing 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 abutment. 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 CPT-20-002A, CPT-20-003, and boring R-20-002. This geologic unit was recently loosely deposited by the flow of the San

Dieguito River as evidenced by 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-20-002A, 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 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 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 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 elevations 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 (Td)

The Del Mar Formation is an Eocene-age geologic unit deposited in an ancient lagoonal environment. This formation 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 to a depth of 5 feet and was observed in one sample to consist of dark reddish brown with grayish green claystone. This unit is known to 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 GEOLOGIC HAZARDS AND UNSUITABLE MATERIALS

3.3.1 Landslides

Landslides are deep-seated ground failures (several tens to hundreds of feet deep) in which a large arcuate or block shaped section of a slope detaches and slides downhill. Landslides can cause damage to structures both above and below the slide mass. Several formations within the San Diego region are particularly prone to landslides. These formations generally have high clay content and mobilize when they become saturated. Other factors, such as steeply-dipping bedding that project out of the face of the slope and/or the presence of fracture planes, will also increase the potential for a landslide.

The nearest substantive slope to the site is located approximately 400 feet to the north. This slope is part of the coastal bluff and is comprised of the Del Mar Formation. The Del Mar Formation is known for instability in steep slopes. However, due to the distance to the project site from these slopes and the relatively flat-lying site topography outside of the bridge footprint, it is our opinion that the hazard with respect to a landslide impact at the site is low.

3.3.2 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 range 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 draft 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 EROSION AND SCOUR

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 along the existing embankment slopes to the north of the bridge outside of the proposed bridge footprint. We understand that the existing rip-rap slope protection will be maintained to protect these slopes from surficial erosion from high flow events.

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, 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 Fault 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 based on 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 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 for design of the project structures in accordance with Caltrans standards.

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 the 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 discussion, 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' Geotechnical Design Report Guidelines (Caltrans, 2020) and cover the preliminary geotechnical recommendations pertinent to the project components located greater than approximately 150 feet beyond the extents of the bridge structure. Preliminary foundations recommendations, as well as other discussions and recommendations pertaining to the geotechnical aspects of the bridge structure and associated approach retaining walls located within 150 feet from the bridge limits are provided in the PFR.

5.1 EMBANKMENT FILLS

Based on the project conceptual design plans and discussions with the project team, we understand that several approach fill profiles are currently under consideration at this phase of the project. Minimal grading is expected beyond 150 feet away from the bridge limits with proposed approach fill heights of less than 5 feet.

5.1.1 Embankment Settlement

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, static settlements are anticipated to occur relatively quickly after construction activities with the majority of the elastic settlement occurring during placement and compaction of fills. Once the final grading profiles have been established, bridge approach settlements due to placement of new approach fills should be evaluated.

Liquefaction-induced settlements between 3 to 7 inches during a seismic event have been estimated along the approaches based on the results of the field investigation 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-earthquakes inspection and repair.

5.1.2 Embankment Global Stability

Based on discussions with the project team and the conceptual plans, we understand that various grade profiles 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 approach embankments outside of the bridge limits will be considered stable with the use of Caltrans Standard fill. However, analyses should be performed to confirm the global stability once the final roadway profile, slope grades, and fill thicknesses have been determined.

Approach fill heights are expected to be the highest at the abutments representing the critical section for global stability analysis. We have performed limit equilibrium slope stability analyses for the bridge abutment walls/slopes which are included in the PFR. These slope stability analyses evaluate the critical stability sections of the wall/approach fill system.

5.2 PAVEMENT

Based on conversations with the project team and review of preliminary plans, we understand the existing roadway pavement within approximately 300 feet of the Camino Del Mar Bridge will be demolished and replaced to facilitate proposed earthwork. We anticipate that the pavement for the approach embankments will consist of asphalt concrete (AC) pavement consistent with the current site conditions and a concrete approach slab is anticipated within approximately 30 feet approaching the bridge deck. Recommendations for the new AC pavement are provided herein.

We understand that the Camino Del Mar Replacement Bridge will primarily be used for vehicular, bicycle, and pedestrian traffic, with occasional truck traffic, although detailed vehicular load and frequency information was not provided.

Two resistance value (R-value) tests were performed on selected bulk samples of the near-surface soils from borings R-20-001 and R-20-002. The R-value test results are given in Appendix C and resulted in R-values of 51 and 63.

The recommended pavement sections provided herein are based on Caltrans Highway Design Manual and the following conditions:

1. A Minimum of 12 inches of existing subgrade soils should be overexcavated and replaced with new, compacted engineered fill. The new, compacted engineered fill should be placed at an optimum moisture content between optimum and 3 percent above optimum at a minimum relative compaction of 95 percent per ASTM D1557.
2. Utility trench backfill should be properly placed and adequately compacted to provide a stable subgrade. Trench backfill within the top 12 inches of pavement soil subgrade should be compacted to a minimum of 95 percent relative compaction (ASTM D1557).

3. An adequate drainage system should be provided to prevent surface water from saturating the subgrade soil. Pavements should be sloped to provide positive drainage and water should not be allowed to pond.
4. A periodic maintenance program should be incorporated to include sealing cracks and other measures.
5. Aggregate base materials and the upper 12 inches of subgrade soil below the aggregate base should be compacted to a minimum of 95 percent of the ASTM D1557 maximum dry density.
6. The finished subgrade should be at, or brought to, a firm and unyielding condition at the time aggregate base is laid and compacted.
7. Concrete curbs separating pavement from landscaped areas extend below the bottom of adjacent aggregate base materials to reduce movement of moisture into the aggregate base layer.
8. Pavement subgrades are placed and compacted according to the project specifications.

5.2.1 Flexible Pavements

Based on our experience with similar projects, we anticipate traffic indices of 5.0 to 7.0 may be anticipated for the project. Based on the R-value test results, potential variability along the approximate 300-foot-long fill sections of the bridge approaches, and the recommendation for pavements to be supported on a minimum of 12 inches of new, engineered fill, we recommend an R-value of 30 may be used for preliminary pavement design. Final pavement sections may be adjusted based on testing of actual subgrade soils during construction.

Preliminary recommended flexible pavement sections using an R-value of 30 have been evaluated in accordance with Caltrans Standards and are provided in Table 1.

Table 1

Recommended Flexible Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5	3	5½
	4	4
6	4	6½
	5	4½

Table 1 (Continued)

Recommended Flexible Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
7	4½	8½
	5	7½

The flexible pavement and aggregate base materials should conform to and be placed in accordance with current Caltrans Standard Specifications. The aggregate base should be compacted to a minimum of 95 percent of the maximum dry density per ASTM D1557.

The above recommendations are contingent on supporting the pavements on a minimum of 12 inches of new, engineered fill. The upper 12 inches of existing fill material encountered within pavement subgrades should be removed. The aggregate base can be placed directly on the pavement subgrades provided it has been compacted to 95 percent of the ASTM D1557 maximum dry density at moisture contents of 0 to 3 percent above optimum.

5.3 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 2 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 2 as well as in Appendix E.

Table 2

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.

6 ADDITIONAL INVESTIGATION AND DESIGN EVALUATION

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, and any other improvements, additional explorations may or may not 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 embankment stability, settlement analyses, 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 geotechnical design report including recommendations for pavement design and earthwork.

7 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.

8 REFERENCES

Abbott, P.L., 1999, The Rise and Fall of San Diego: 150 Million Years of History Recorded in Sedimentary Rocks, Sunbelt Publications, San Diego.

American Association of State Highway and Transportation Officials (AASHTO), 2017, AASHTO LRFD Bridge Design Specifications, 8th Edition, September 2017.

American Concrete Institute (ACI), 2019, Building Code Requirements for Structural Concrete (ACI 318-19), 2019.

American Society of Testing and Materials (ASTM), various standards.

Bennett, R.A., Relinger, R.E., and Rodi, W., 1996, "Global Positioning System Constraints on Fault Slip Rates in Southern California and Northern Baja, Mexico", Journal of Geophysical Research, Vol. 101, 1996.

Boulanger, R. W. and Idriss, I. M., 2006, "Liquefaction Susceptibility Criteria for Silts and Clays," Journal of Geotechnical and Geo-environmental Engineering, ASCE, Vol. 132, No. 11.

California Department of Public Works, Division of Highways, 1960, "Progress Report of Foundation Investigation on Road XI-SD-2-SD,A, San Dieguito River Basin, Station 1216 to Station 1280," May 25, 1960, available on GeoDOG at https://geodog.dot.ca.gov/.

California Department of Public Works, Division of Highways, 1960, "Supplemental Report of Foundation Investigation on Road XI-SD-2-SD,A, San Dieguito River Basin, Station 1216 to Station 1280," September 12, 1960, available on GeoDOG at https://geodog.dot.ca.gov/.

California Department of Transportation (Caltrans), 1951, As-Built Plans, 1951.

California Department of Transportation (Caltrans), Division of New technology, Materials & Research, Office of Engineering Geology – South, 1991, "San Dieguito River Bridge Foundation Update, San Dieguito River Br. (Widen and Retrofit), Bridge No. 57-0488R/L," July 15, 1991, available on GeoDOG at https://geodog.dot.ca.gov/.

California Department of Transportation (Caltrans), Division of New technology, Materials & Research, Office of Engineering Geology – South, 1991, "Foundation Recommendations, San Dieguito River Bridge (Widen and Retrofit), Bridge No. 57-0488R/L," November 22, 1991, available on GeoDOG at https://geodog.dot.ca.gov/.

California Department of Transportation (Caltrans), Division of Structures, 1992, Log of Test Borings, San Dieguito River (Widen), January 22, 1992, available on GeoDOG at https://geodog.dot.ca.gov/.

California Department of Transportation (Caltrans), Division of Engineering Services, 2003, “Preliminary Seismic Design Recommendations, San Dieguito River Br. (Widen NB and SB), Br. No. 57-0488R/L,” October 11, 2002, available on GeoDOG at https://geodog.dot.ca.gov/.

California Department of Transportation (Caltrans), Division of Engineering Services, 2003, “Preliminary Geological Recommendations, San Dieguito River Bridge (widen NB and SB), Br. No. 57-0488R/L,” January 23, 2003, available on GeoDOG at https://geodog.dot.ca.gov/.

California Department of Transportation (Caltrans), Engineering Service Center, 2004, “Preliminary Foundation/Seismic Recommendations, San Dieguito River Bridge (replace), Bridge No. 57-0488L/R,” December 6, 2004, available on GeoDOG at https://geodog.dot.ca.gov/.

California Department of Transportation (Caltrans), 2010, Tsunami Hazard Guidelines, Memo to Designers 20-13, January 2010.

California Department of Transportation (Caltrans), 2017, Mechanically Stabilized Embankment, Caltrans Geotechnical Manual, June 2017.

California Department of Transportation (Caltrans), Division of Engineering Services, 2018, “Corrosion Guidelines”, Version 3.0, March 2018.

California Department of Transportation (Caltrans), 2018, “Standard Specifications,” 2018.

California Department of Transportation (Caltrans), 2018, “Standard Plans,” 2018.

California Department of Transportation (Caltrans), 2019, California Amendments to the AASHTO LRFD bridge Design Specifications (2017 Eighth Edition), April 2019.

California Department of Transportation (Caltrans), 2019 “Caltrans Seismic Design Criteria,” Version 2.0, April 2019.

California Department of Transportation (Caltrans), Division of Engineering Services, 2020 “Geotechnical Design Report Guidelines,” January 2020.

California Department of Transportation (Caltrans), 2020, Liquefaction Evaluation, Caltrans Geotechnical Manual, January 2020.

California Emergency Management Agency, 2009, “Tsunami Inundation Map for Emergency Planning, State of California, County of San Diego, Del Mar Quadrangle”, California Geological Survey, June 1, 2009.

Federal Emergency Management Agency (FEMA), National Flood Insurance Program, available online at http://hazards.fema.gov/mapviewer/.

Grant, L. B., and Shearer, P. M., 2004, “Activity of the offshore Newport-Inglewood Rose Canyon Fault Zone, coastal Southern California, from relocated microseismicity: Bulletin of Seismological Society of America”, Volume 94, No. 2; pp. 747-752.

Jennings, C.W., 1994, “Fault Activity Map of California and Adjacent Areas”, California Division of Mines and Geology, California Geologic Map Series, Map No. 6.

Jennings, C.W. and Bryant, W.A., 2010, “Fault Activity Map of California”, Geologic Data Map No. 6.

Kennedy, M.P. 1975. Geology of the San Diego Metropolitan Area, California, Del Mar, La Jolla, Point Loma, La Mesa, Poway, and SW1/4 Escondido 7½ minute quadrangles, California Division of Mines and Geology, Bulletin 200, 56p.

Kennedy, M.P., and Welday E.E., 1980, “Character and Recency of Faulting Offshore, Metropolitan San Diego, California”, California Division of Mines and Geology, Map Sheet 40, Scale 1:50,000.

Leighton Consulting, Inc., 2017, “San Dieguito River Bridge Replacement, Double Track and Del Mar Fairgrounds Special Events Platform (Milepost 242 to Milepost 244) 90% Design, Draft Foundation Report, Bridge 243.0,” May 5, 2017.

National Oceanic and Atmospheric Administration (NOAA), Tides and Currents, accessed May 2020 at https://tidesandcurrents.noaa.gov/.

Ninyo & Moore, 2018, “Preliminary Foundation Report (PFR), Camino Del Mar Bridge Replacement (Bridge No. 57C-0209), Del Mar, California”, July 31, 2018.

Ninyo & Moore, 2018, “Preliminary Geotechnical Design Report (PGDR), Camino Del Mar Bridge Replacement (Bridge No. 57C-0209), Del Mar, California”, July 31, 2018.

Norris, R.M. and Webb R.W., 1990, “Geology of California”, Second Edition, John Wiley and Sons, Inc., pp. 169-190.

Portland Cement Association (PCA), 1988, Design and Control of Concrete Mixtures, 1988.

Powell, R.E., Weldon, II, R.J and Matti, J.C. (editors) 1993. “The San Andreas Fault System: displacement, palinspastic reconstruction, and geologic evolution”, Geological Society of America Memoir 178, 332p.

Robert Prater Associates, 1980, “Foundation Investigation for Jimmy Durante Bridge, Del Mar, California,” May 1980.

Rockwell, T.K., 2010, “The Rose Canyon Fault in San Diego, Proceedings of the Fifteenth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics”, May 24-29, 2010, San Diego, California, Paper No. 7.06C.

Sahakian, V., Bormann, J., Driscoll, N., Harding, A., Kent, G., and Wesnousky, S., 2017, “Seismic Constraints on the Architecture of the Newport-Inglewood/Rose Canyon Fault: Implications for the Length and Magnitude of Future Earthquake Ruptures”, Journal of Geophysical Research: Solid Earth, 2017: DOI: 10.1002/2016JB013467.

Sampo Engineering, 2018, Topographic Survey, April 13, 2018.

Tokimatsu K, Seed HB (1987): Evaluation of settlements in sands due to earthquake shaking. Journal of Geotechnical Engineering, 113 (8), 861-878.

Treiman, J.A., 1993, “The Rose Canyon Fault Zone, Southern California”, California Division of Mines and Geology, Open File Report 93-02.

United States Department of Agriculture (USDA) 1953, Aerial Photographs.

United States Department of Agriculture (USDA) National Resources Conservation Service, Custom Soil resource Report for San Diego County Area, California, accessed May 2020 at https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm.

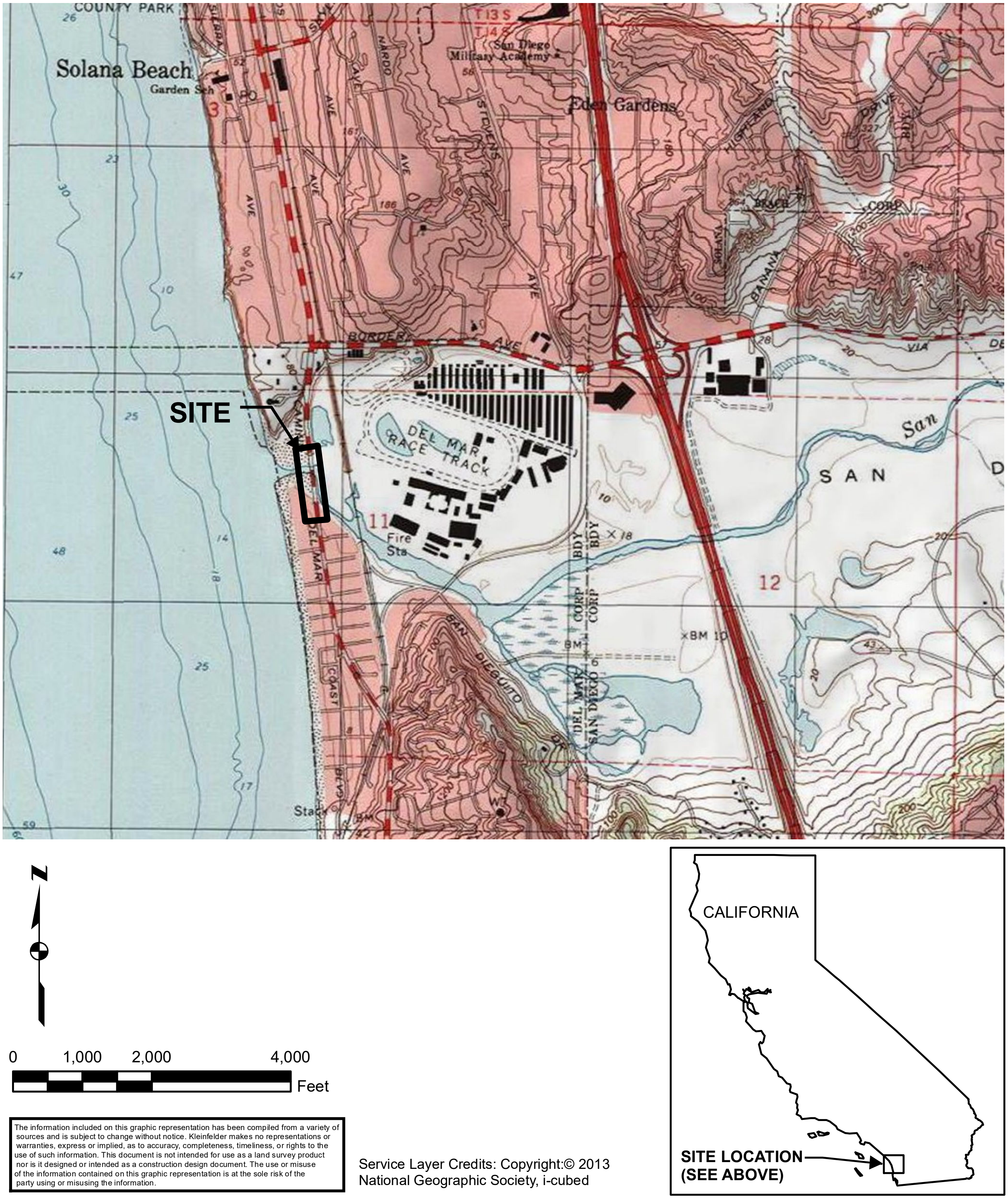
Van Delinder, L.S., 1984, “Corrosion Basics: An Introduction”, National Association of Corrosion Engineers (NACE), 1984.

Wallace, R.E., 1990, “General features: The San Andreas fault system”, U.S. Geological Survey Professional Paper 1515, p. 3-12.

Youd TL, Idriss IM, Andrus RD, Arango I, Castro G, Christian J, Dobry R, Finn WDL, Harder Jr. LF, Hynes ME, Ishihara K, Koester JP, Liao SSC, Marcuson III WF, Martin GF, Mitchell JK, Moriwaki Y, Power MS, Robertson PK, Seed RB, Stokoe II KH (2001): Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. Journal of Geotechnical and Geoenvironmental Engineering, 127 (10), 817-833.

Zhang G, Robertson PK, Brachman RWI (2002): Estimating liquefaction-induced ground settlements from CPT for level ground. Canadian Geotechnical Journal, 39 (5), 1168-1180.

FIGURES



Service Layer Credits: Copyright:© 2013
National Geographic Society, i-cubed

KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

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

SITE VICINITY MAP

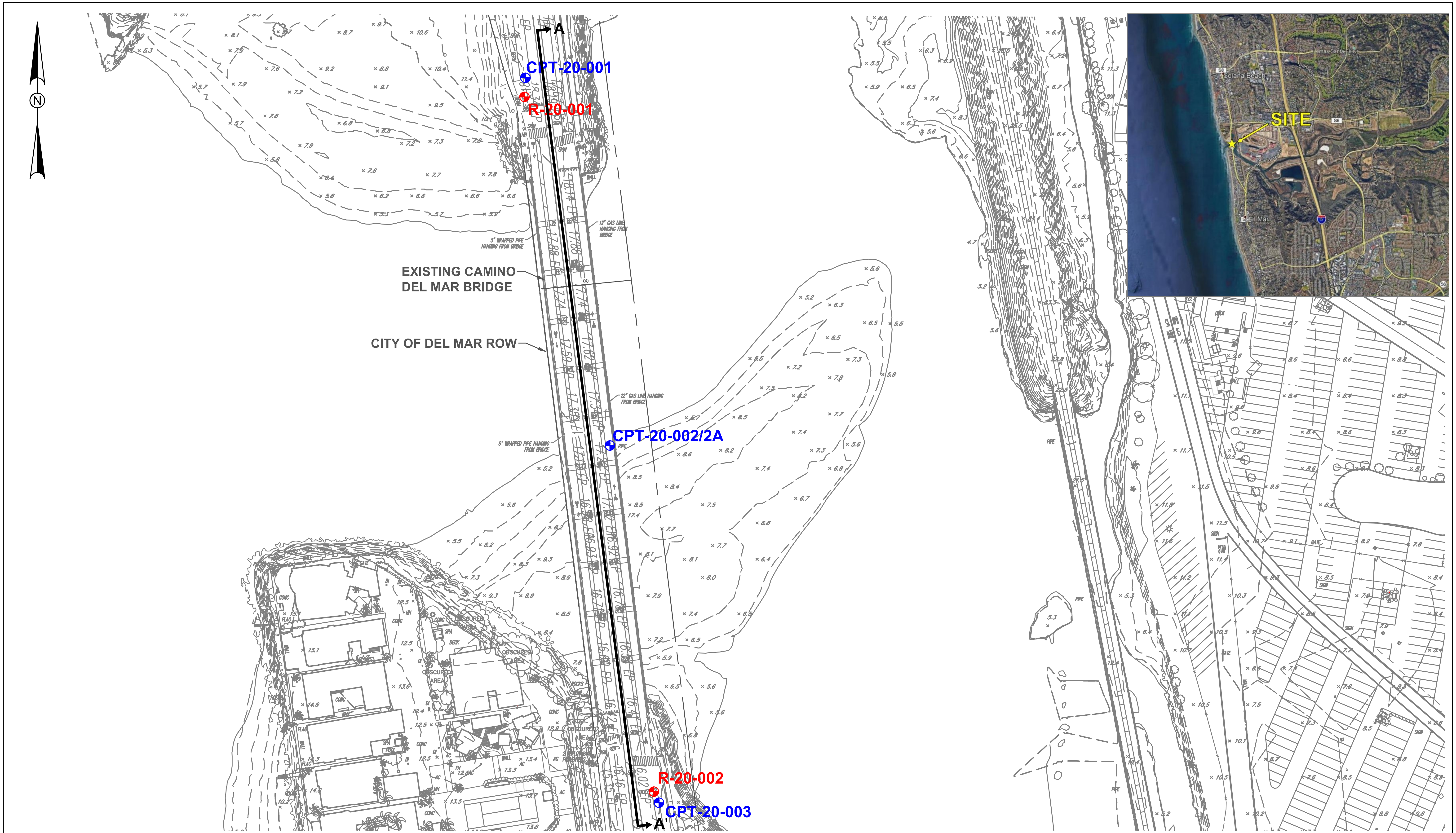
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
1

\\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

PLOTTED: 4/7/2020 12:15 PM BY: janna bonfiglio LAYOUT: Layout1 (2)

CAD FILE: C:\Users\JBonfiglio\OneDrive - Kleinfelder\Desktop\Camino Del Mar Desktop\Resources For Section\Del Mar Bridge TOPO.dwg



NOTES:

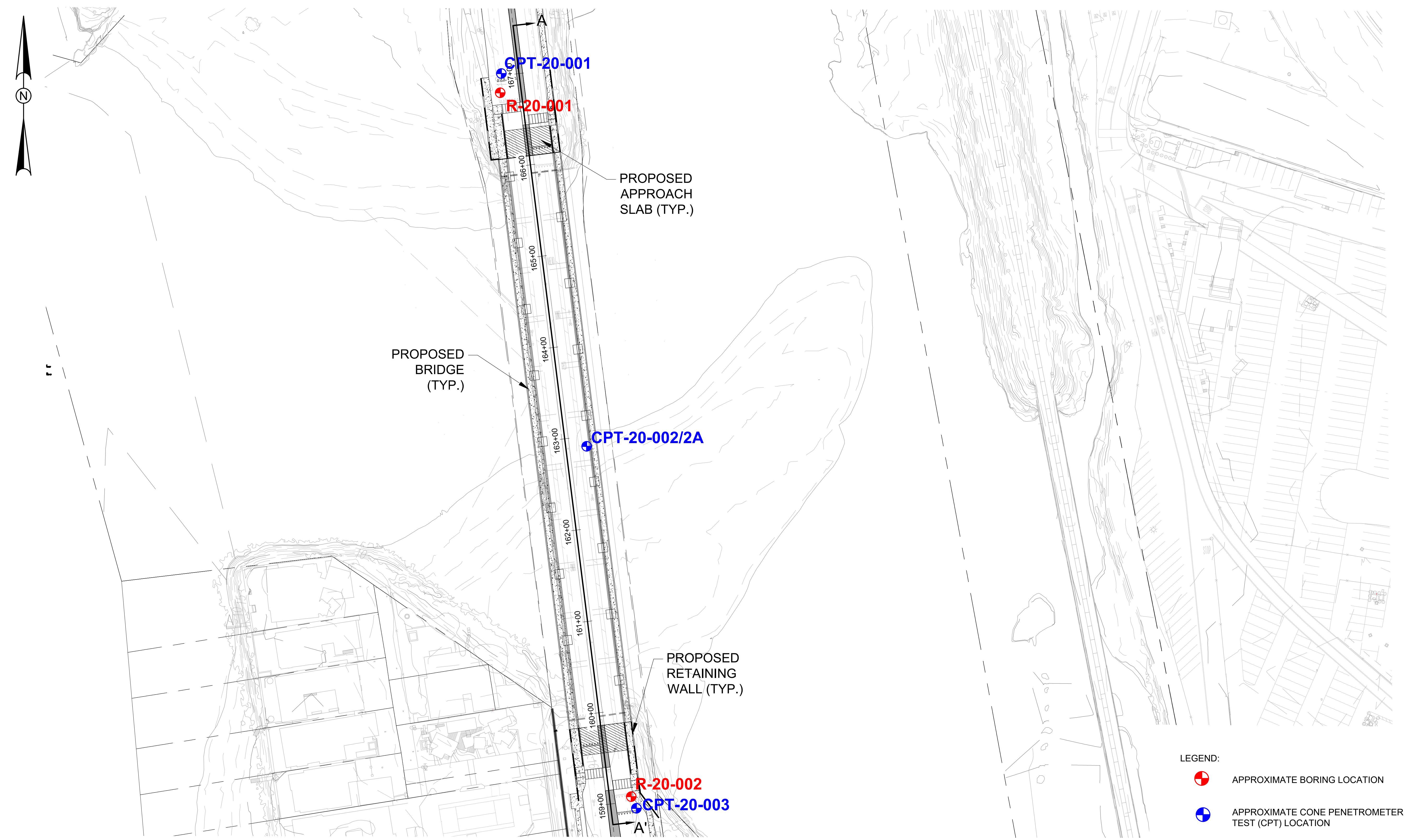
1. EXISTING CONDITIONS REFERENCED FROM THE TOPOGRAPHIC SURVEY PREPARED BY SAMPO ENGINEERING, INC. DATED APRIL 13, 2018.

SCALE: 1" = 100'

100 50 0 100 200

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO. 20180876		SITE AND PHASE 0 EXPLORATION LOCATION PLAN	FIGURE
DRAWN BY	JLB		
CHECKED BY	RAT	CAMINO DEL MAR BRIDGE REPLACEMENT OVER SAN DIEGUITO RIVER - PHASE 0 DEL MAR, CALIFORNIA	2
DATE:	04/2019		
REVISED:	-		



LEGEND:

🔴 APPROXIMATE BORING LOCATION

🔵 APPROXIMATE CONE PENETROMETER TEST (CPT) LOCATION

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

KLEINFELDER
Bright People. Right Solutions.

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

PROPOSED CONDITIONS AND PHASE 0
EXPLORATION LOCATION PLAN

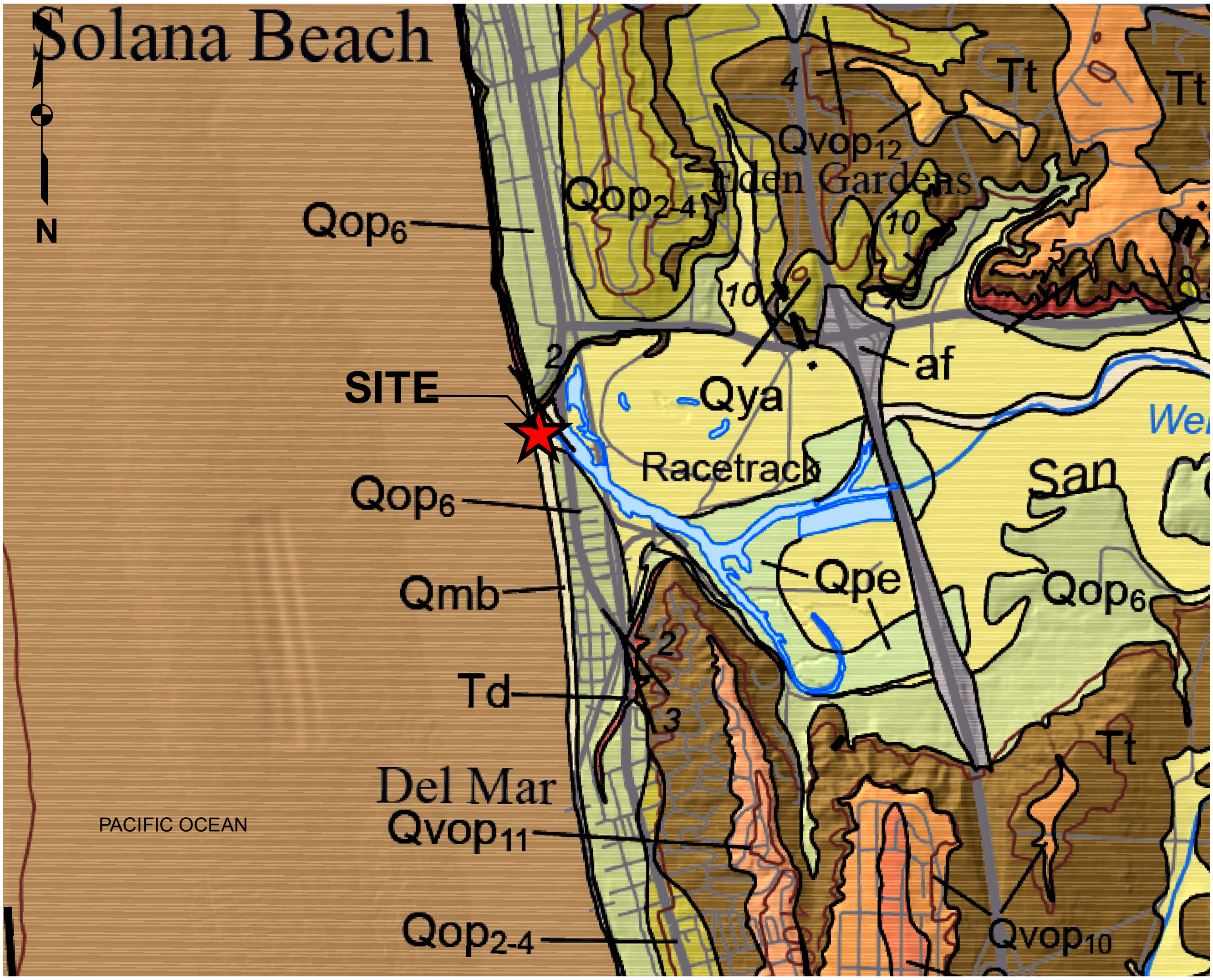
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
3

PAGE: 1 of 1

CAD FILE: C:\Users\JBonfiglio\OneDrive - Kleinfelder\Desktop\Camino Del Mar Desktop\Camino Del Mar Bridge Plan ▯ Profile - Standard 05192020\Camino Del Mar Bridge Plan ▯ Profile.dwg LAYOUT: Layout1 PLOTTED: 5/28/2020 5:56 PM BY: janna bonfiglio

\\sandiego\swe-data\G\Bridge Division\Job Files\03 FY2018\0876 - Camino Del Mar Bridge Replacement\Phase B- PAED\Geotech\Deliverables\Figures\Figure 3_Geo Map.mxd



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

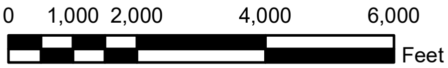
🟧 Qop_{10} - OLD PARALIC DEPOSITS, UNDIVIDED, UNIT 10

🟧 Qop_{10a} - OLD PARALIC DEPOSITS, UNDIVIDED, UNIT 10a

🟧 Qop_{12} - OLD PARALIC DEPOSITS, UNDIVIDED, UNIT 12

🟥 Td - DELMAR FORMATION

🟫 Tt - TORREY SANDSTONE



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

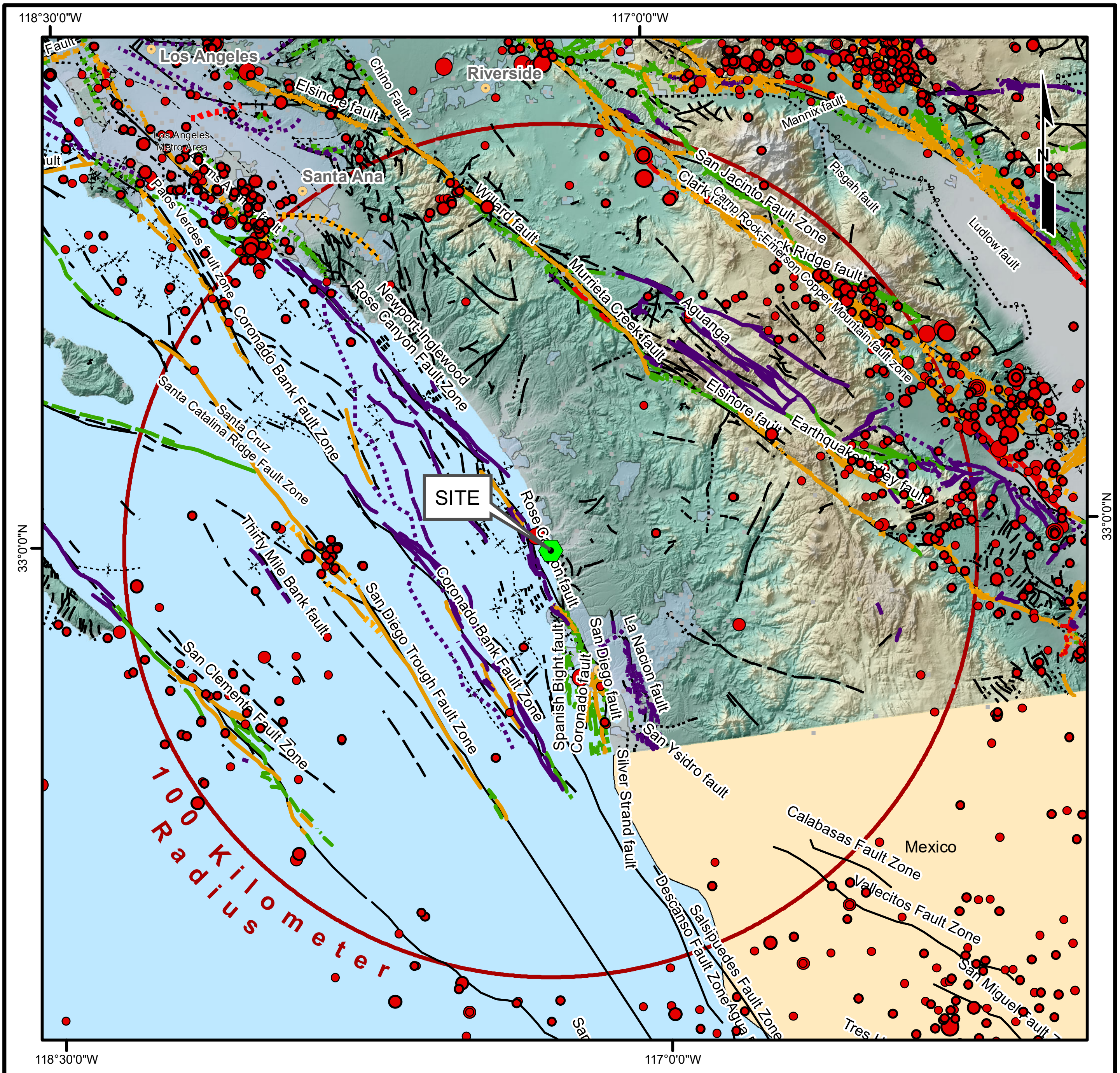
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO. 20191449
DRAWN: 12/26/2019
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



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

KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

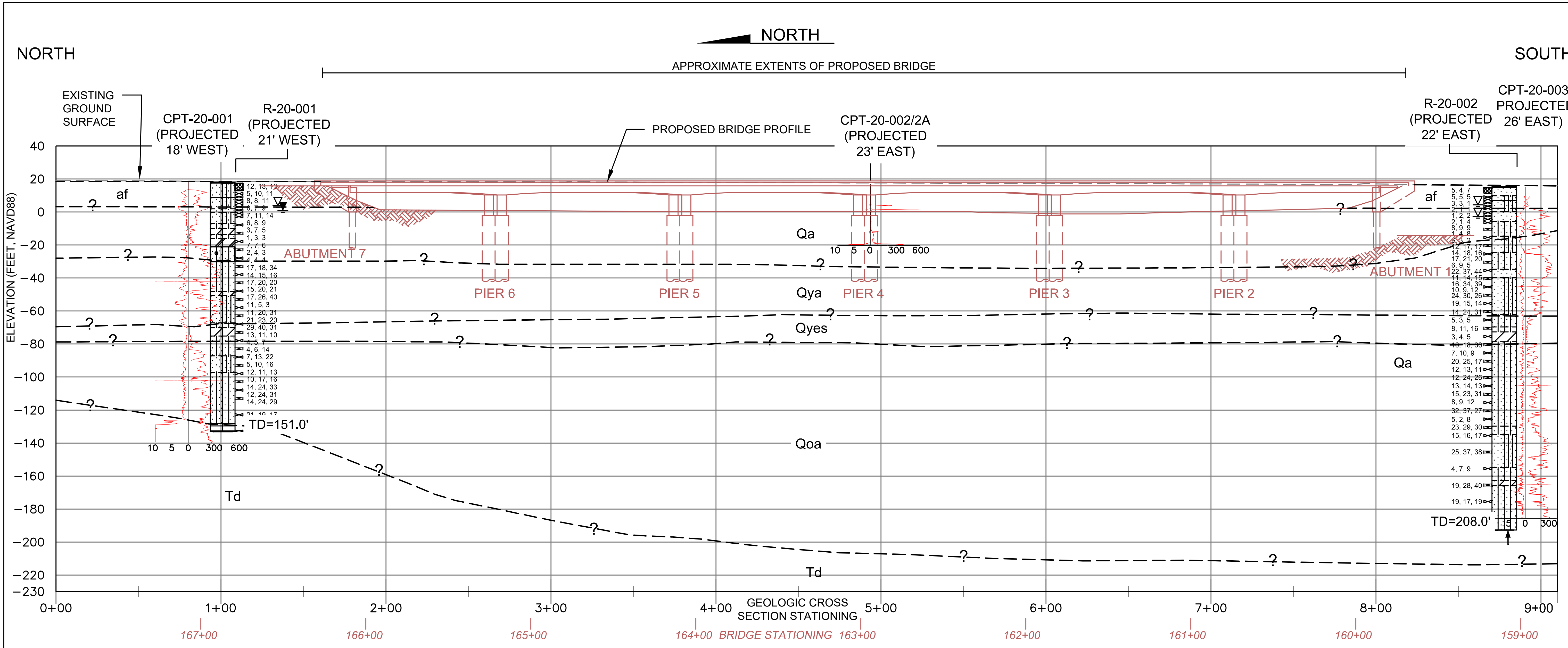
PROJECT NO. 20180876
DRAWN: 12/26/2019
DRAWN BY: JB
CHECKED BY: KR
FILE NAME:
EQ Map.MXD

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

\\sandiego\swe-data\G\Bridge Division\Job Files\03 FY2018\0876 - Camino Del Mar Bridge Replacement\Phase B- PAED\Geotech\Deliverables\Figures\Figure 4_EQ Map.mxd



LEGEND

▨ ARTIFICIAL FILL

▨ RECENT ALLUVIAL DEPOSITS

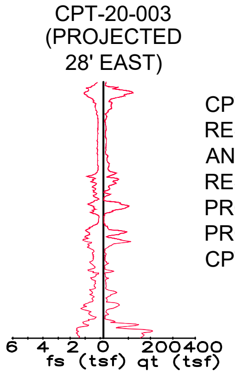
▨ YOUNG ALLUVIAL DEPOSITS

▨ YOUNG ESTAURINE DEPOSITS

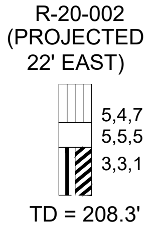
▨ OLD ALLUVIAL DEPOSITS

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)

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO. 20180876
DRAWN BY JLB/JSP
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

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)

KLEINFELDER
Bright People. Right Solutions.

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

gINT FILE: Klf_gint_master_2018 PROJECT NUMBER: 20180876.001A
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.

KLEINFELDER
Bright People. Right Solutions.

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

CORE RECOVERY CALCULATION (%)

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

RQD CALCULATION (%)

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

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

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

KLEINFELDER
Bright People. Right Solutions.

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	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
S.Tena	**2-18-20**	**2-21-20**	**32.97607° / -117.26928° WGS84**	**R-20-001**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Pacific Drilling	**Sta N/A**	**~16.00 ft NAVD88**

DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Mud Rotary	**Marl 10**	**8 in / 4 in**

SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT (1.4"), CAL (2.5")	**Auto; 140 lbs / 30-inch drop**	**94%**

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite and grout		**14.0 ft**	**17.0 ft on 2-21-20**	**151.0 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; (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
	5		- yellow (10YR 7/6) and dark yellowish brown (10YR 4/4); coarse to fine SAND.												
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
-5.0	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
	25														

(continued)

KLEINFELDER
Bright People. Right Solutions.

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**	**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
-10.0	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							
			LEAN CLAY (CL); very soft; dark gray (2.5Y 4/1); wet; few fine SAND; mostly fines; medium plasticity.												
-15.0	30				S11	2 4 3	7	33		55	66	PP=0.0			M, UW, PI
			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.												
-20.0	35				S12	4 4 4	8	55				PP=0.0			PA, PI
			SILTY SAND with GRAVEL (SM); wet; (inferred from drilling action).												Hard drilling due to gravel layers.
-25.0	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.
-30.0	45					14 15 16	31	NR							No sample recovery at 45 to 46.5 feet.
			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)).												
-35.0	50				S13	17 20 20	40	44		21	110				M, UW, PA
	55														

(continued)

KLEINFELDER
Bright People. Right Solutions.

REPORT TITLE
BORING RECORD

HOLE ID
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**	**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
	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							
-40.0															
	60		- very dense; micaceous.		S15	17 26 40	66	55		25	102				M, UW, PA, PI
-45.0															
	65		- loose.		S16	11 5 3	8	55				PP=0.0			
-50.0			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.												
	70				S17	11 20 31	51	66		21	108				M, UW, PA
-55.0															
	75		- dense.		S18	21 23 20	43	77							PI
-60.0															
	80		- very dense.		S19	29 40 31	71	66		21	107				M, UW, PA
-65.0															
	85														

(continued)

KLEINFELDER
Bright People. Right Solutions.

REPORT TITLE
BORING RECORD

HOLE ID
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**	**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
-70.0	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							
			LEAN CLAY (CL); medium stiff; black (10YR 2/1); wet; few fine SAND; mostly fines; medium plasticity; micaceous, trace shell fragments.												
-75.0	90				S21	4 5 7	12	66		47	77	PP=0.5			M, UW, PI, WA
-80.0	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							
-85.0	100		- dense.		S23	7 13 22	35	66							DS
-90.0	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							
-95.0	110		- SAA.		S25	12 11 13	24	66		32	92				M, UW, PA
	115														

(continued)

KLEINFELDER
Bright People. Right Solutions.

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
-100.0	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							
-105.0	120		- very dense.	⧗	S27	14 24 33	57	33		25	103				M, UW, PA
-110.0	125		- SAA.	⧖	S28	12 24 31	55	89							
-115.0	130		- SAA.	⧗	S29	14 24 29	53	66							DS Increase in drilling effort at 132 feet.
-120.0	135														
-125.0	140		- dense.	⧖	S30	21 19 17	36	100							
	145														

(continued)

KLEINFELDER
Bright People. Right Solutions.

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 **5 of 6**

PLOTTED: 04/21/2020 02:03 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
	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														

KLEINFELDER
Bright People. Right Solutions.

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	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
S.Tena	**2-10-20**	**2-13-20**	**32.97396° / -117.26878° WGS84**	**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)	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
	5													
10.0		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
		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							
	10													
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							
	15													
0.0		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
					1 4 8	12	NR							No sample recovery at 19 to 20.5 feet.
	20													
-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)

KLEINFELDER
Bright People. Right Solutions.

REPORT TITLE				HOLE ID
BORING RECORD				**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**

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); 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
-10.0															
	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
-15.0															
	35		- SAA.		S12	17 21 20	41	55		24	101				M, UW
-20.0															
	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
-25.0															
	45		- very dense; trace GRAVEL, 3 in. max. dia.; medium SAND; little fines.		S14	22 37 44	81	66		24	105				M, UW
-30.0															
	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
-35.0															
	55														

(continued)

KLEINFELDER
Bright People. Right Solutions.

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**

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		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
-45.0	60		- medium dense.	⧖	S17	10 9 12	21	77							PA
-50.0	65		- very dense.	⧗	S18	24 30 26	56	66		29	96				M, UW
-55.0	70		- medium dense.	⧖	S19	19 15 14	29	77							PA
-60.0	75		- very dense.	⧗	S20	14 24 31	55	66		30	96				M, UW
			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)).												
-65.0	80			⧖	S21	5 3 5	8	100				PP=0.5			PA, PI
	85														

(continued)

KLEINFELDER
Bright People. Right Solutions.

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**

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] 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
	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
-70.0															
			LEAN CLAY (CL); medium stiff; dark gray (10YR 4/1); wet; few fine SAND; mostly fines; low plasticity; micaceous, trace shell fragments.												
	90			⧖	S23	3 4 5	9	100				PP=0.5			PA
-75.0															
			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)).												
	95			⧗	S24	10 18 30	48	66		28	95				M, UW, UC
-80.0															
	100		- medium dense; some fines; non-plastic to low plasticity; interbeded layer (1") of Silty Clay material.	⧖	S25	7 10 9	19	100							PA
-85.0															
	105		- dense; little fines; non-plastic.	⧗	S26	20 25 17	42	33							DS
-90.0															
	110		- medium dense.	⧖	S27	12 13 11	24	83							PA, PI
-95.0															
	115														

(continued)

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; 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
-100.0															
	120		- medium dense; coarse to medium SAND.	⧖	S29	13 14 13	27	77							
-105.0															
	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
-110.0															
	130		- medium dense.	⧖	S31	8 9 12	21	89							
-115.0															
	135		- very dense.	⧗	S32	32 37 27	64	44							
-120.0															DS
	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
-125.0															
	145														

(continued)

KLEINFELDER
Bright People. Right Solutions.

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 **5 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
-130.0	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
-135.0	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
-140.0	155														
-145.0	160		- very dense; coarse to medium SAND.		S36	25 37 38	75	44							DS
-150.0	165														
-155.0	170		SILTY SAND (SM); medium dense; very dark gray; wet; some fines; non-plastic to low plasticity.		S37	4 7 9	16	100							PA
	175														

(continued)

KLEINFELDER
Bright People. Right Solutions.

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 **6 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
	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)

KLEINFELDER
Bright People. Right Solutions.

REPORT TITLE				HOLE ID
BORING RECORD				**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														

KLEINFELDER
Bright People. Right Solutions.

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,

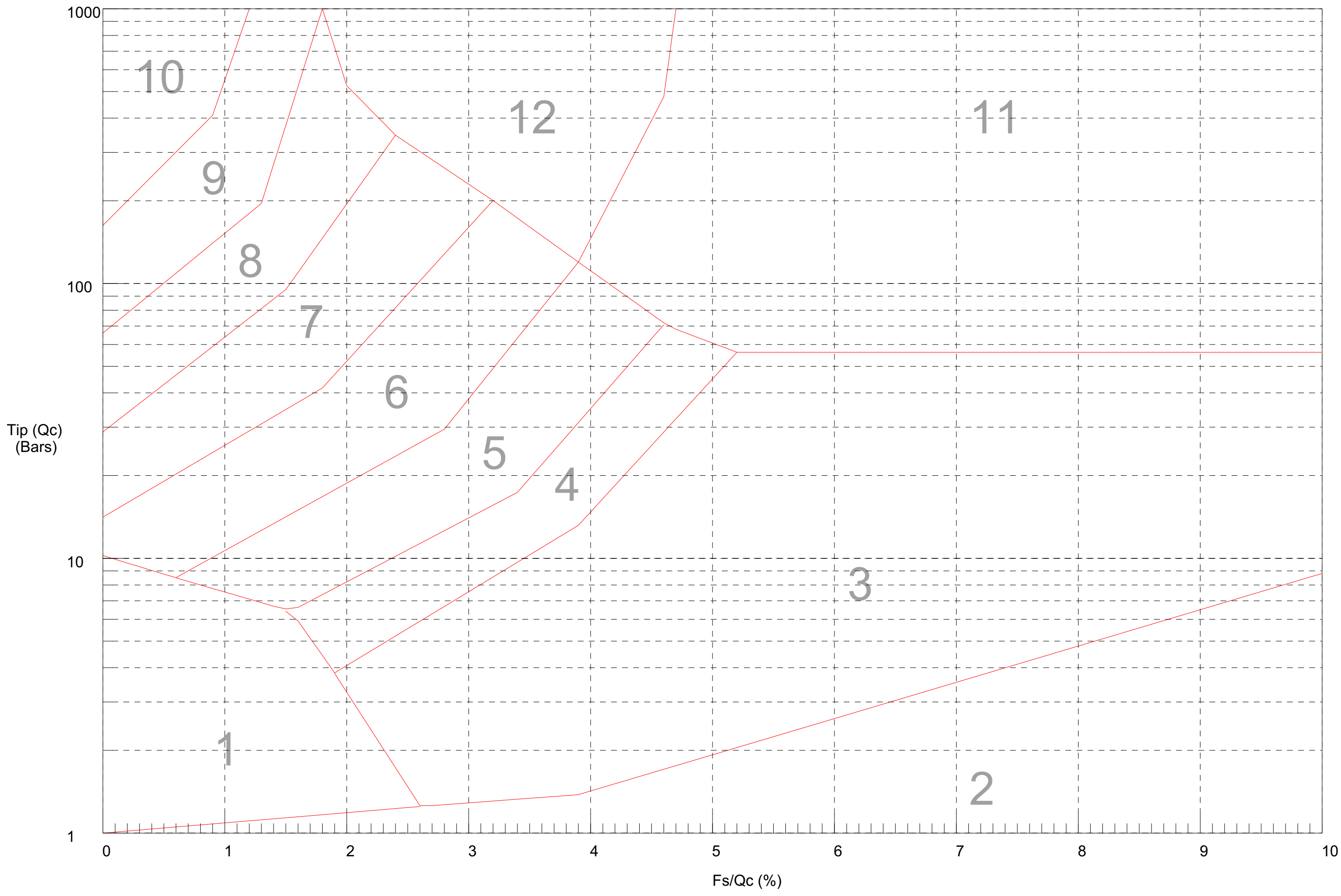
Sheldon Collins
Service Line Manager – CPT
North America

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



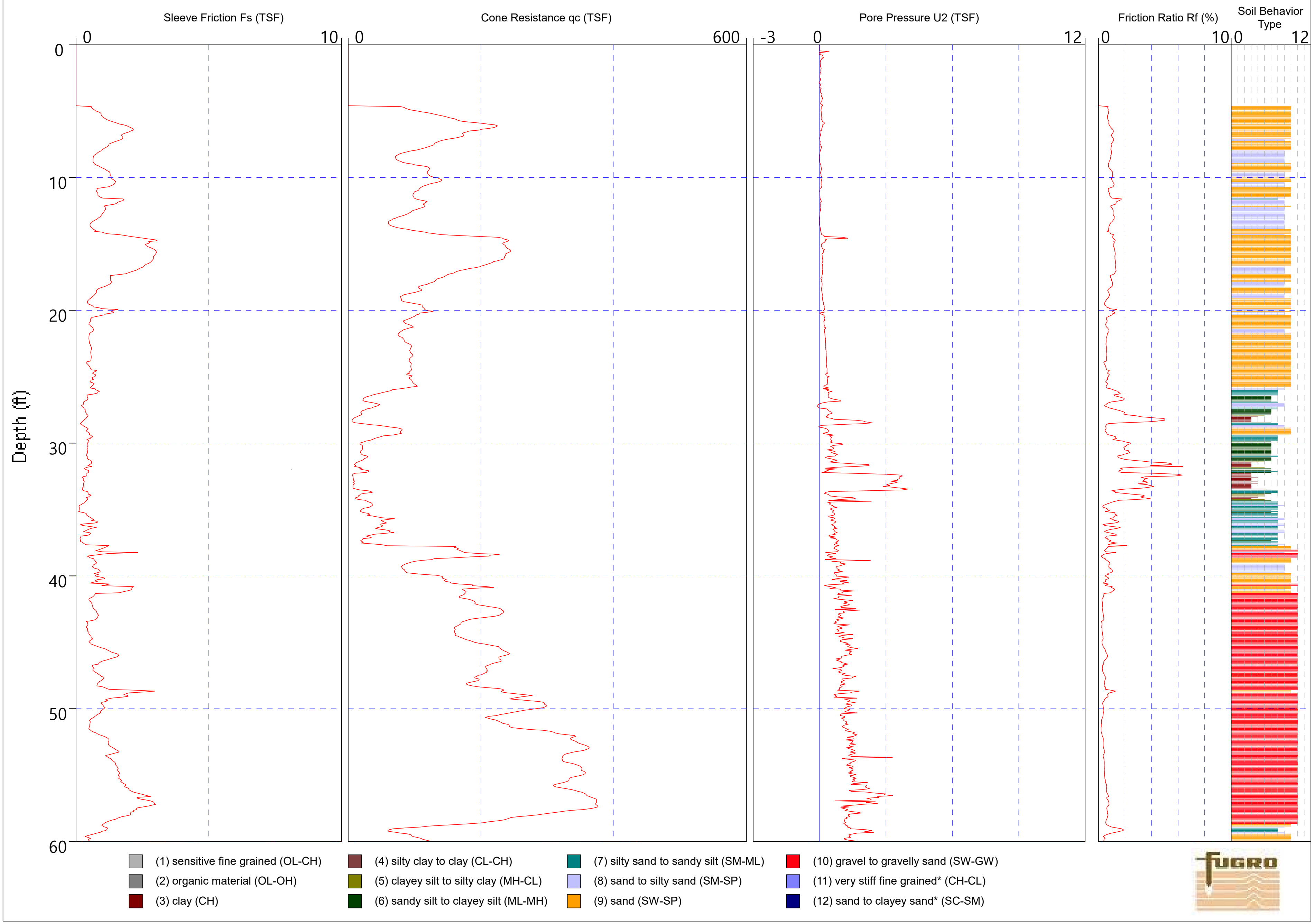
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

* 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

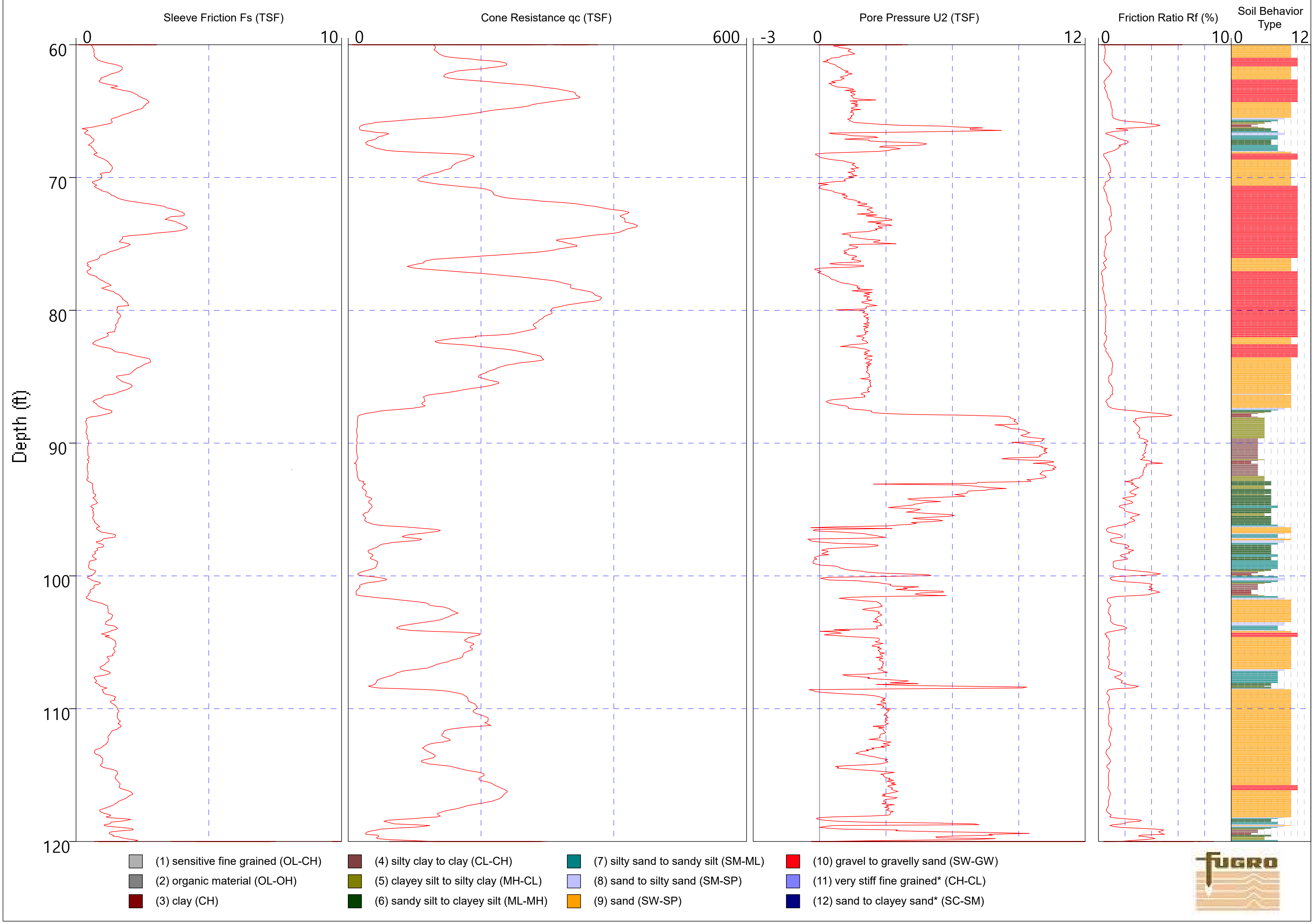


Robertson et al. 1986 *Overconsolidated or Cemented

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

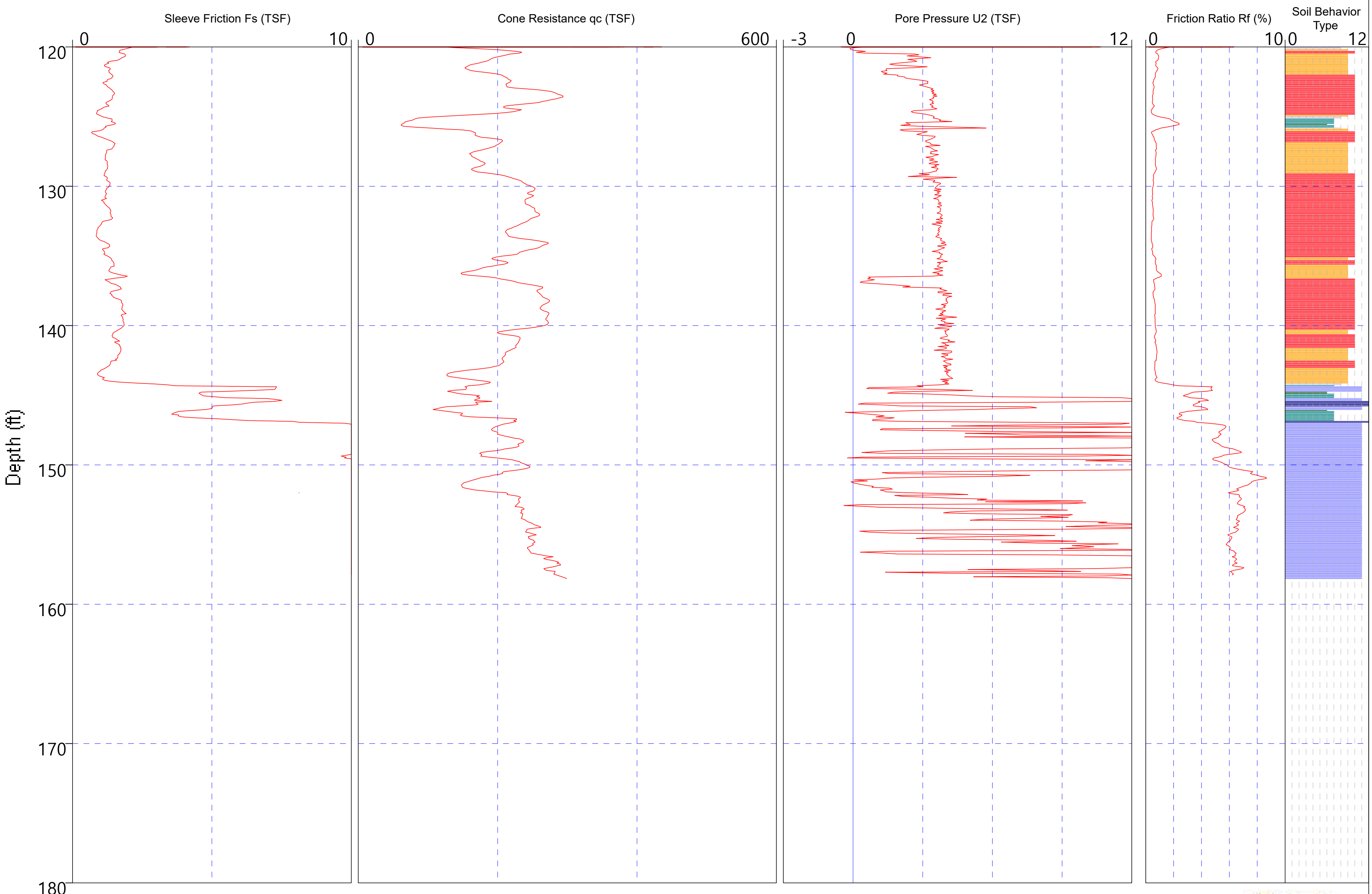


Robertson et al. 1986 *Overconsolidated or Cemented

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



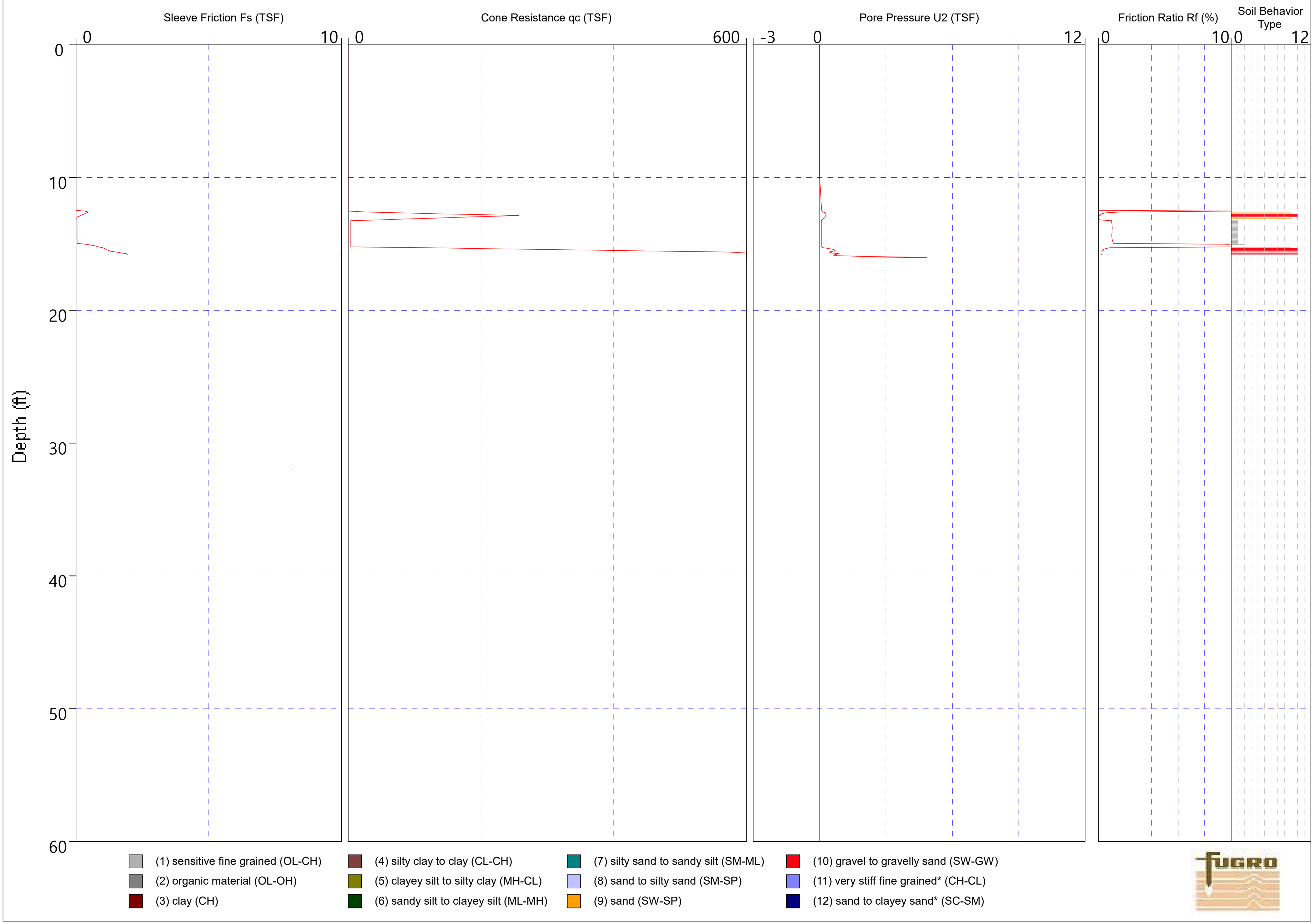
■ (1) sensitive fine grained (OL-CH)
■ (2) organic material (OL-OH)
■ (3) clay (CH)
■ (4) silty clay to clay (CL-CH)
■ (5) clayey silt to silty clay (MH-CL)
■ (6) sandy silt to clayey silt (ML-MH)
■ (7) silty sand to sandy silt (SM-ML)
■ (8) sand to silty sand (SM-SP)
■ (9) sand (SW-SP)
■ (10) gravel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

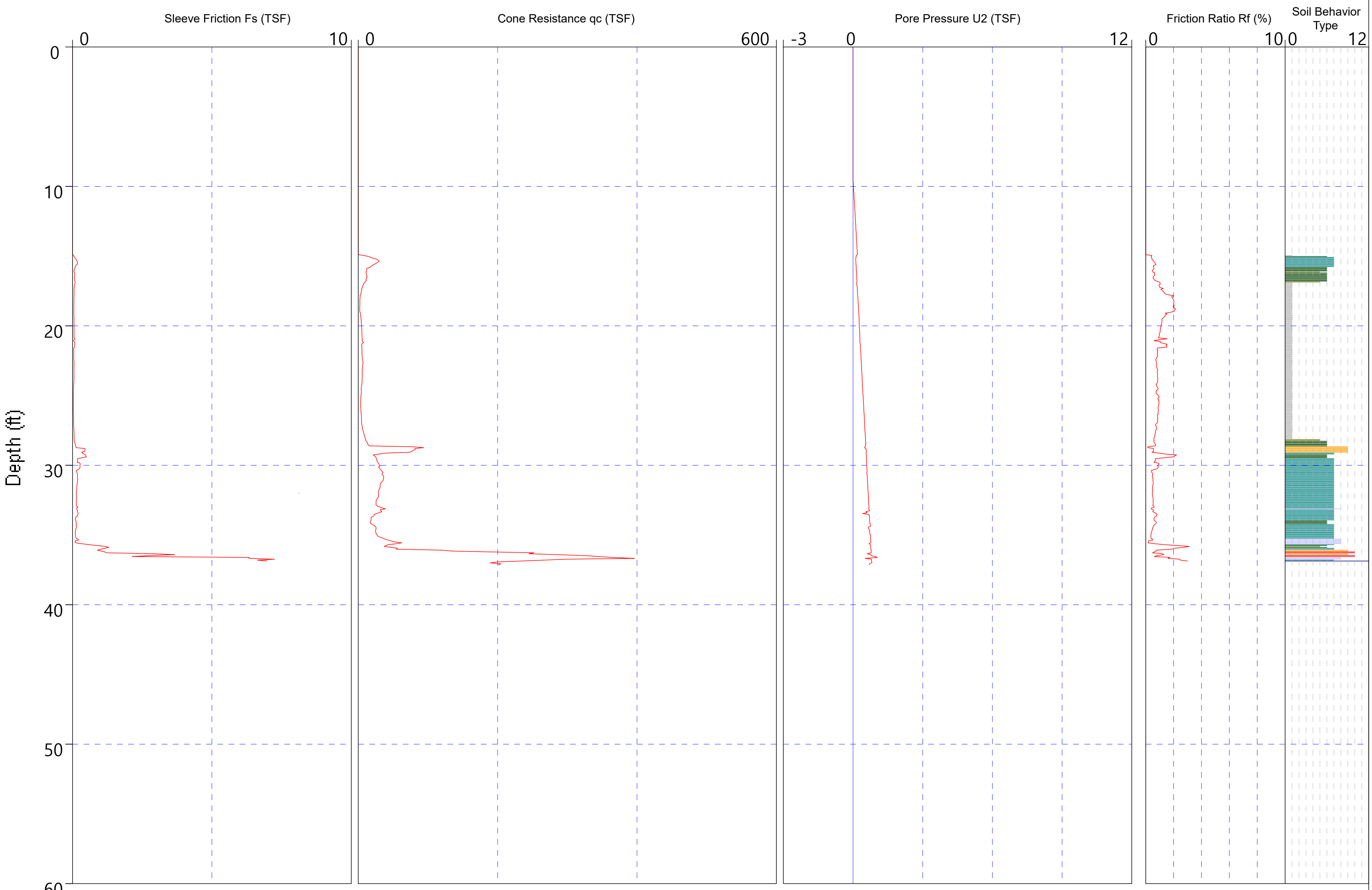


Robertson et al. 1986 *Overconsolidated or Cemented

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



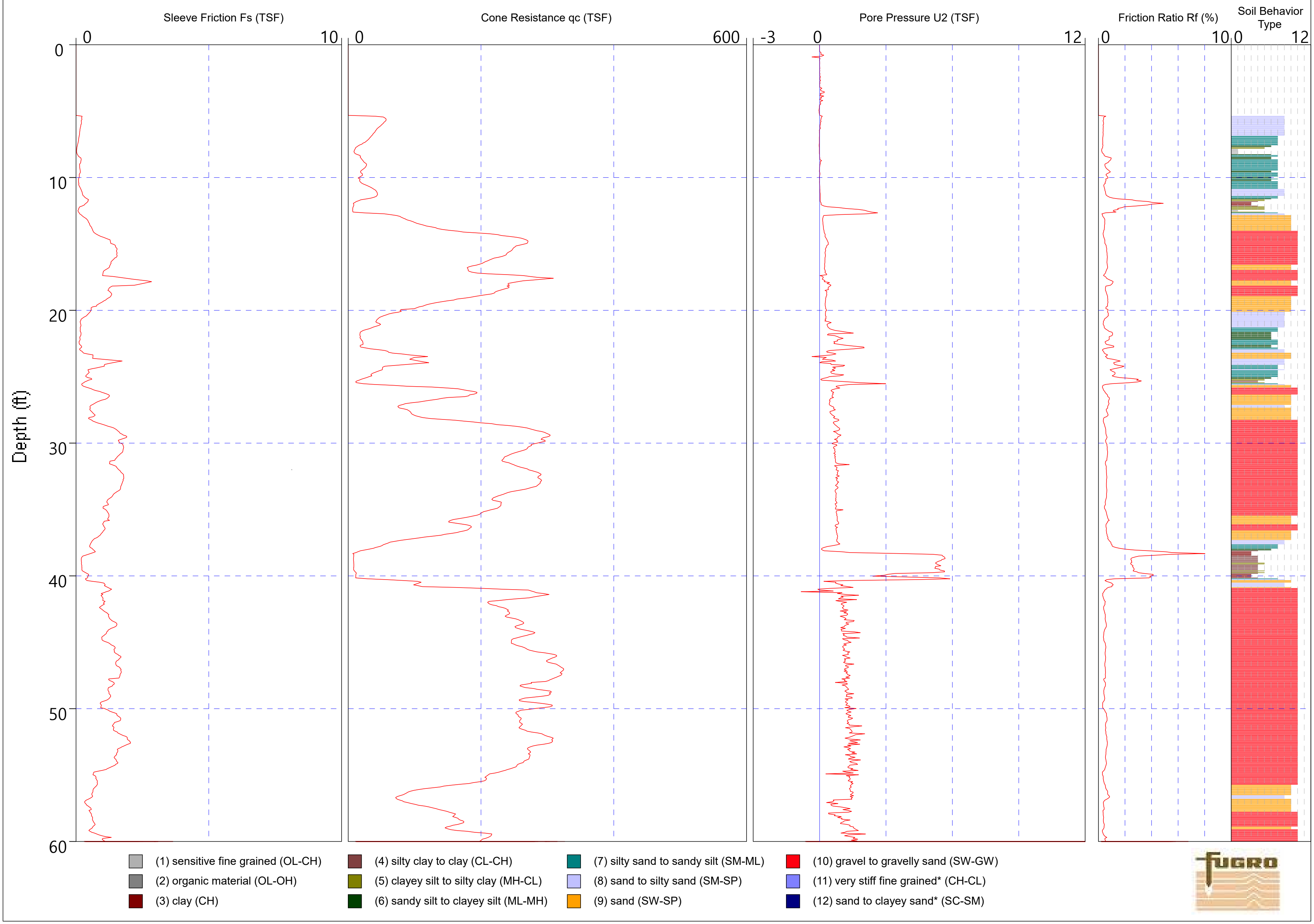
- 🟫 (1) sensitive fine grained (OL-CH)
- 🟫 (2) organic material (OL-OH)
- 🟥 (3) clay (CH)
- 🟫 (4) silty clay to clay (CL-CH)
- 🟫 (5) clayey silt to silty clay (MH-CL)
- 🟩 (6) sandy silt to clayey silt (ML-MH)
- 🟦 (7) silty sand to sandy silt (SM-ML)
- 🟪 (8) sand to silty sand (SM-SP)
- 🟧 (9) sand (SW-SP)
- 🟥 (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

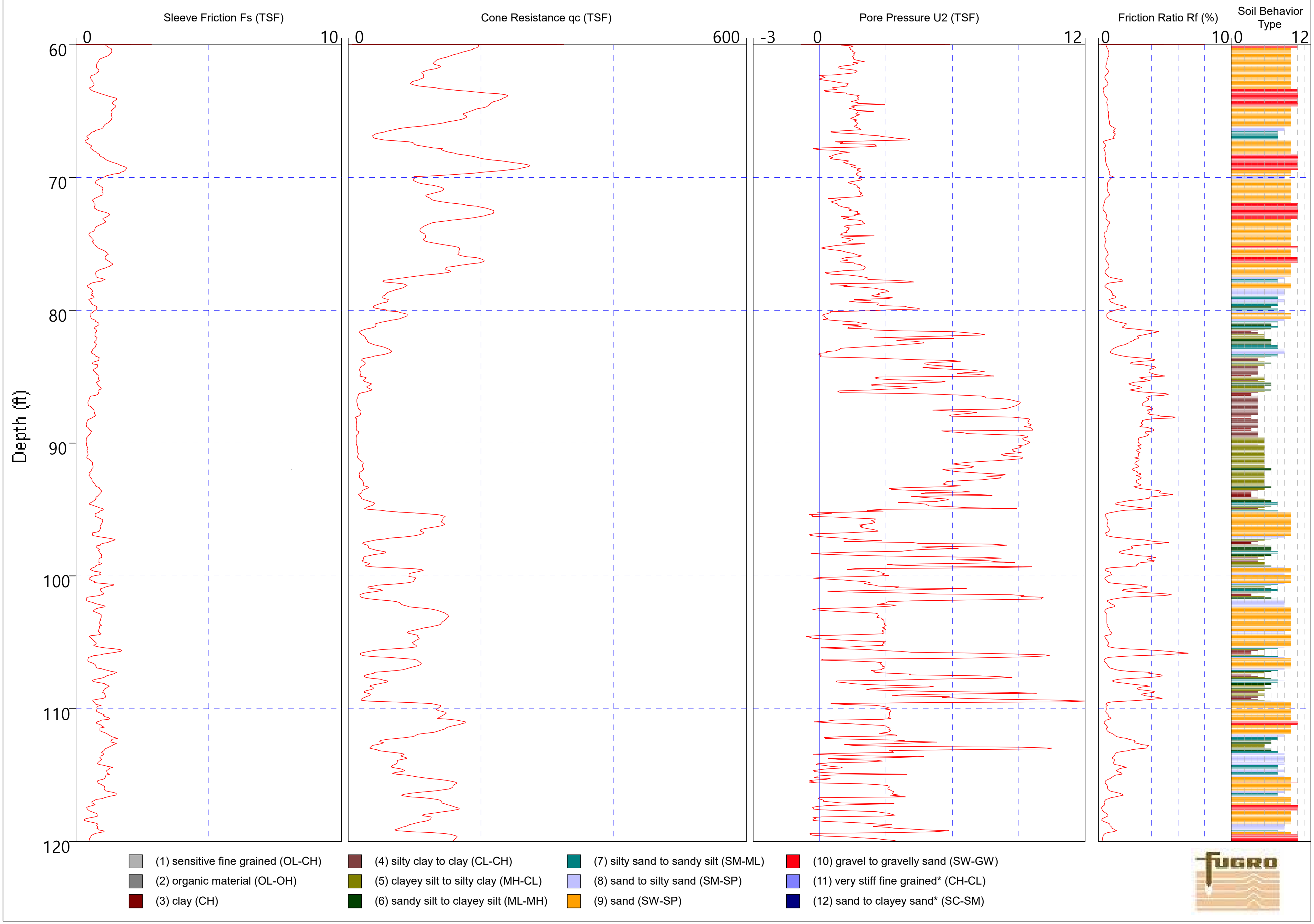


Robertson et al. 1986 *Overconsolidated or Cemented

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

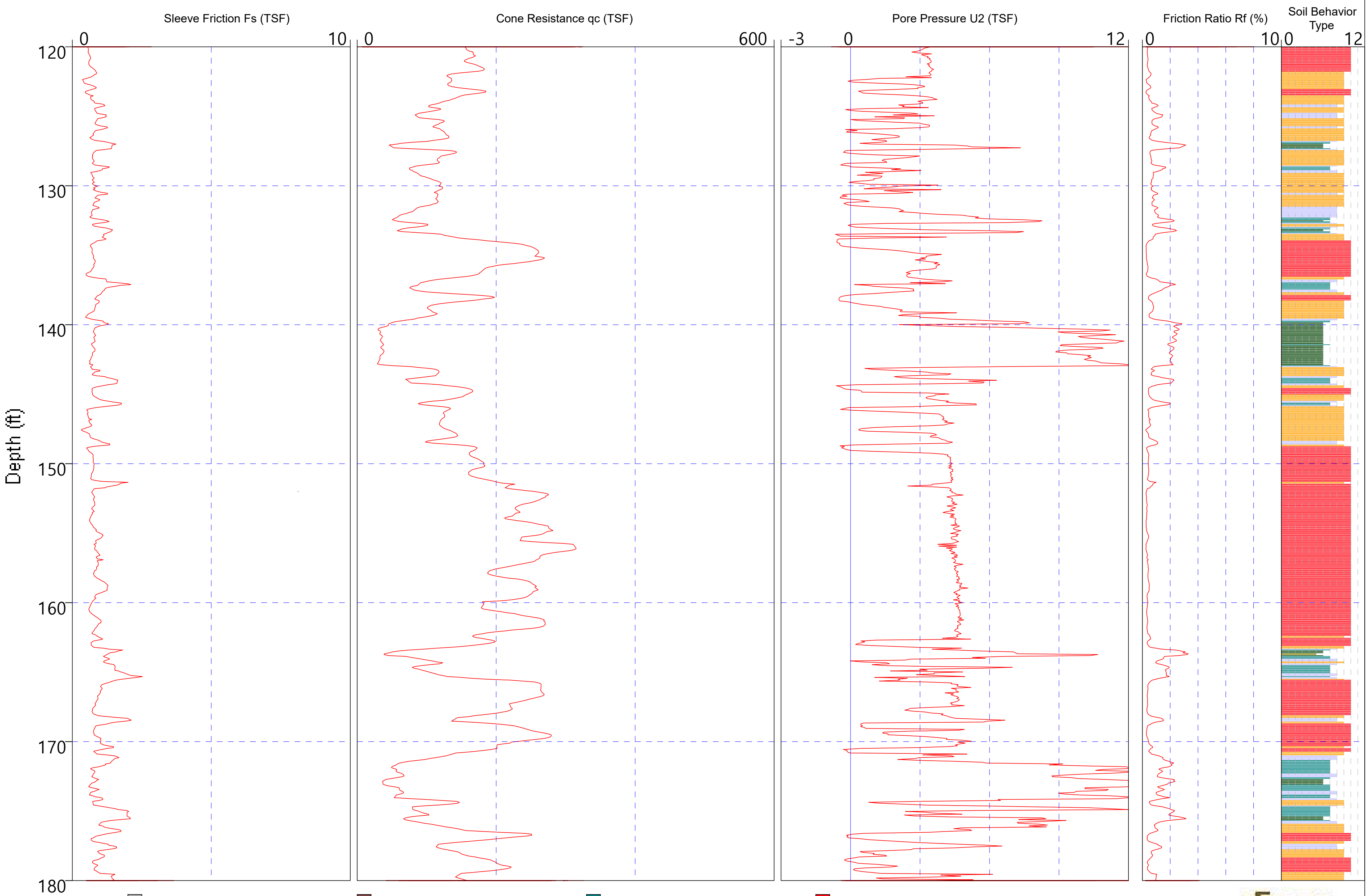
CPT Number: CPT-20-003
Date: 19-Feb-2020
Elevation: 0.00

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



Robertson et al. 1986 *Overconsolidated or Cemented

Job Number:	04.09200002	**CPT Number:**	CPT-20-003	**Coordinates:**	32.973928, -117.268763
Operator:	D. Garza	**Date:**	19-Feb-2020	**Cone Number:**	CP15-CF75PB7SN2-P1E1 2999
Location:	Del Mar, CA	**Elevation:**	0.00		



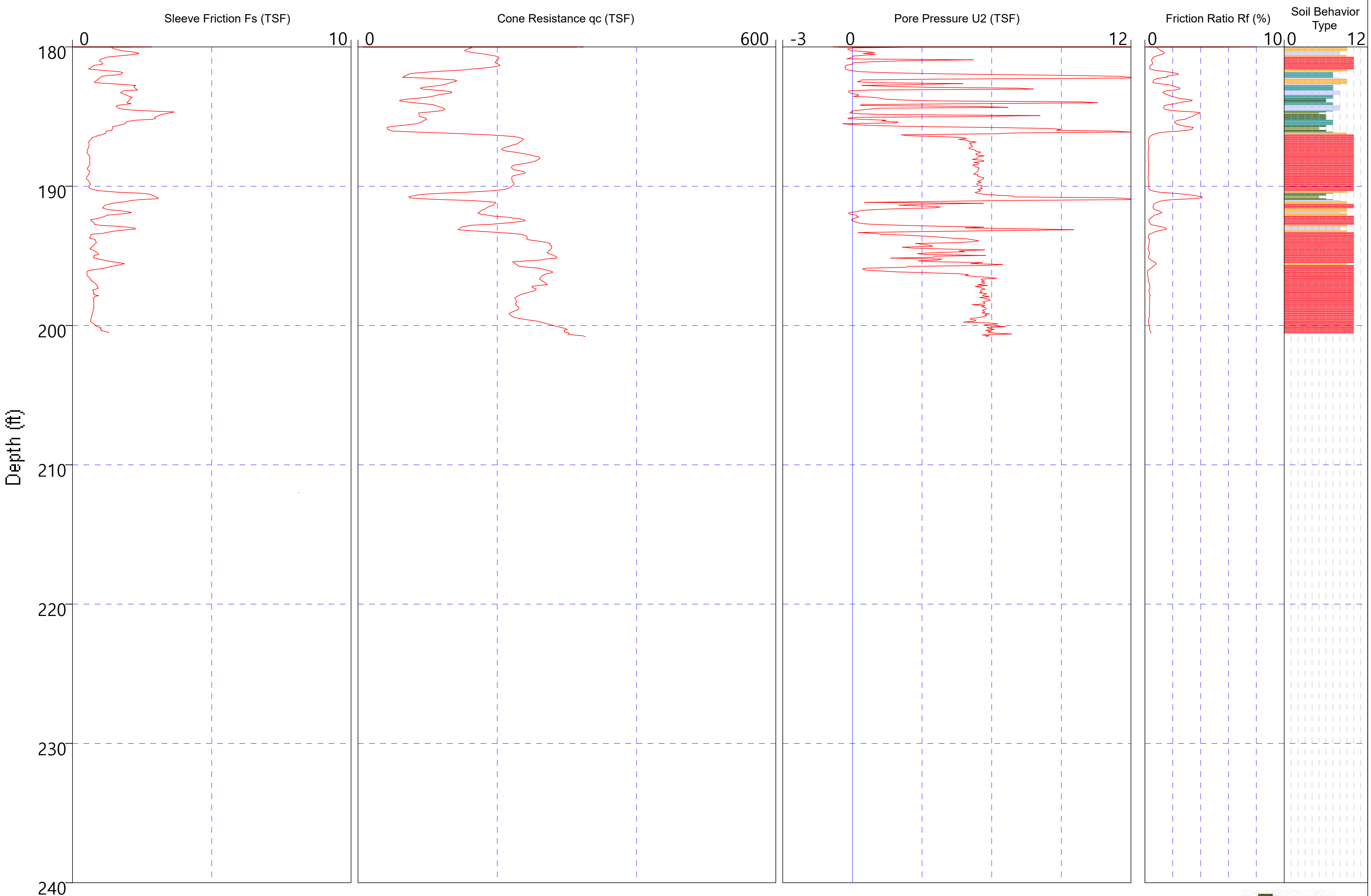
- ■ (1) sensitive fine grained (OL-CH)
- ■ (2) organic material (OL-OH)
- ■ (3) clay (CH)
- ■ (4) silty clay to clay (CL-CH)
- ■ (5) clayey silt to silty clay (MH-CL)
- ■ (6) sandy silt to clayey silt (ML-MH)
- ■ (7) silty sand to sandy silt (SM-ML)
- ■ (8) sand to silty sand (SM-SP)
- ■ (9) sand (SW-SP)
- ■ (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

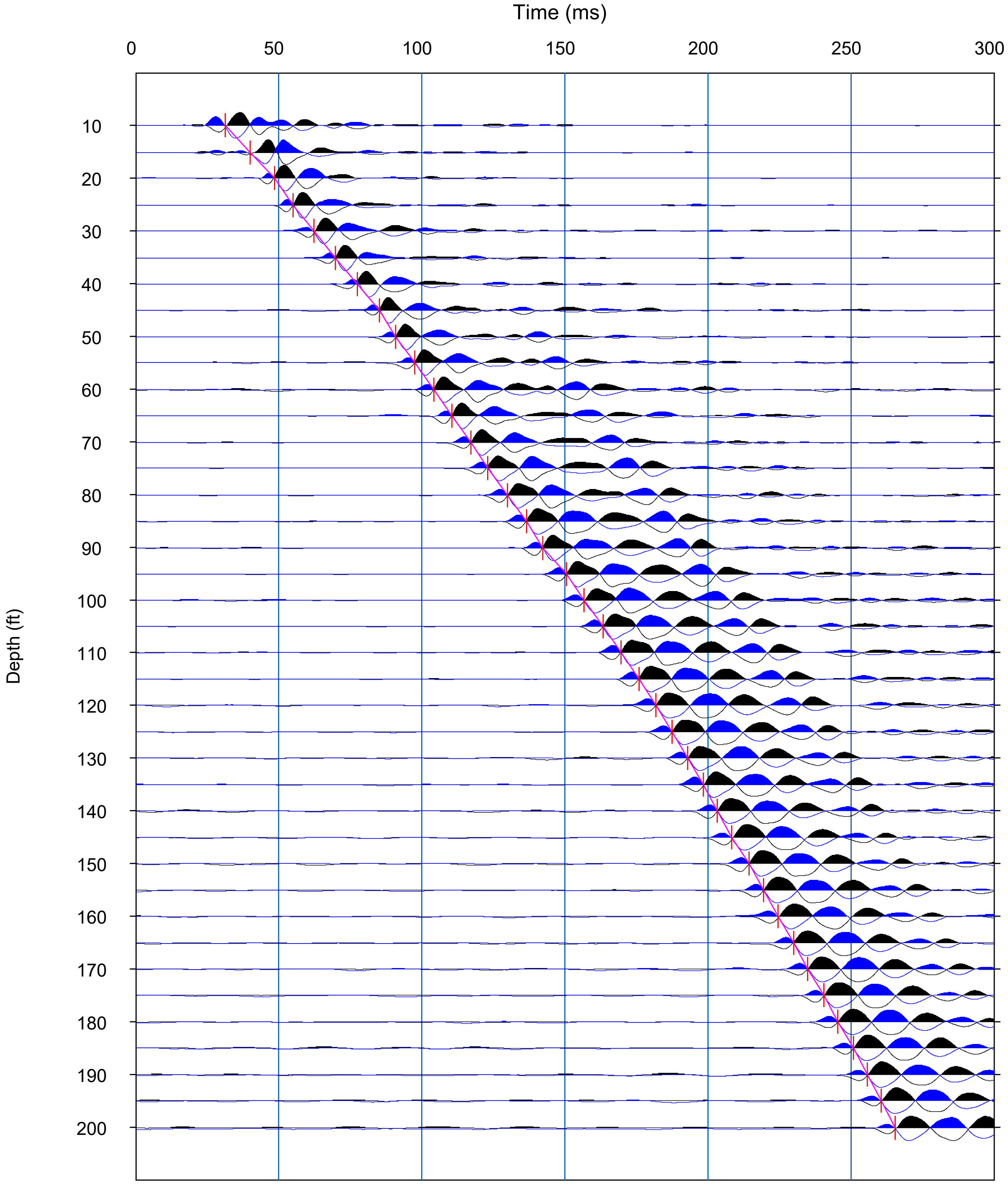
CPT Number: CPT-20-003
Date: 19-Feb-2020
Elevation: 0.00

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



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

Robertson et al. 1986 *Overconsolidated or Cemented



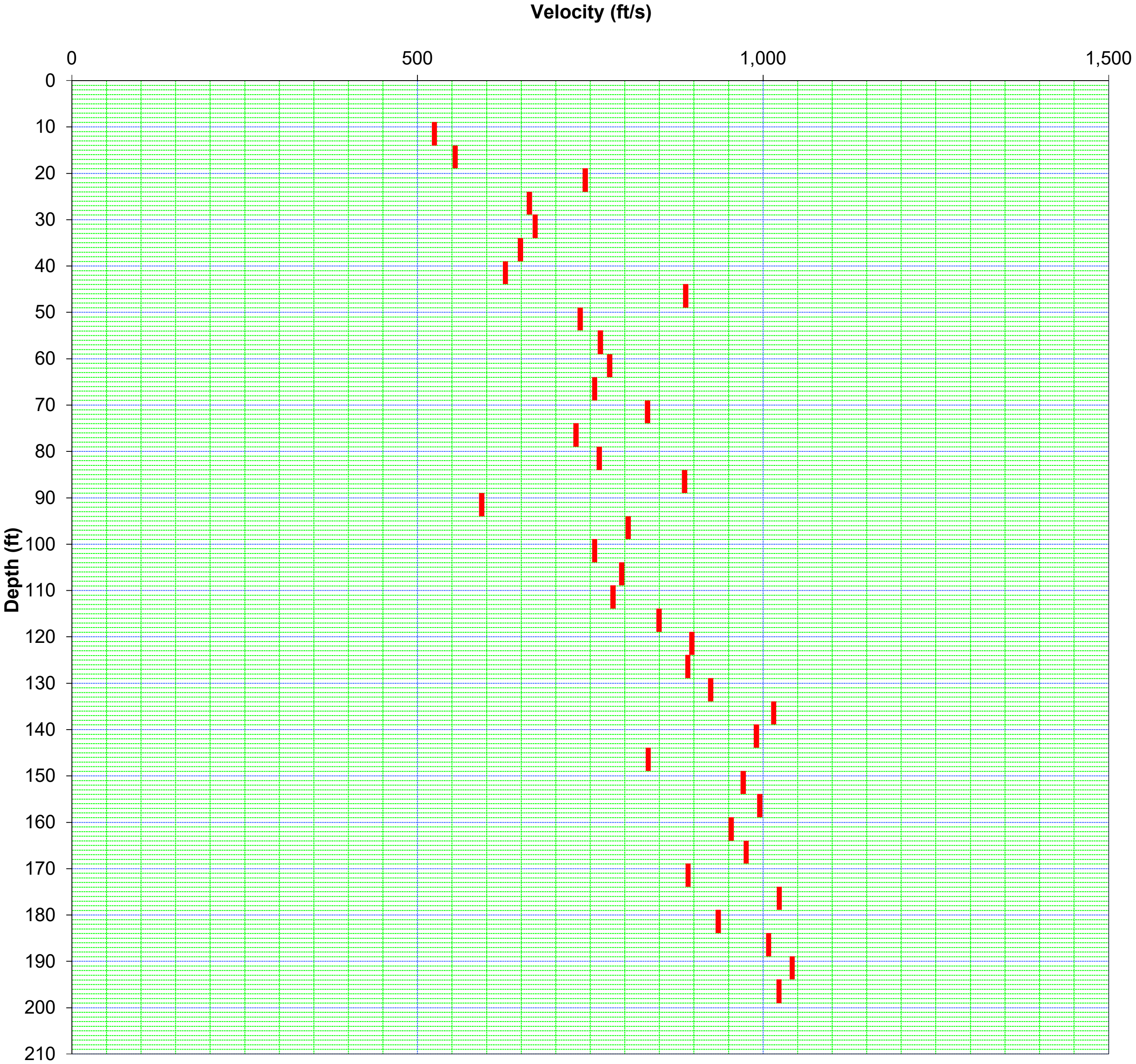
CPT-20-003

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
Bright People. Right Solutions.

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

KLEINFELDER
Bright People. Right Solutions.

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

KLEINFELDER
Bright People. Right Solutions.

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

KLEINFELDER
Bright People. Right Solutions.

Materials Finer than 75 um (No 200) Sieve

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

FIGURE

C-4

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

KLEINFELDER
Bright People. Right Solutions.

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

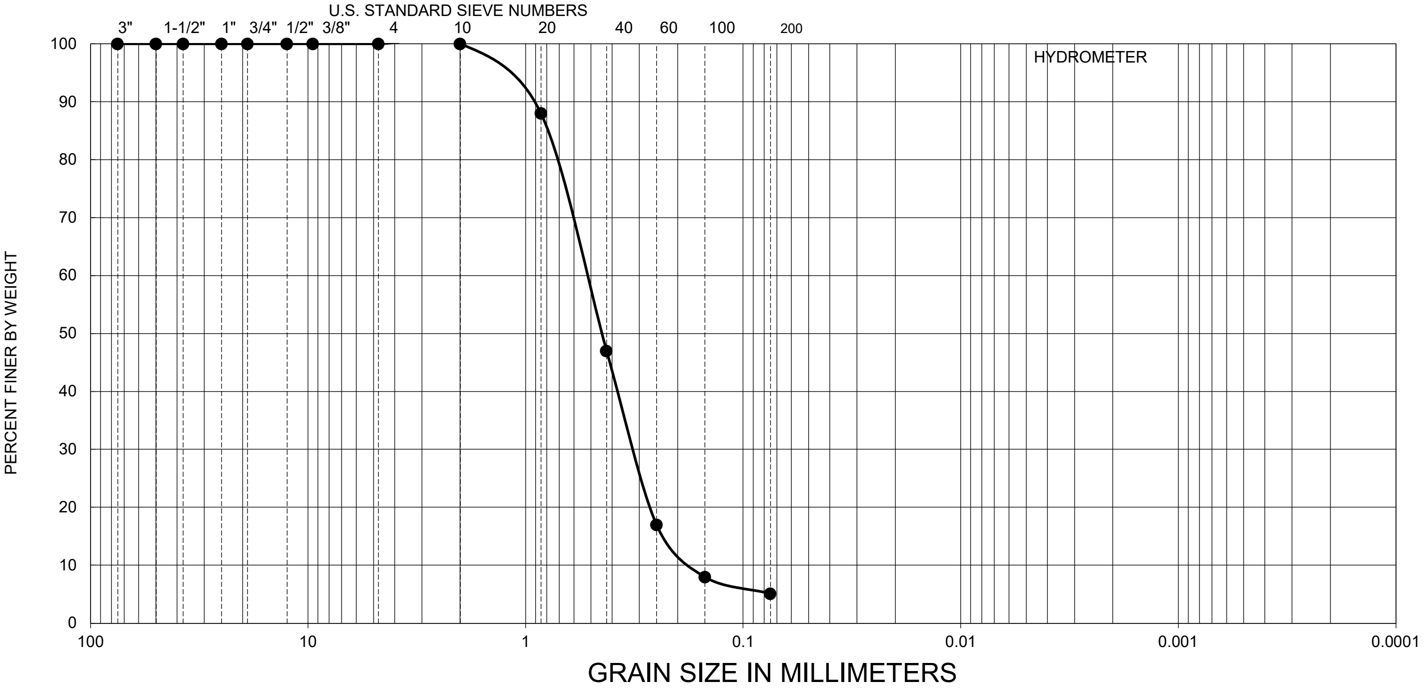
FIGURE

C-5

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

USCS

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



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

KLEINFELDER
Bright People. Right Solutions.

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

GRADATION TEST RESULTS

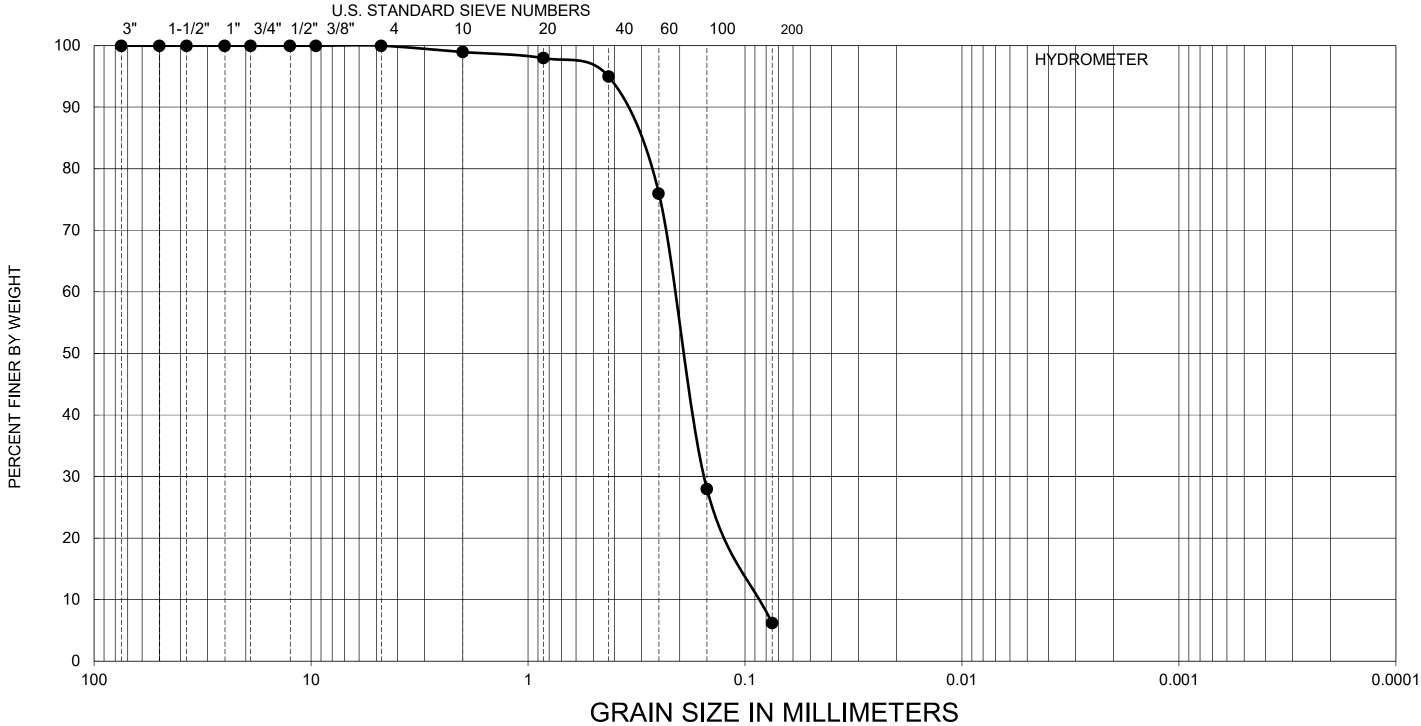
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

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

KLEINFELDER
Bright People. Right Solutions.

GRADATION TEST RESULTS

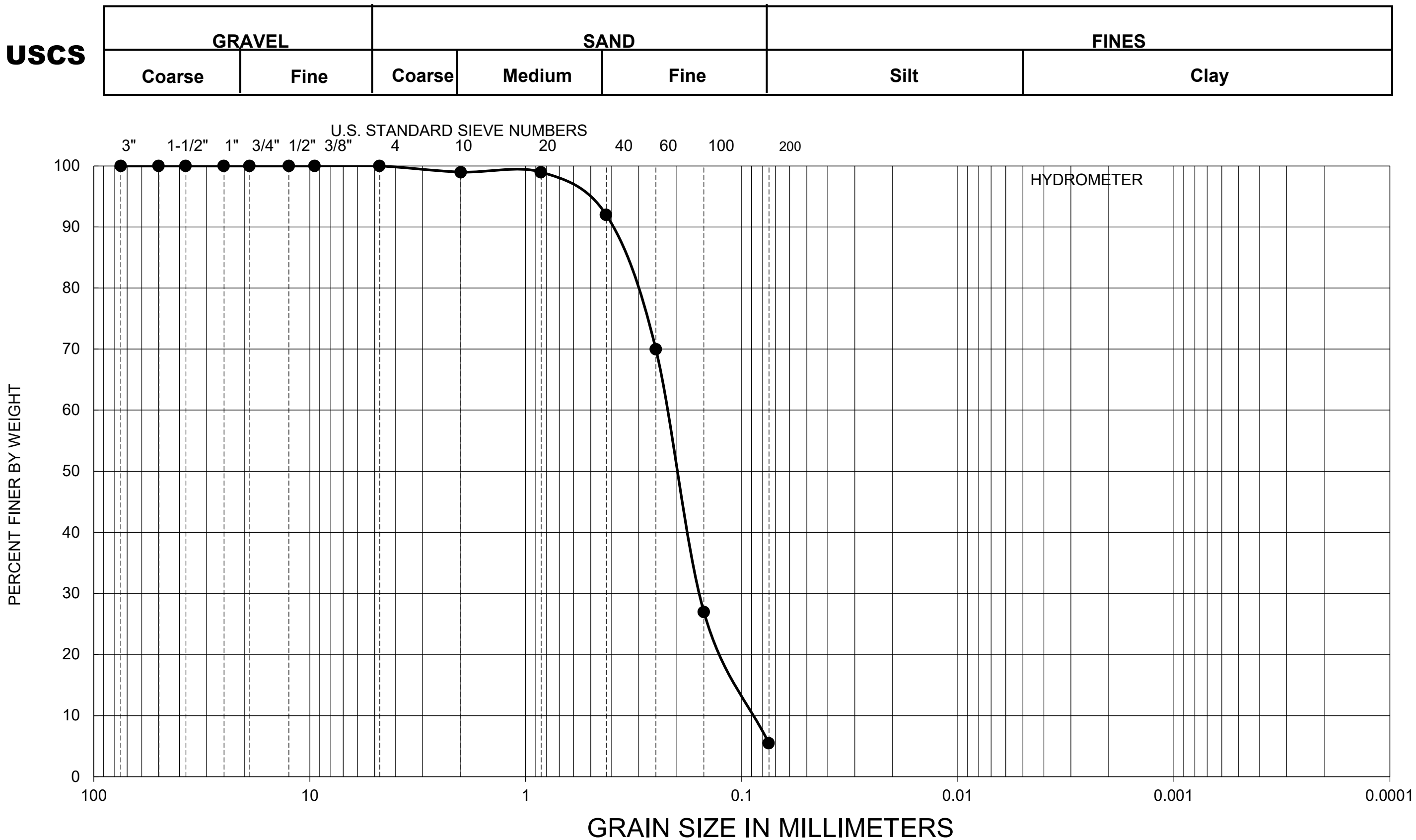
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-7

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

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

KLEINFELDER
Bright People. Right Solutions.

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

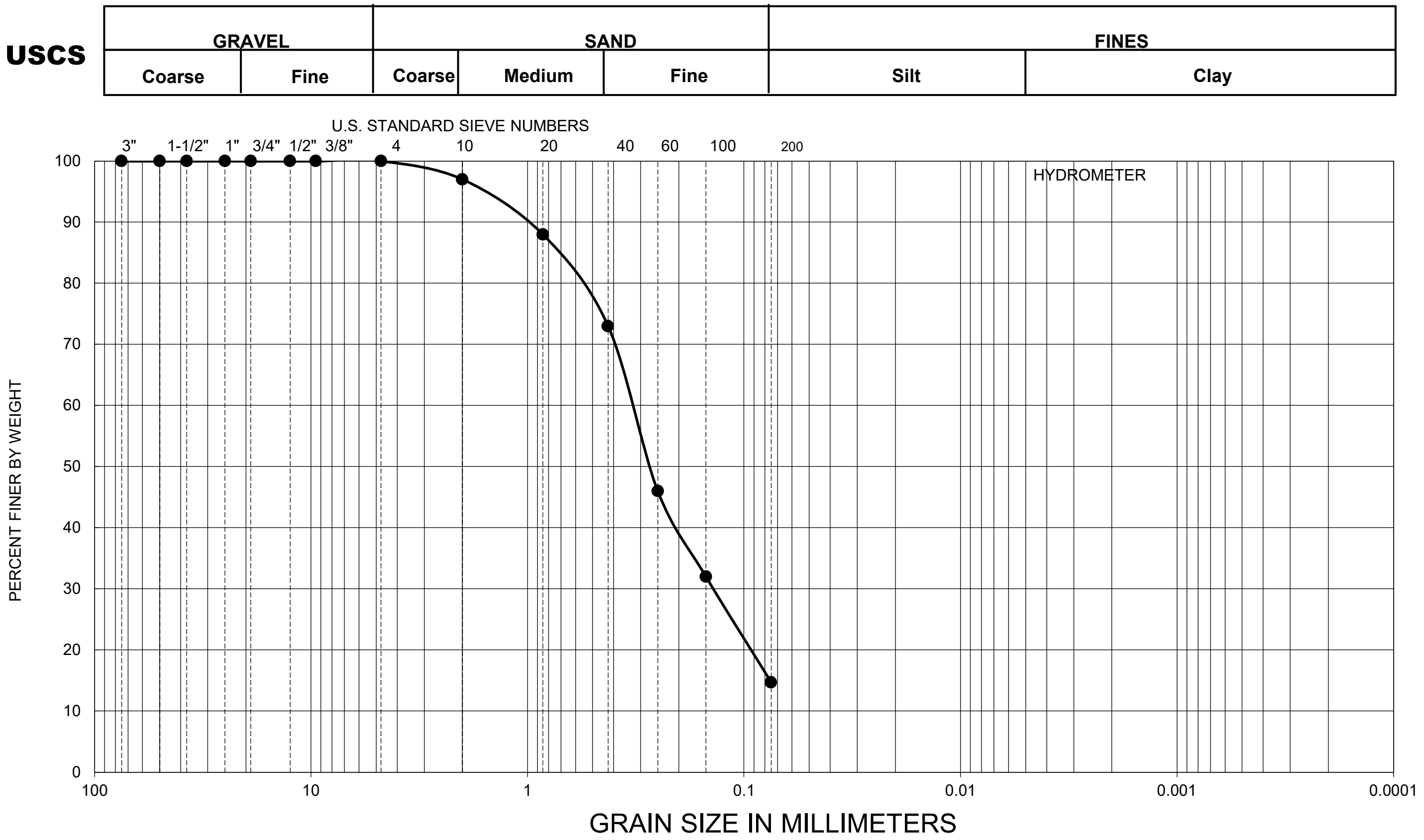
GRADATION TEST RESULTS

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

KLEINFELDER
Bright People. Right Solutions.

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

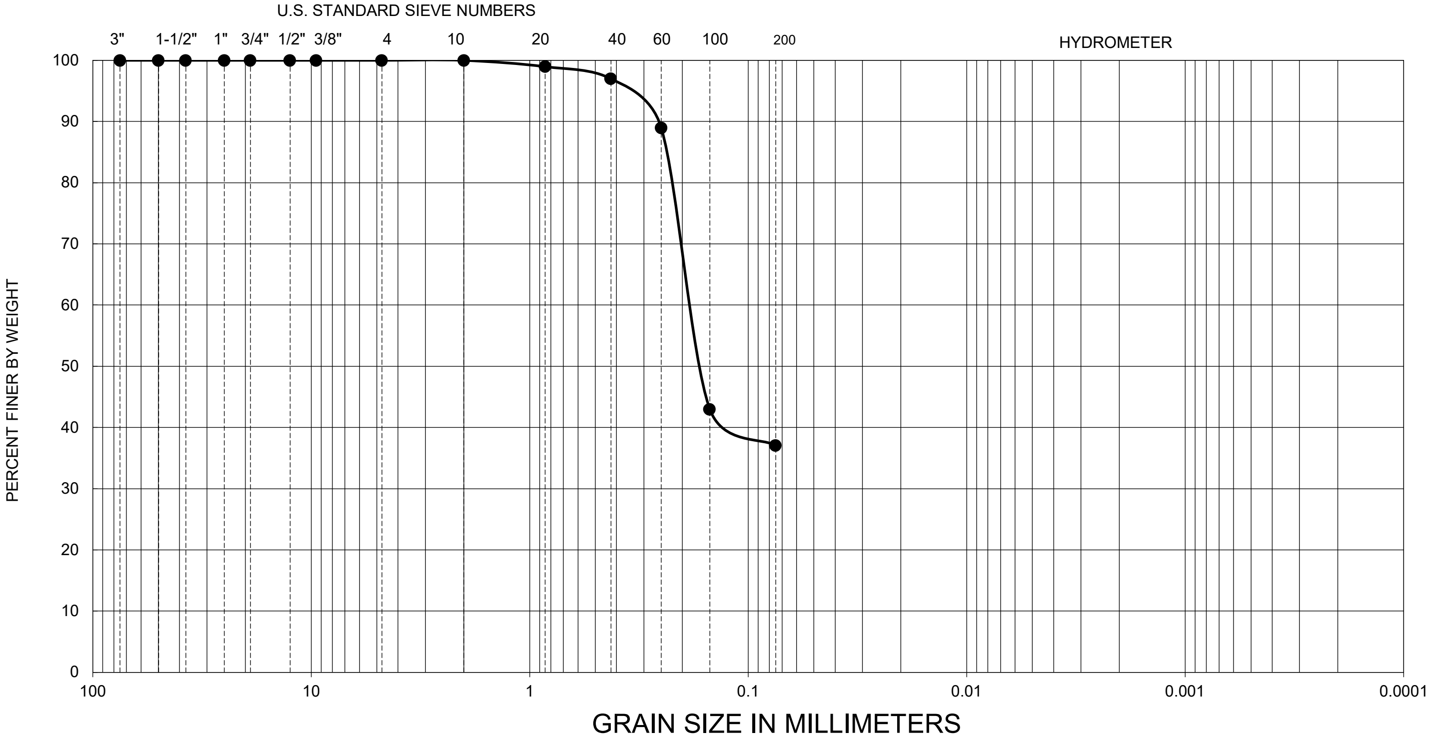
FIGURE

C-9

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

USCS

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



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

KLEINFELDER
Bright People. Right Solutions.

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

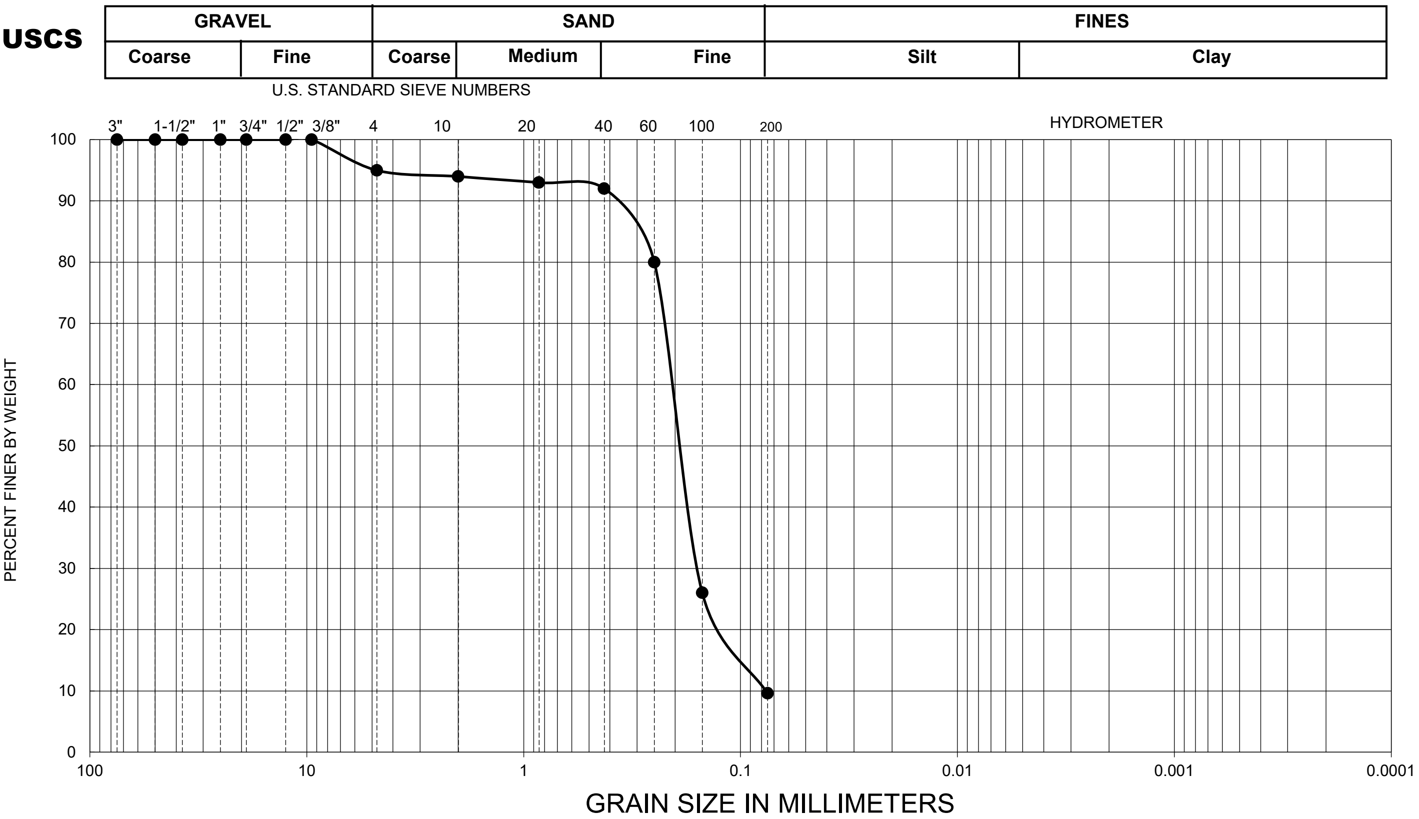
GRADATION TEST RESULTS

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

FIGURE

C-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

KLEINFELDER
Bright People. Right Solutions.

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

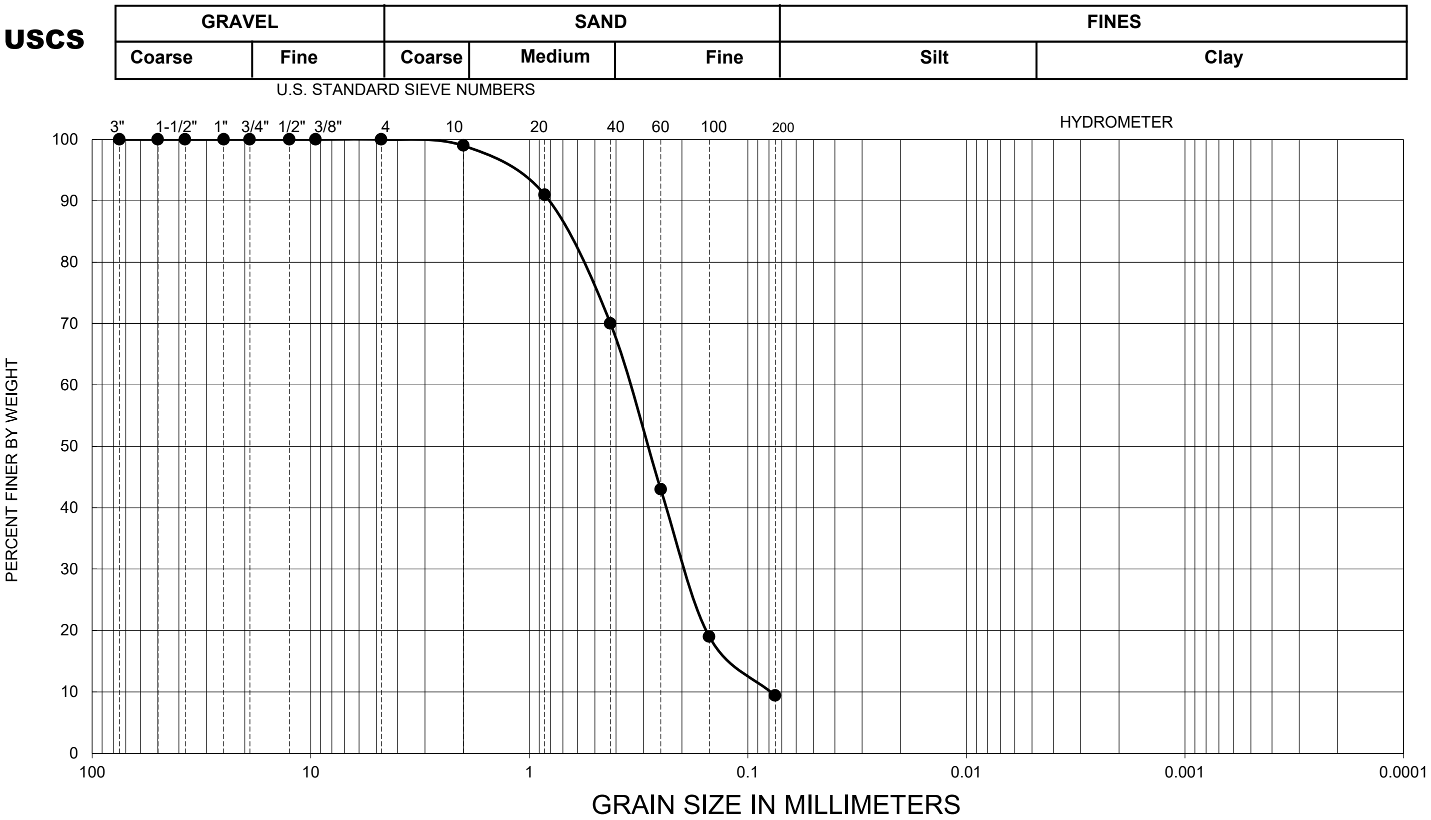
GRADATION TEST RESULTS

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

FIGURE

C-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

KLEINFELDER
Bright People. Right Solutions.

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

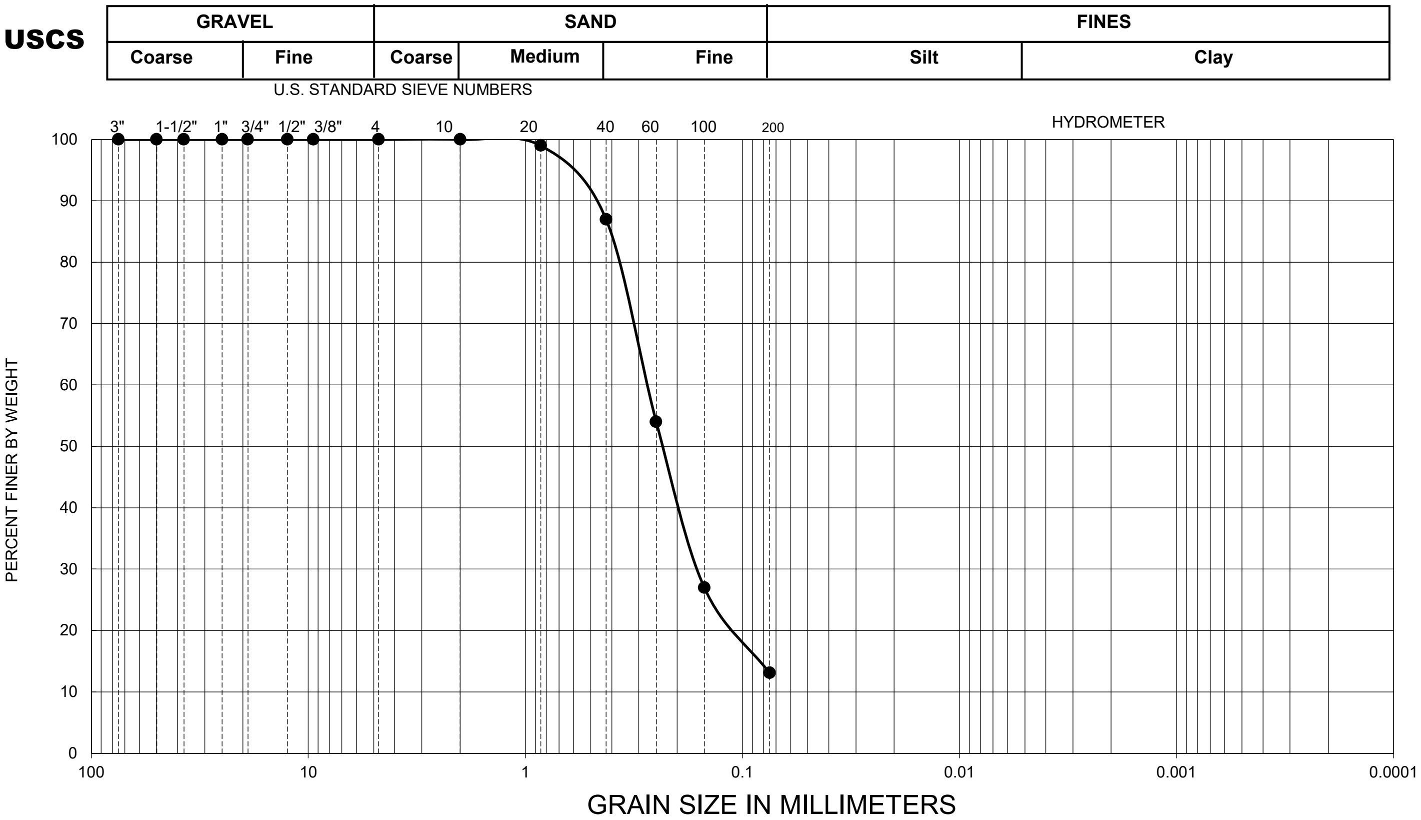
GRADATION TEST RESULTS

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

FIGURE

C-12

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



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

KLEINFELDER
Bright People. Right Solutions.

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

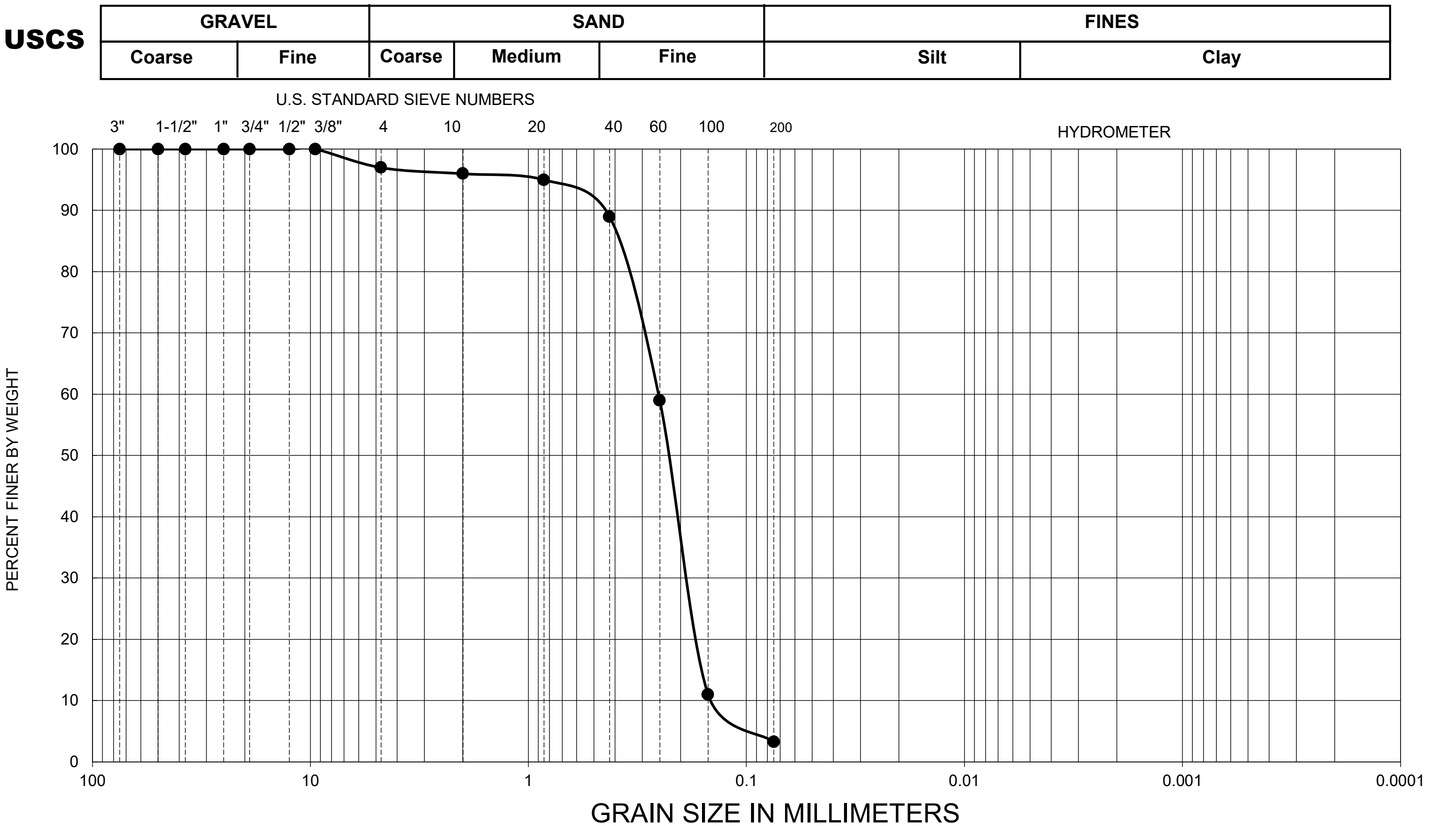
GRADATION TEST RESULTS

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

KLEINFELDER
Bright People. Right Solutions.

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

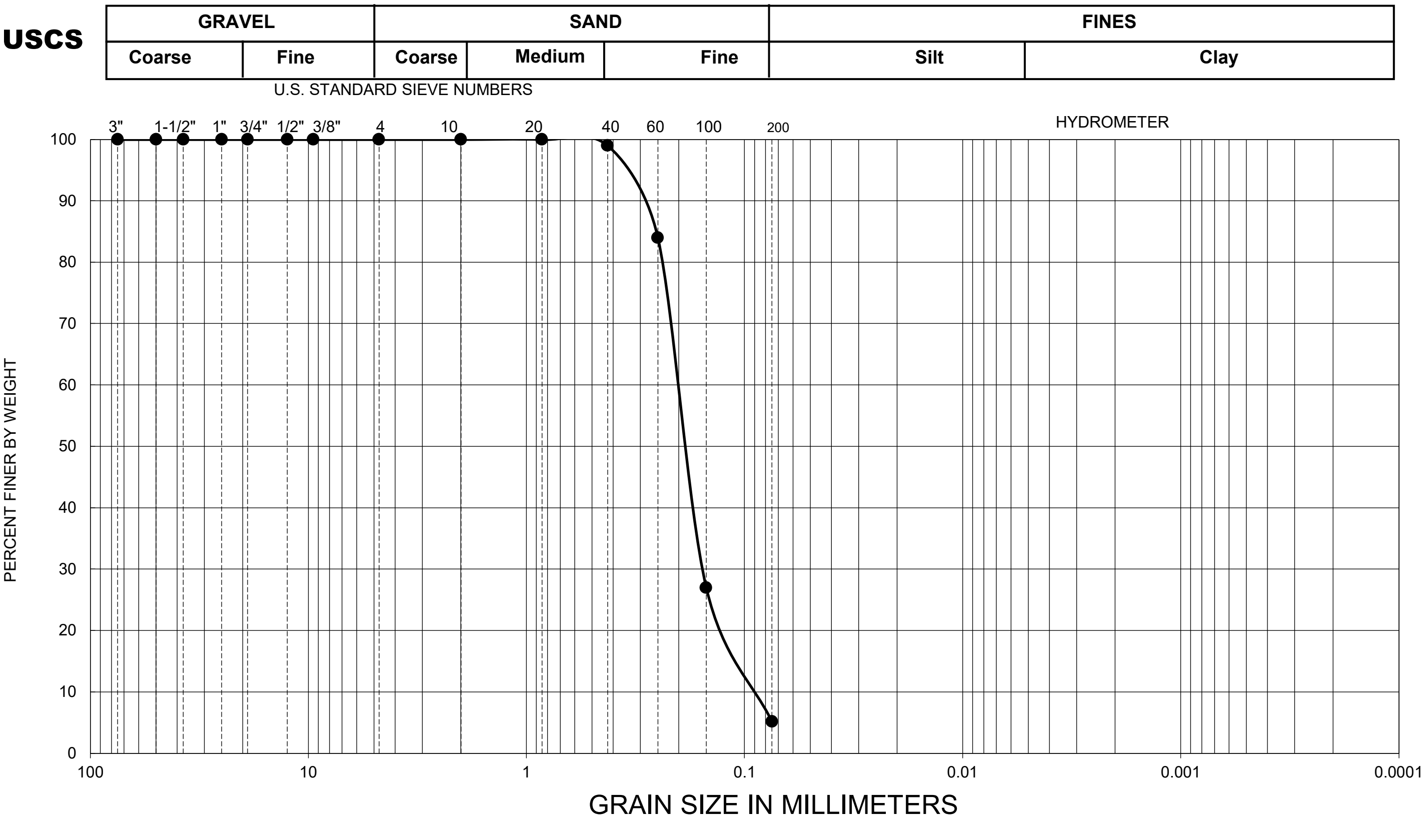
GRADATION TEST RESULTS

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

FIGURE

C-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

KLEINFELDER
Bright People. Right Solutions.

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

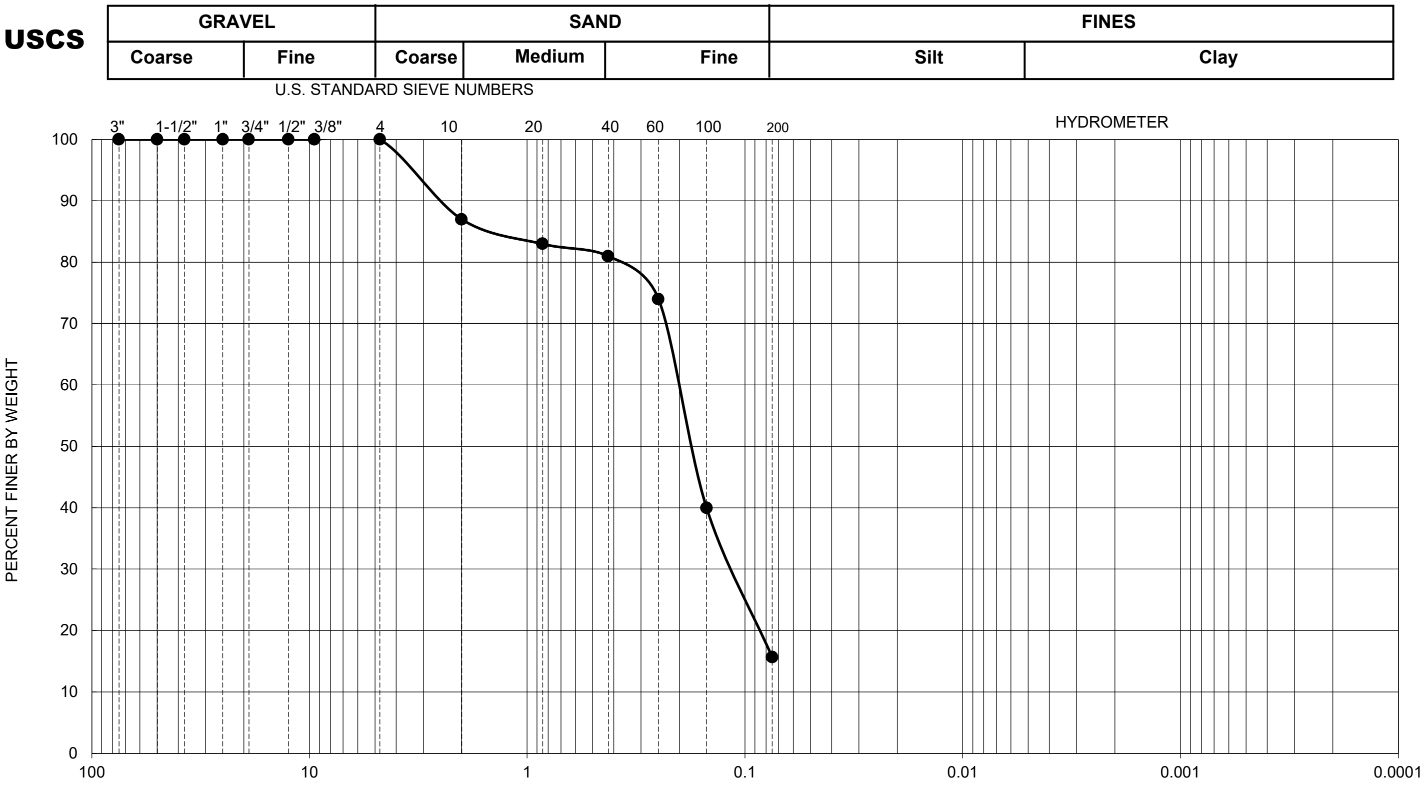
GRADATION TEST RESULTS

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

FIGURE

C-15

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



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

KLEINFELDER
Bright People. Right Solutions.

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

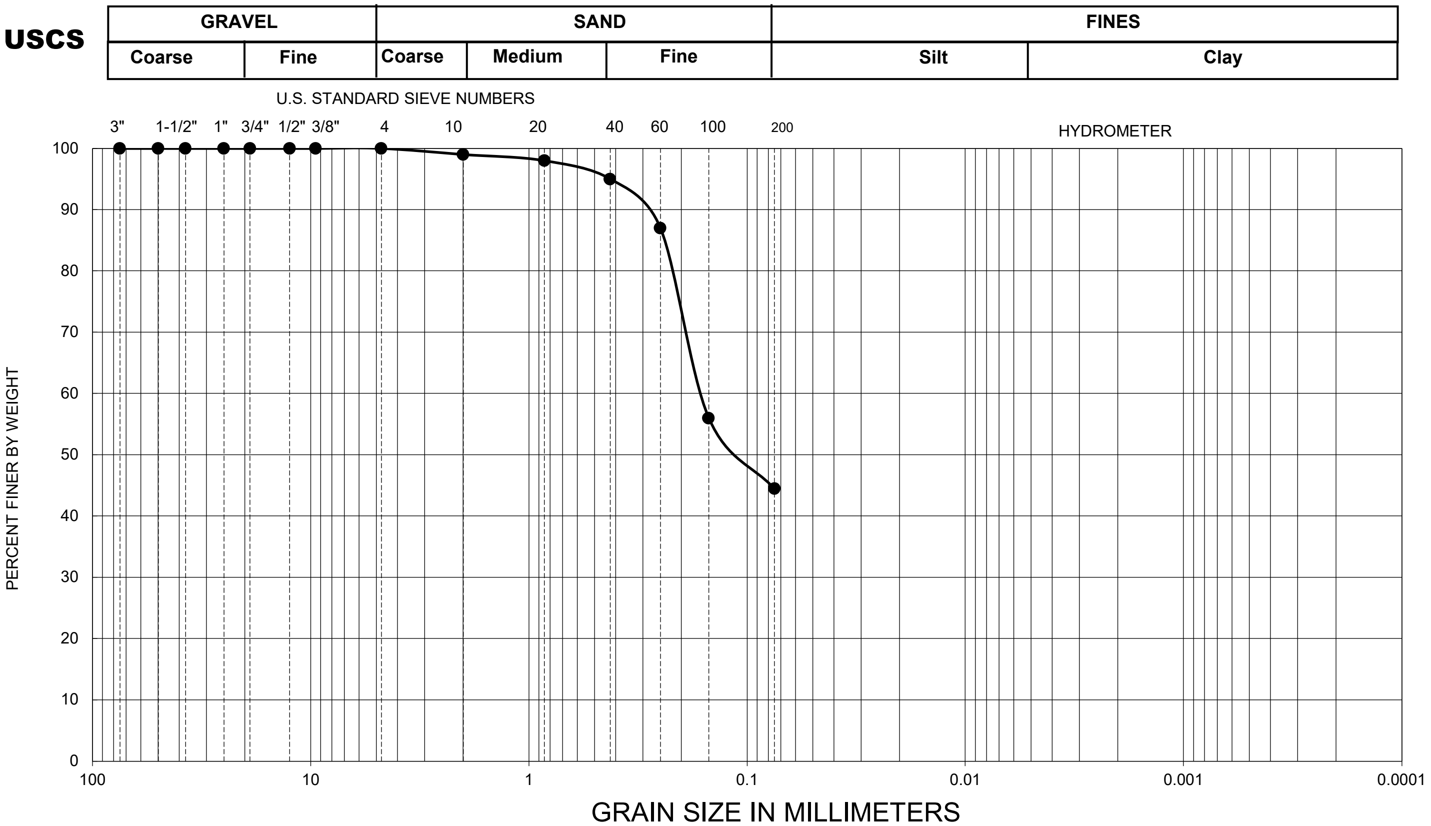
GRADATION TEST RESULTS

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

FIGURE

C-16

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



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

KLEINFELDER
Bright People. Right Solutions.

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

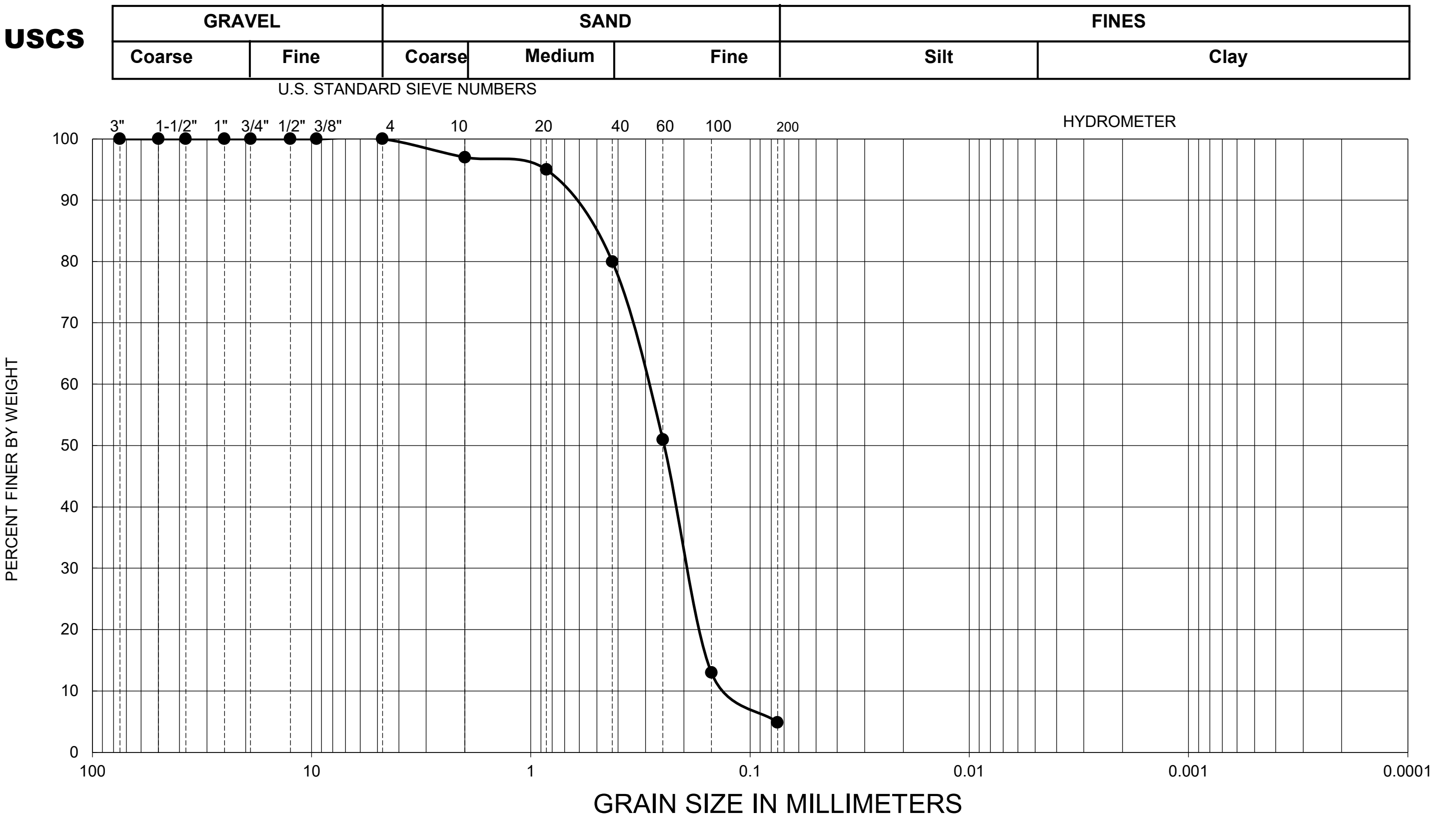
GRADATION TEST RESULTS

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

KLEINFELDER
Bright People. Right Solutions.

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

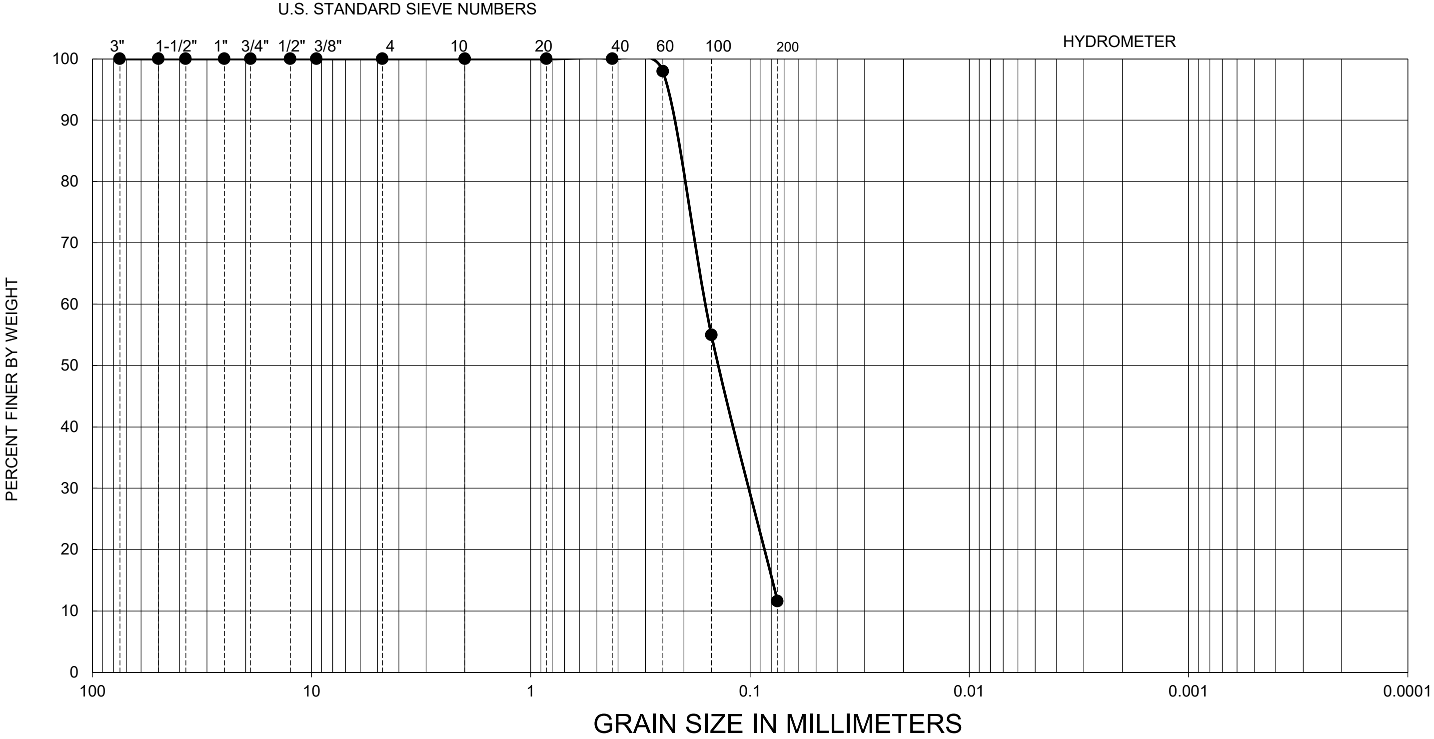
FIGURE

C-18

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

USCS

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



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

KLEINFELDER
Bright People. Right Solutions.

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

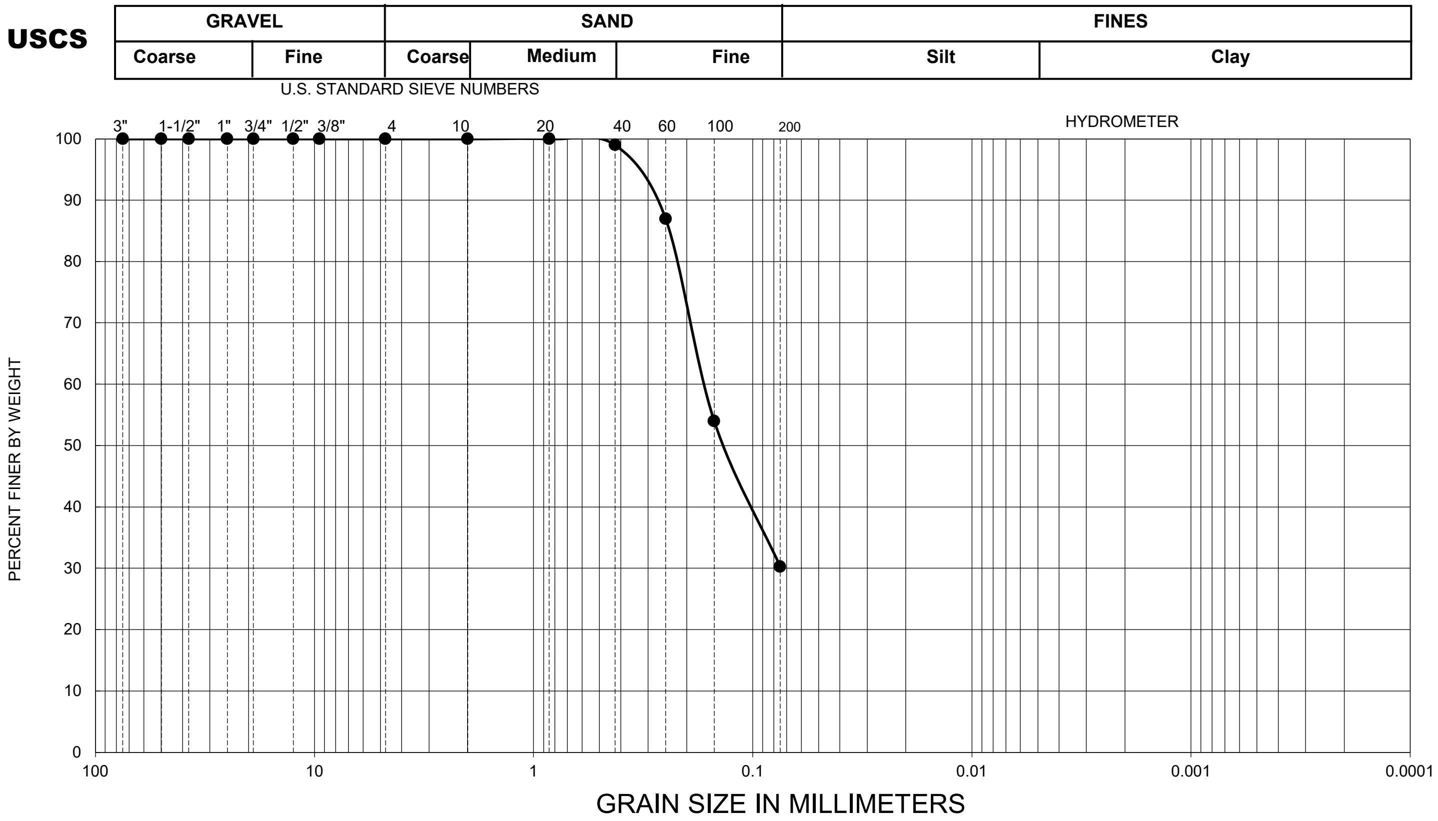
GRADATION TEST RESULTS

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

KLEINFELDER

Bright People. Right Solutions.

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

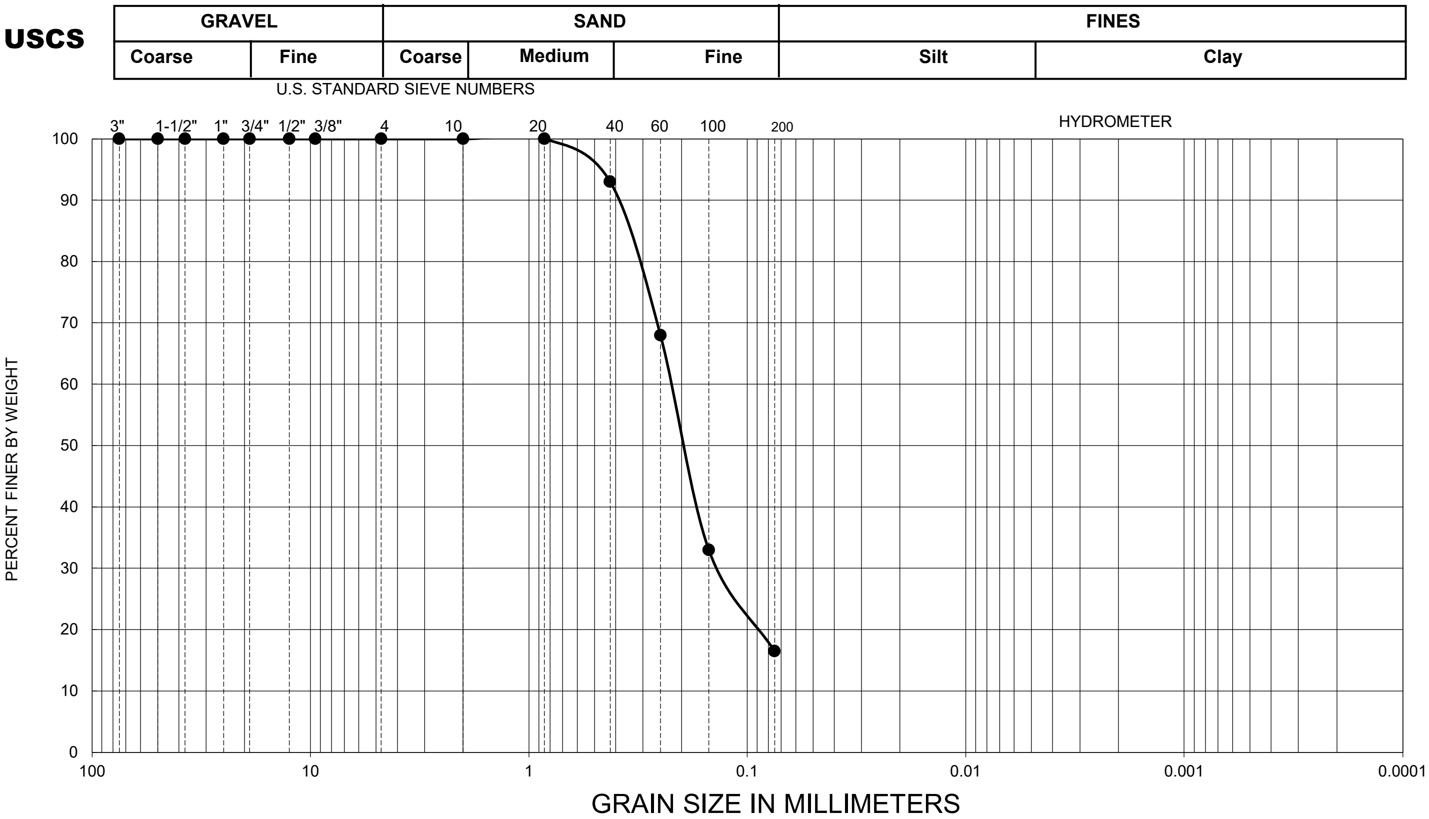
GRADATION TEST RESULTS

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

KLEINFELDER
Bright People. Right Solutions.

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

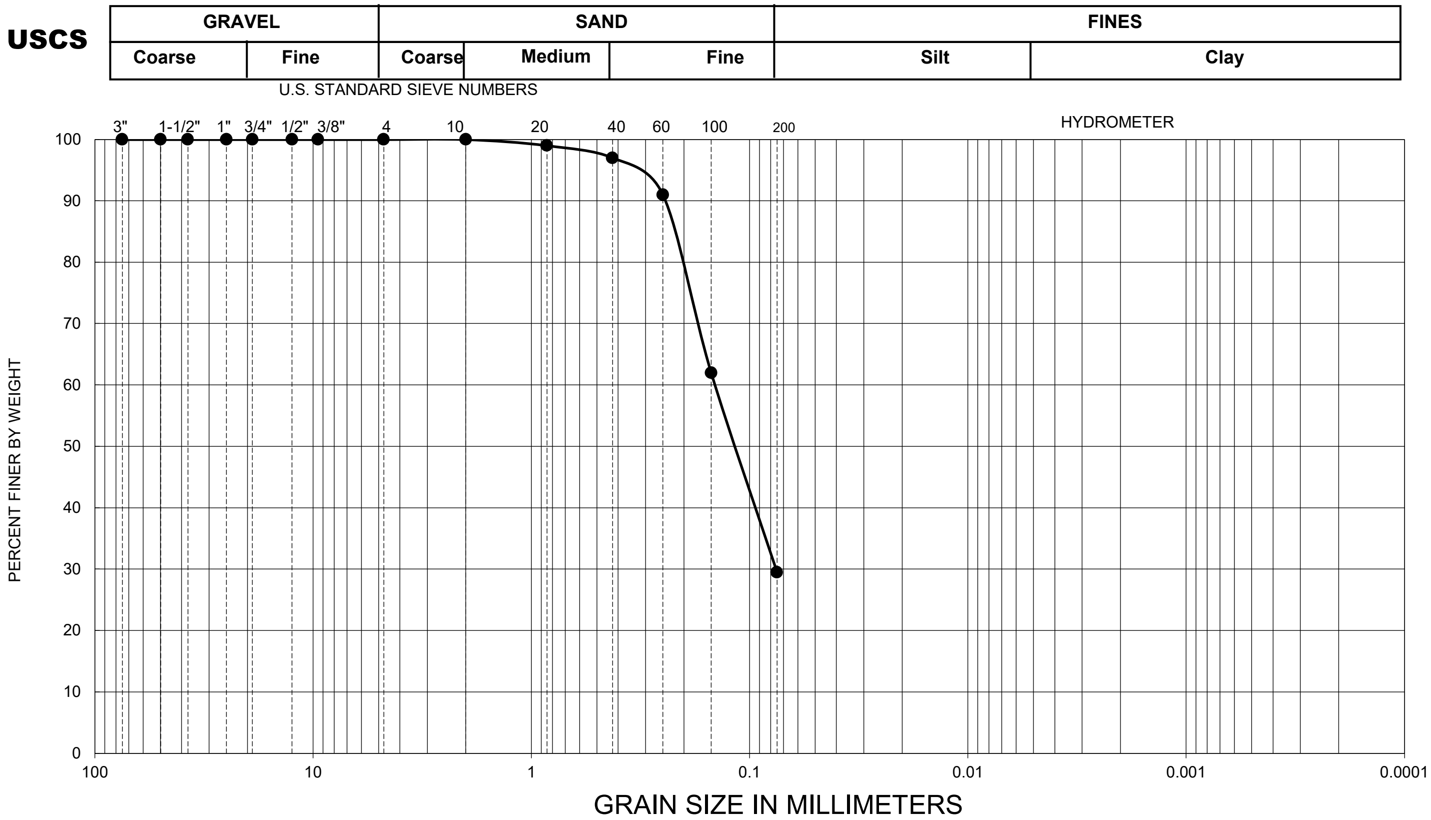
GRADATION TEST RESULTS

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

FIGURE

C-21

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

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

KLEINFELDER
Bright People. Right Solutions.

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

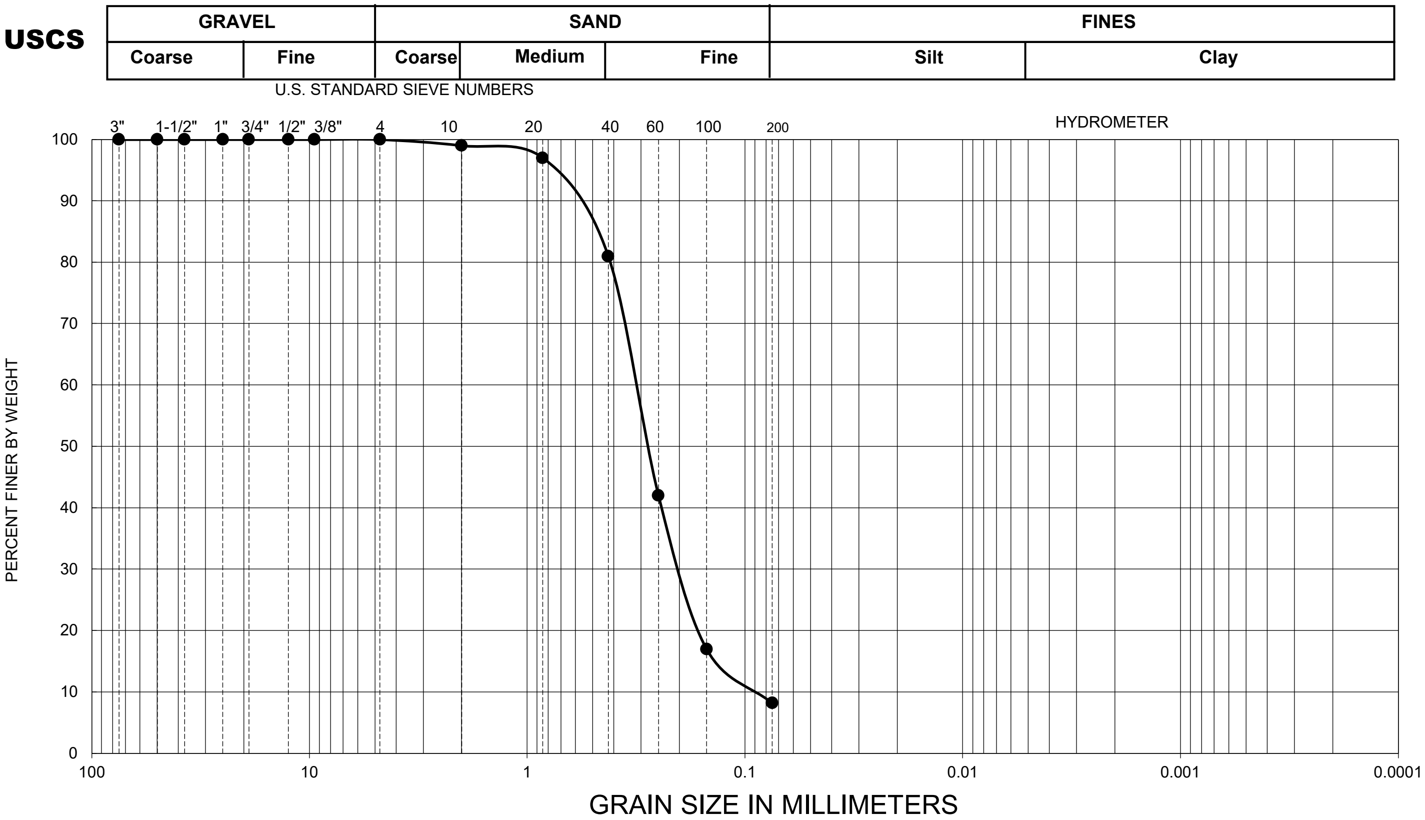
GRADATION TEST RESULTS

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

FIGURE

C-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

KLEINFELDER
Bright People. Right Solutions.

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

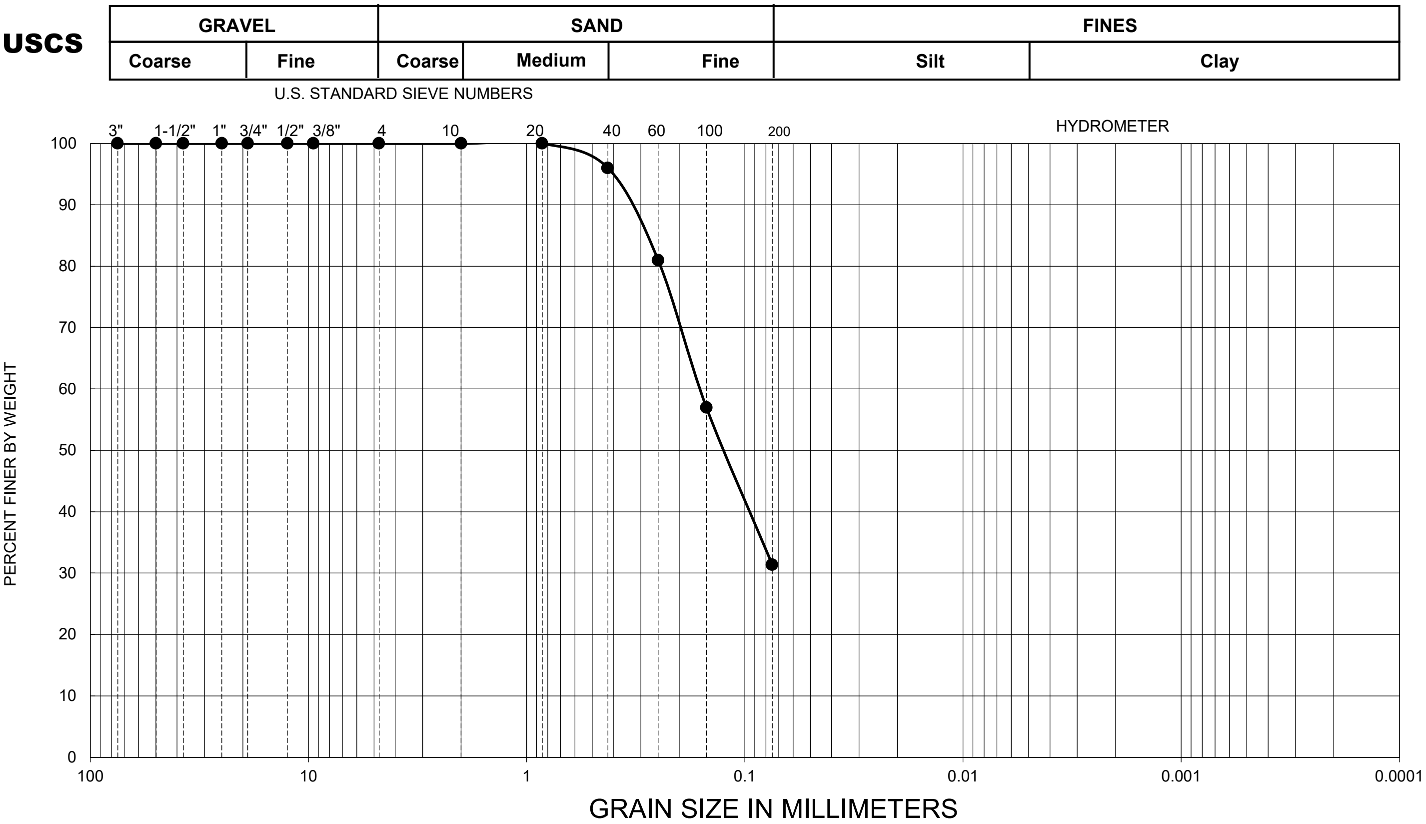
GRADATION TEST RESULTS

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

KLEINFELDER
Bright People. Right Solutions.

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

GRADATION TEST RESULTS

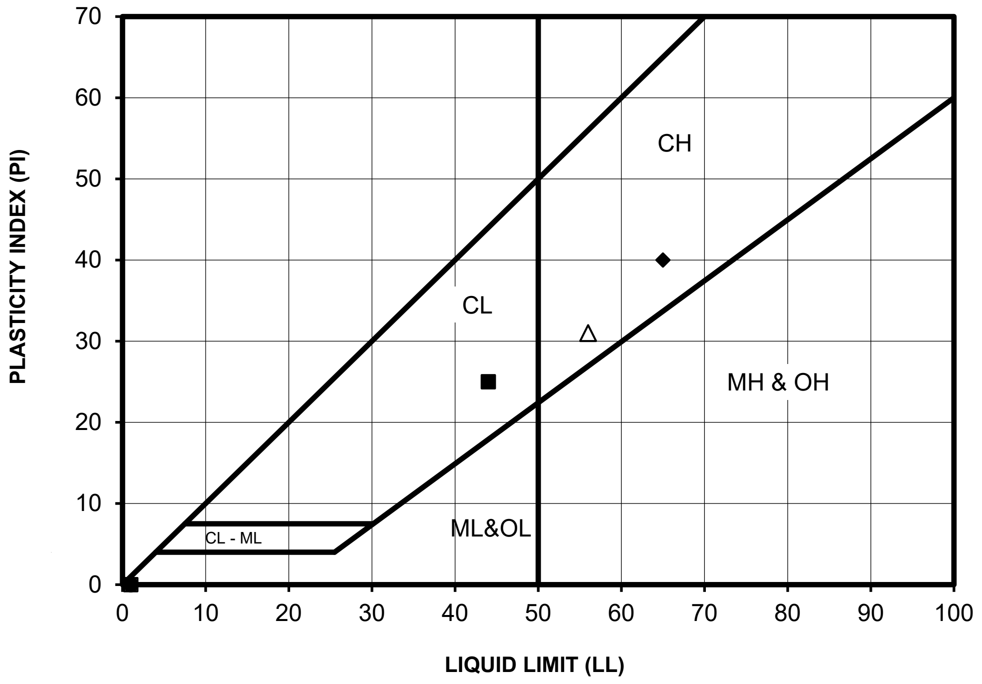
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

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



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.

KLEINFELDER
Bright People. Right Solutions.

ATTERBERG LIMITS TEST RESULTS

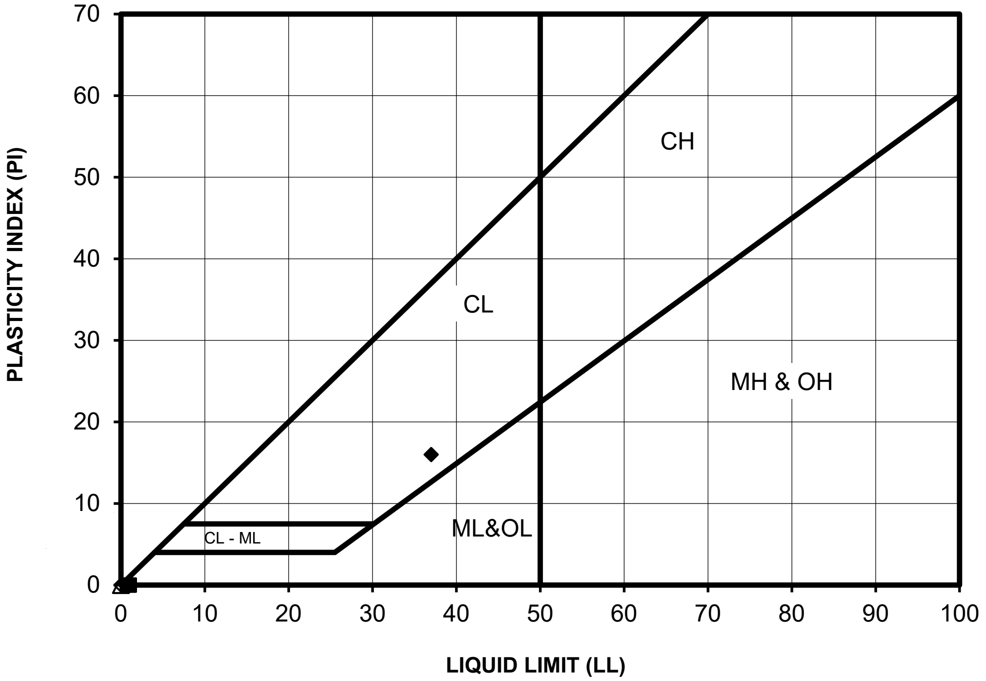
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE
C-25

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.

KLEINFELDER
Bright People. Right Solutions.

ATTERBERG LIMITS TEST RESULTS

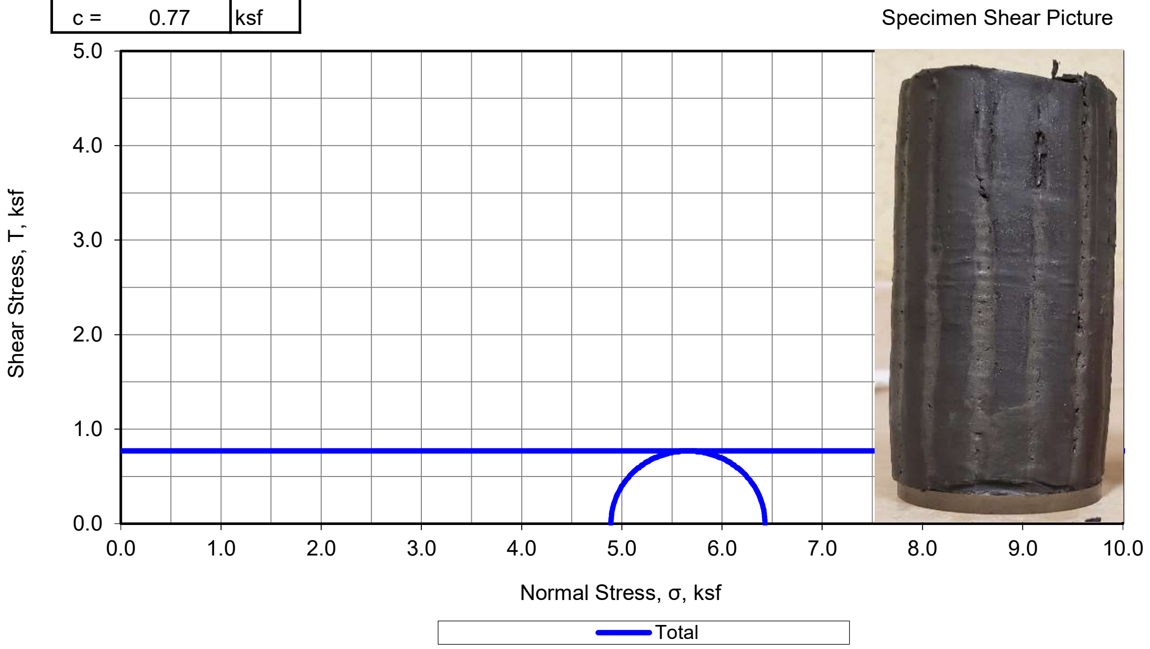
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

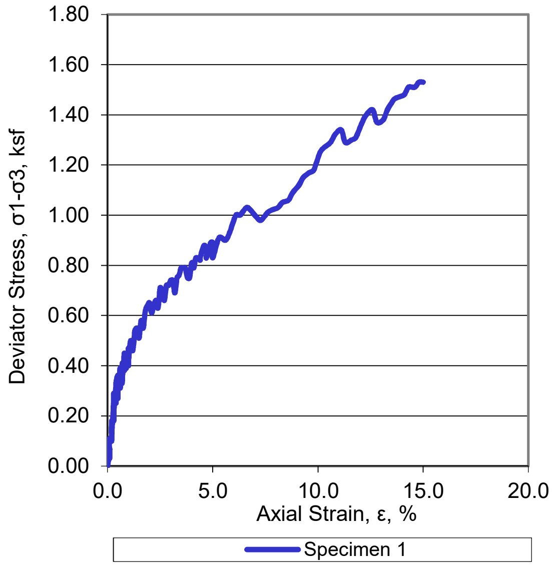
FIGURE

C-26

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

Total	
c = 0.77	ksf





Specimen 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 $(\sigma_1\text{-}\sigma_3)_{max}$, 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

KLEINFELDER
Bright People. Right Solutions.
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)

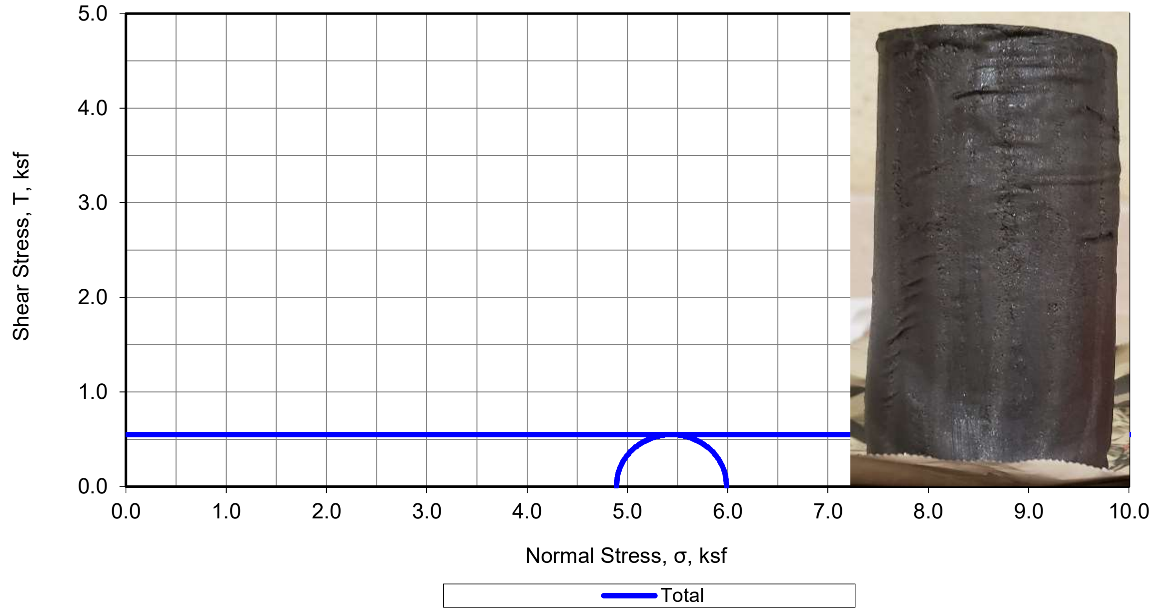
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

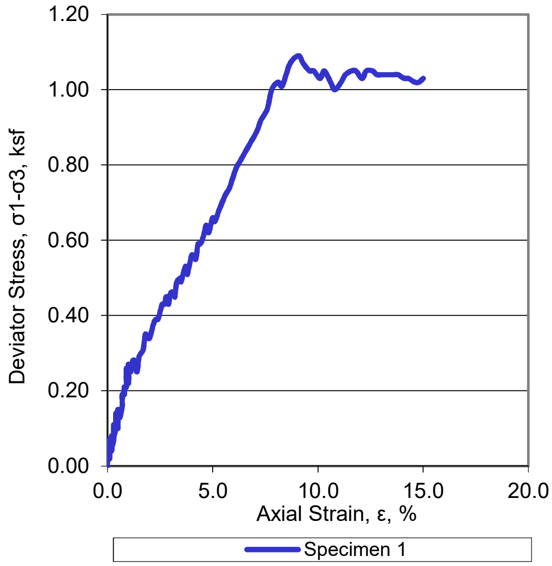
Figure

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	γd_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 $(\sigma_1\text{-}\sigma_3)_{max}$, 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

KLEINFELDER
Bright People. Right Solutions.
2601 Barrington Ct, Hayward, CA 94545

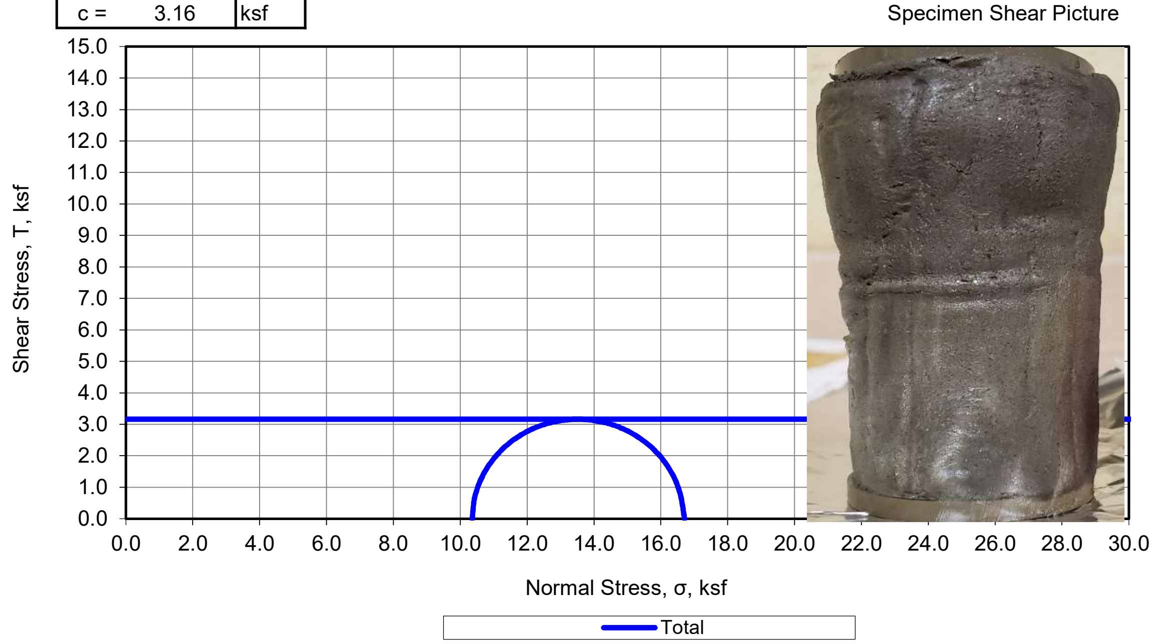
Project No.:	20180876.001A
Date:	3/20/20
Entry By:	CP
Checked By:	CP
File Name:	HL12966

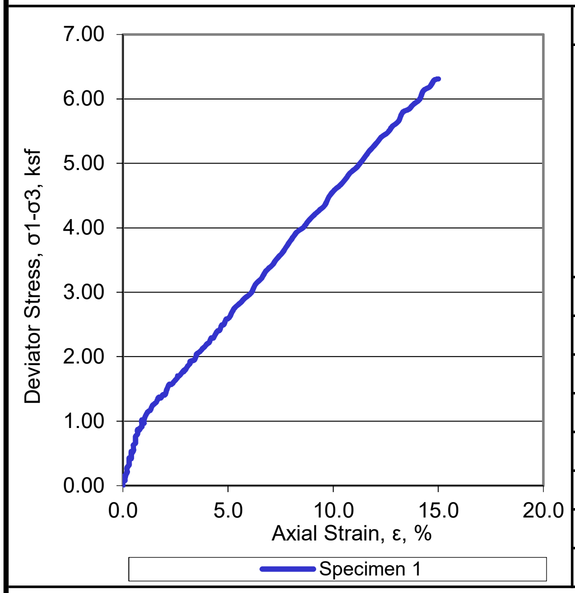
TRIAXIAL COMPRESSION TEST (UU)

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

Figure
C-28

Total	
c = 3.16	ksf





Specimen 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 $(\sigma_1\text{-}\sigma_3)_{max}$, 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

KLEINFELDER
Bright People. Right Solutions.
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)

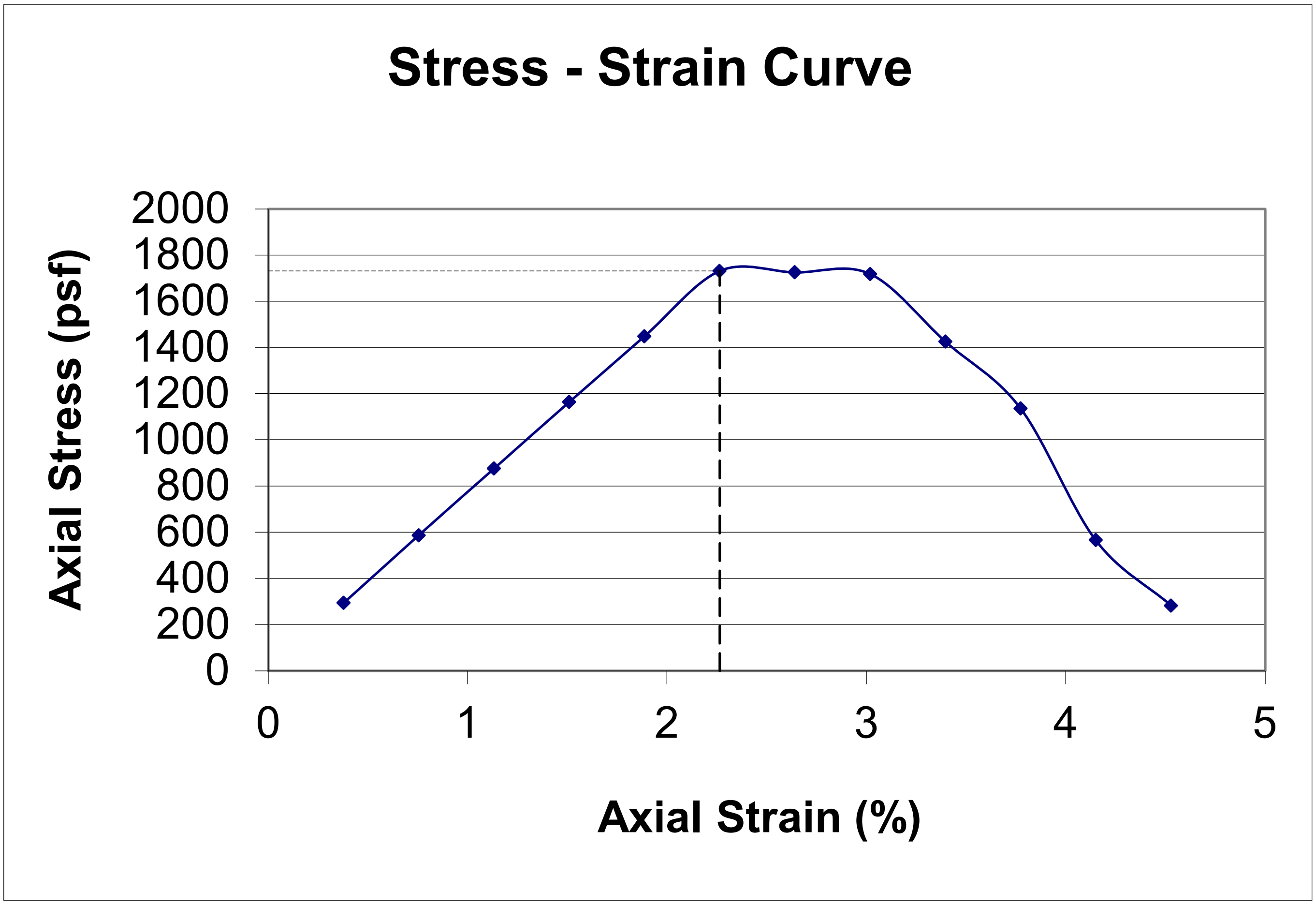
**Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California**

Figure

C-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

KLEINFELDER
Bright People. Right Solutions.

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

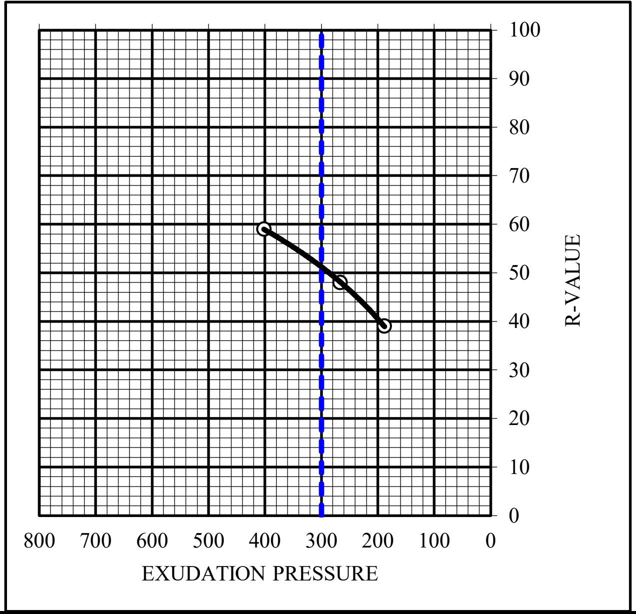
C-30

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:	



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.

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

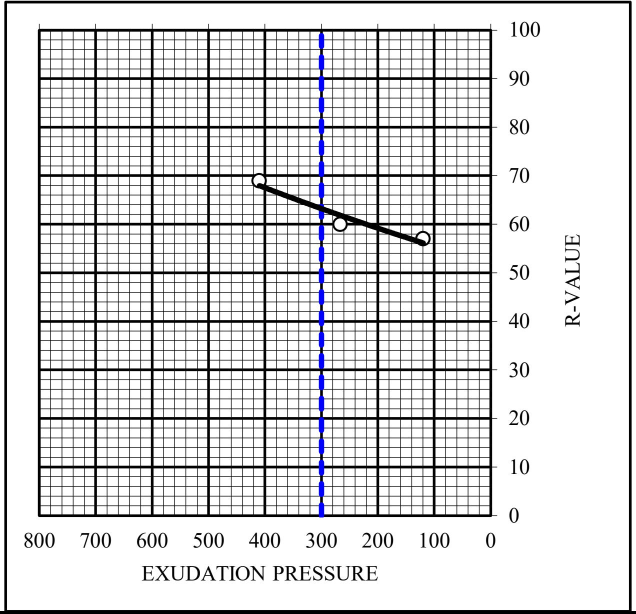
C-31

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:	



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.

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.

Corrosion Testing

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

FIGURE
C-33

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

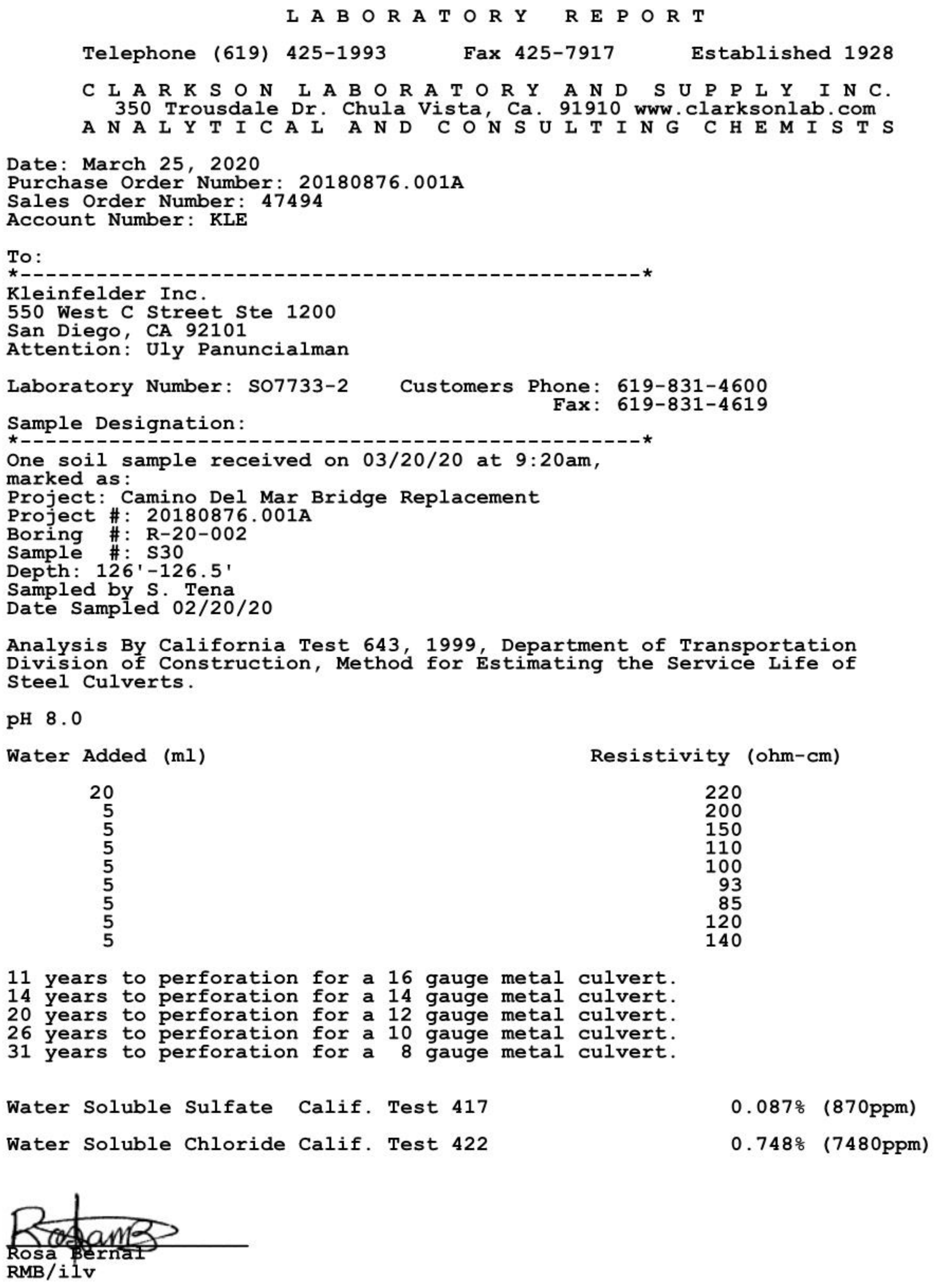
KLEINFELDER
Bright People. Right Solutions.

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-35



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)

Rosa Bernal
RMB/ilv

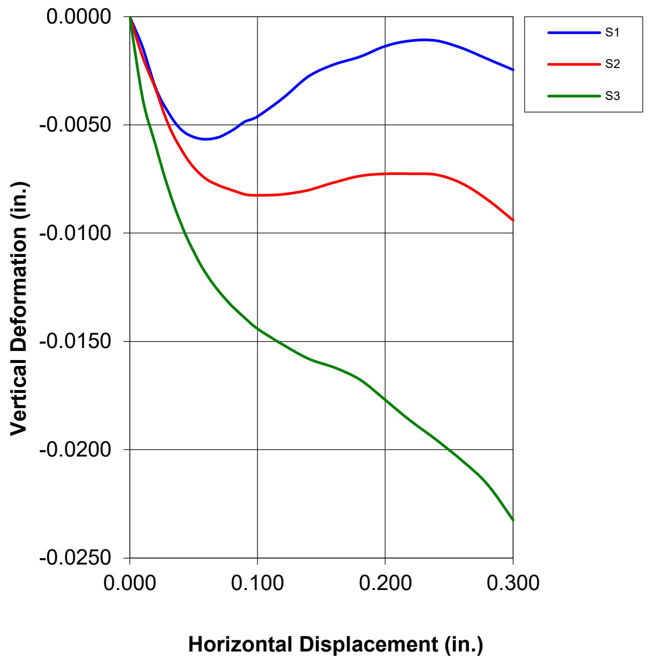
KLEINFELDER
Bright People. Right Solutions.

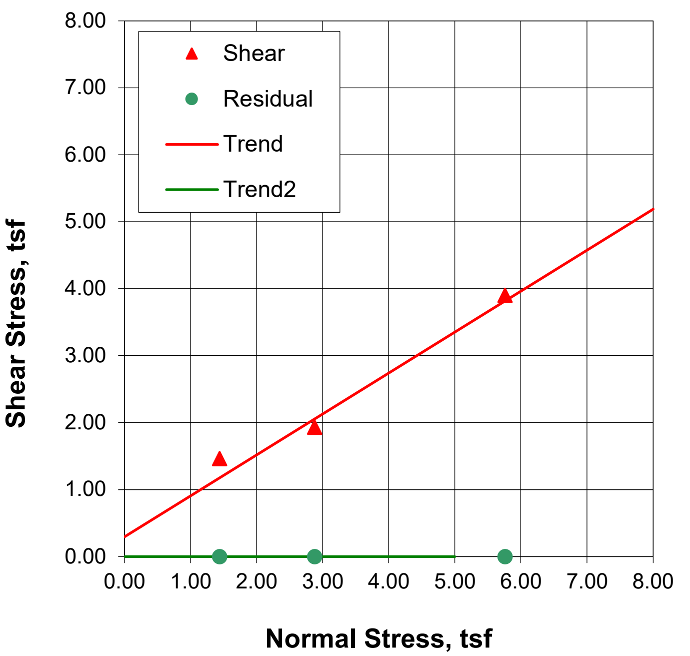
CHECKED BY: J.B.
TECH: Clarkson Lab
JOB NUMBER: 20180876.001A
DATE: 13-May-20

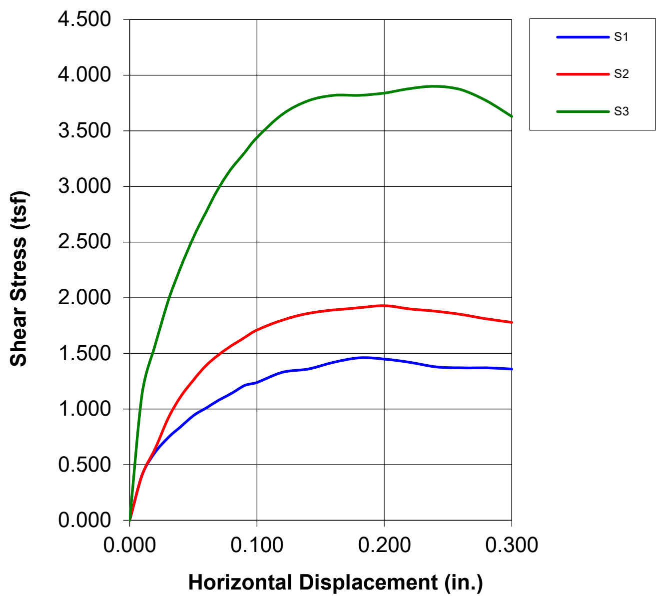
Corrosion Testing

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

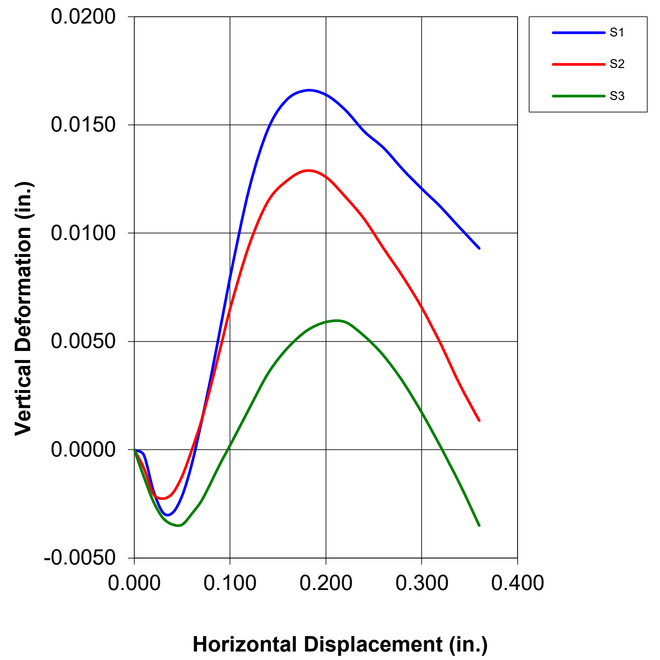
PROJECT NO.:	20180876
ENTRY BY:	A. Wohletz
CHECKED BY:	S. Rader
DATE:	3/23/2020

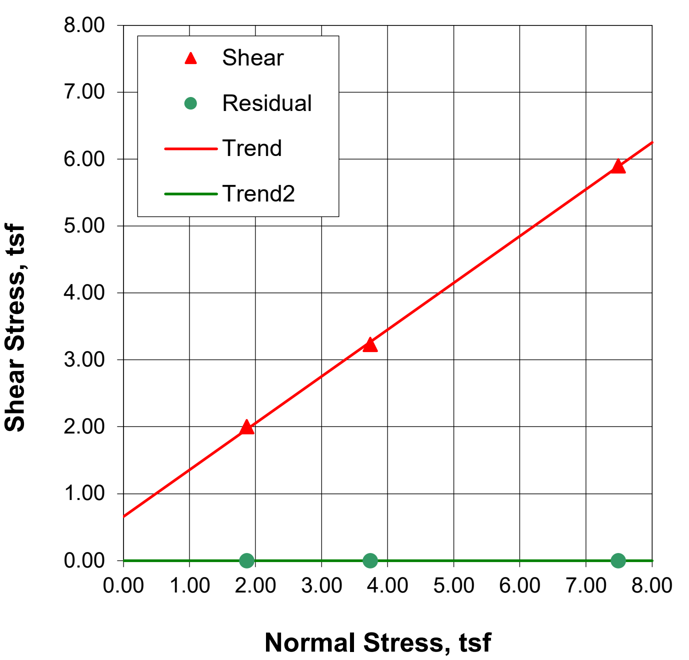
DIRECT SHEAR TEST ASTM D3080

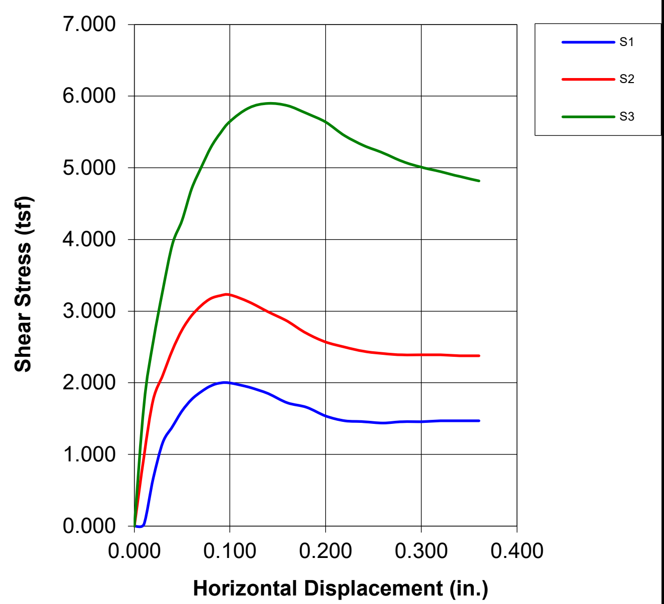
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

FIGURE

C-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

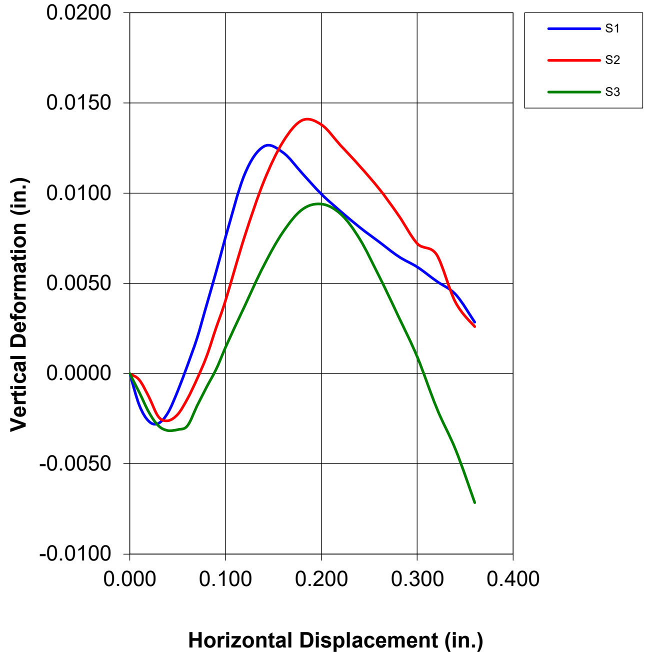
PROJECT NO.:	20180876
ENTRY BY:	A. Wohletz
CHECKED BY:	S. Rader
DATE:	3/23/2020

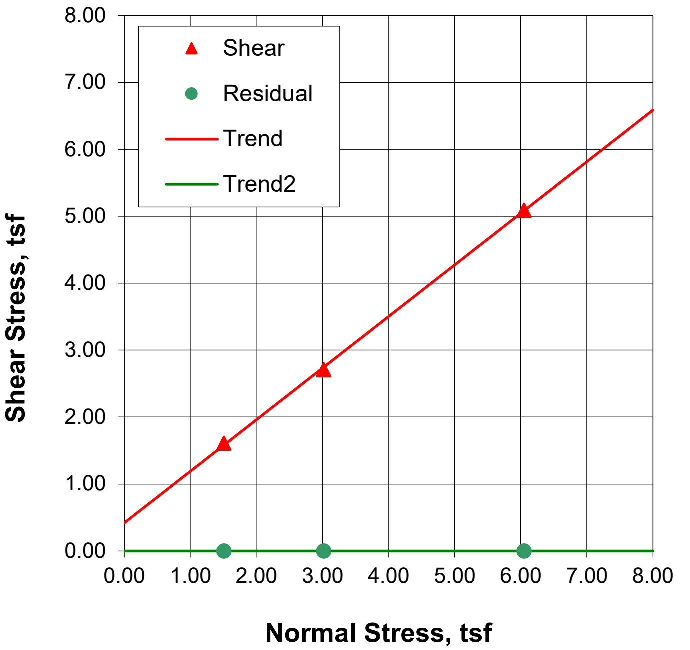
DIRECT SHEAR TEST ASTM D3080

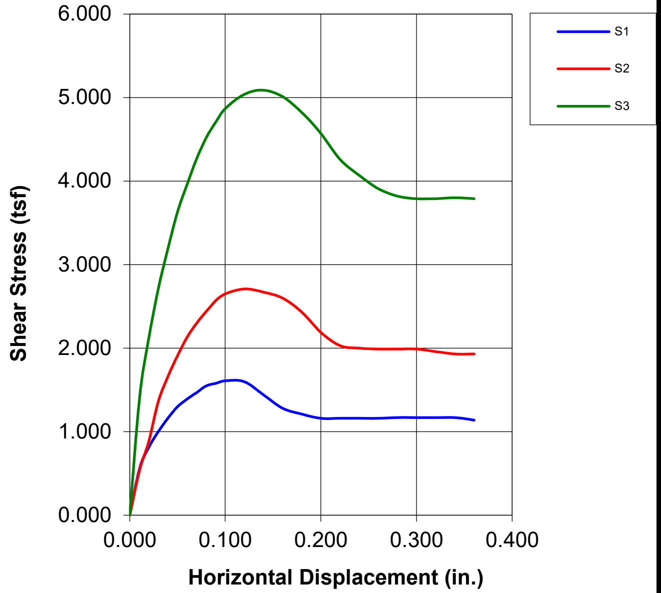
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

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

	c, tsf	ϕ, deg.	Tan ϕ
Failure	0.4	37.6	0.77
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:	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

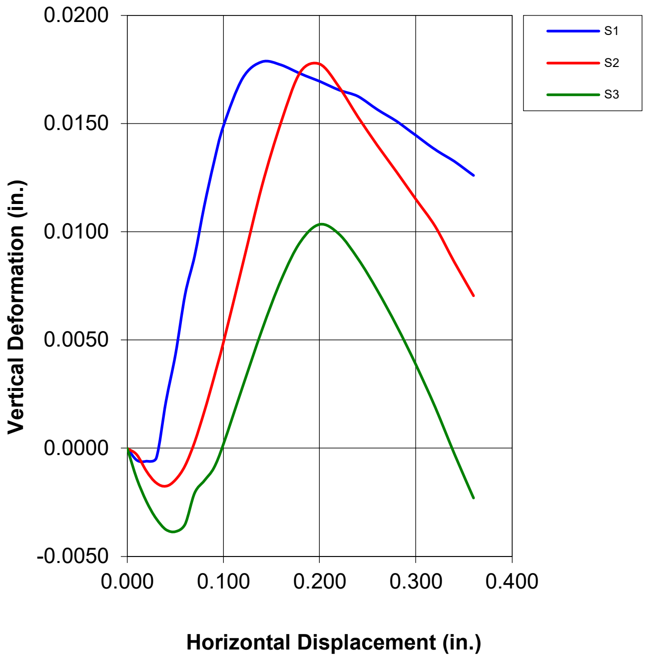
PROJECT NO.:	20180876
ENTRY BY:	A. Wohletz
CHECKED BY:	S. Rader
DATE:	3/25/2020

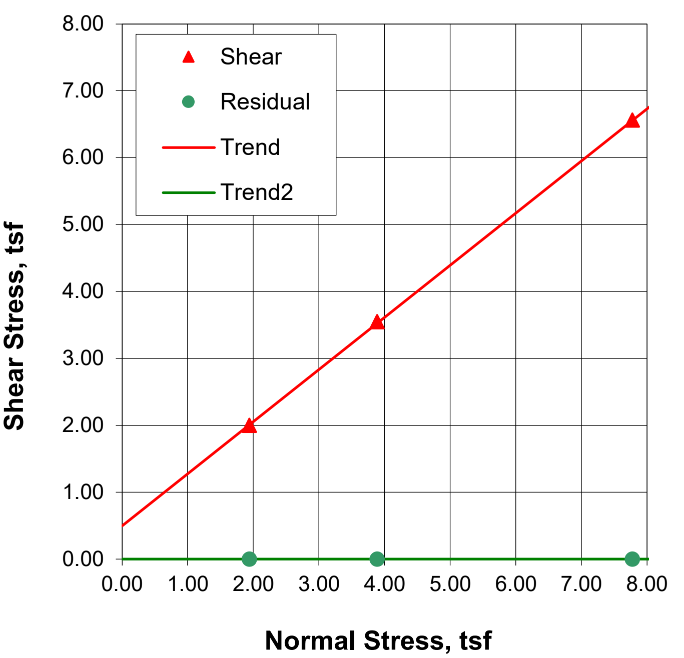
DIRECT SHEAR TEST ASTM D3080

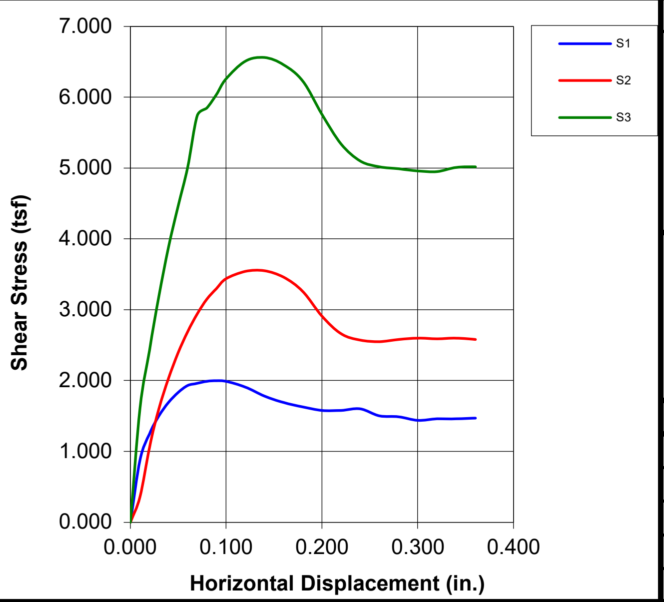
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

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.

KLEINFELDER
Bright People. Right Solutions.

9969 Horn Rd., Sacramento, CA 95827

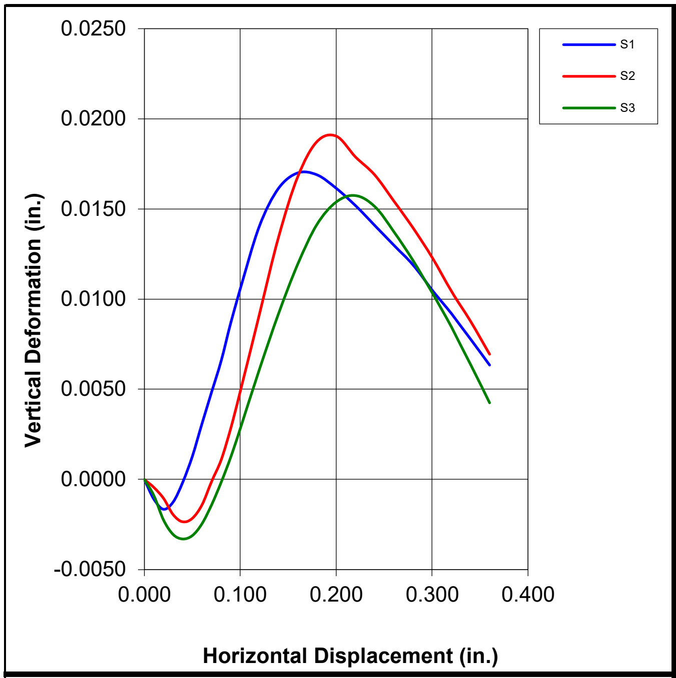
PROJECT NO.:	20180876
ENTRY BY:	A. Wohletz
CHECKED BY:	S. Rader
DATE:	4/6/2020

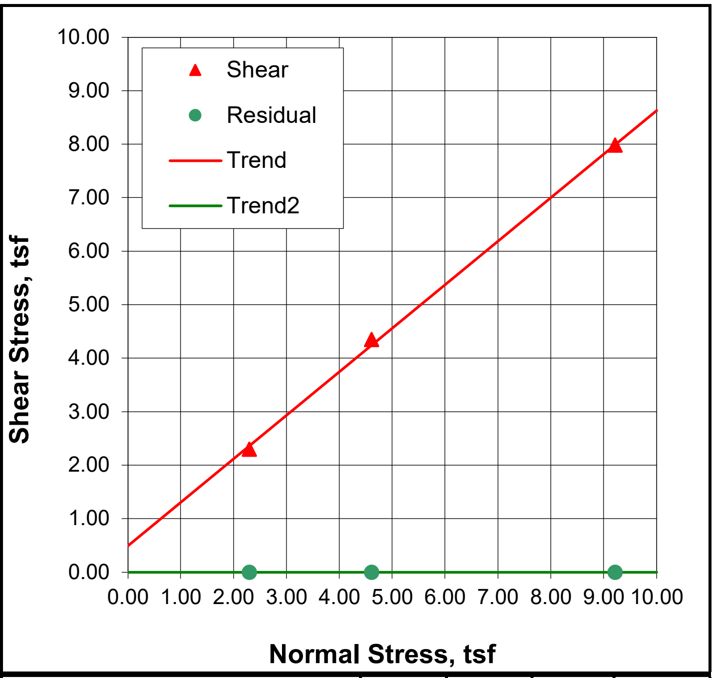
DIRECT SHEAR TEST ASTM D3080

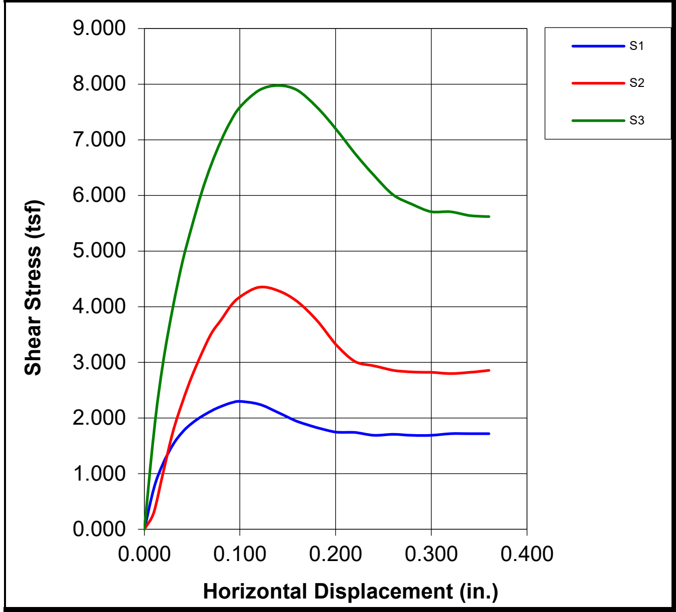
Camino Del Mar Bridge Replacement
Over San Dieguito River - Phase 0
Del Mar, California

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 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

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

- very dense

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)).

LEAN CLAY (CL); medium stiff; black (10YR 2/1); wet; few fine SAND; mostly fines; medium plasticity; micaceous, trace shell fragments; PP=0.5 tsf

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)).

- dense

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

- SAA

SILTY SAND (SM); dense; dark gray (10YR 4/1); mostly medium to fine SAND; little fines; non-plastic

- very dense

- SAA

- SAA

- dense

SILTY SAND (SM); dense; dark gray (10YR 4/1); mostly medium to fine SAND; little fines; non-plastic

CLAYSTONE; dark reddish brown (2.5YR 3/4) with grayish green (GLEY 1-5/5GY); medium plasticity (DEL MAR FORMATION (Td)).
-PP >=4.5 tsf

02-21-20
Terminated at Elev ~-135.00
Hammer Energy Ratio (ERi) = 94%

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:	DRAWN:	CHECKED:
D. Fahrney	D. Fahrney	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

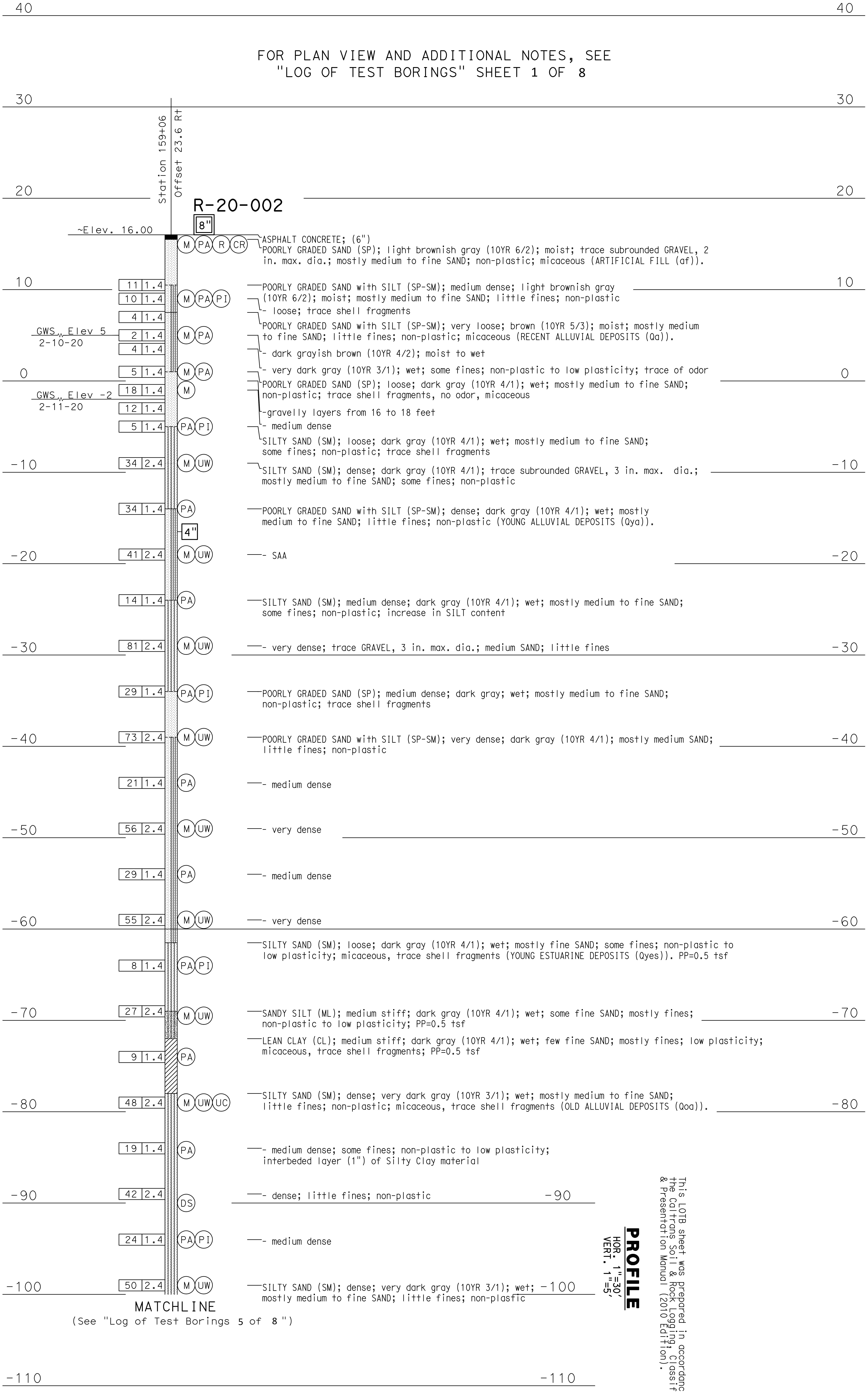
PUBLIC WORKS DIRECTOR
DATE

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

SHEET
3
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



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

MATCHLINE
(See "Log of Test Borings 5 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 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

MATCHLINE
(See "Log of Test Borings 4 of 8")

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

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

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

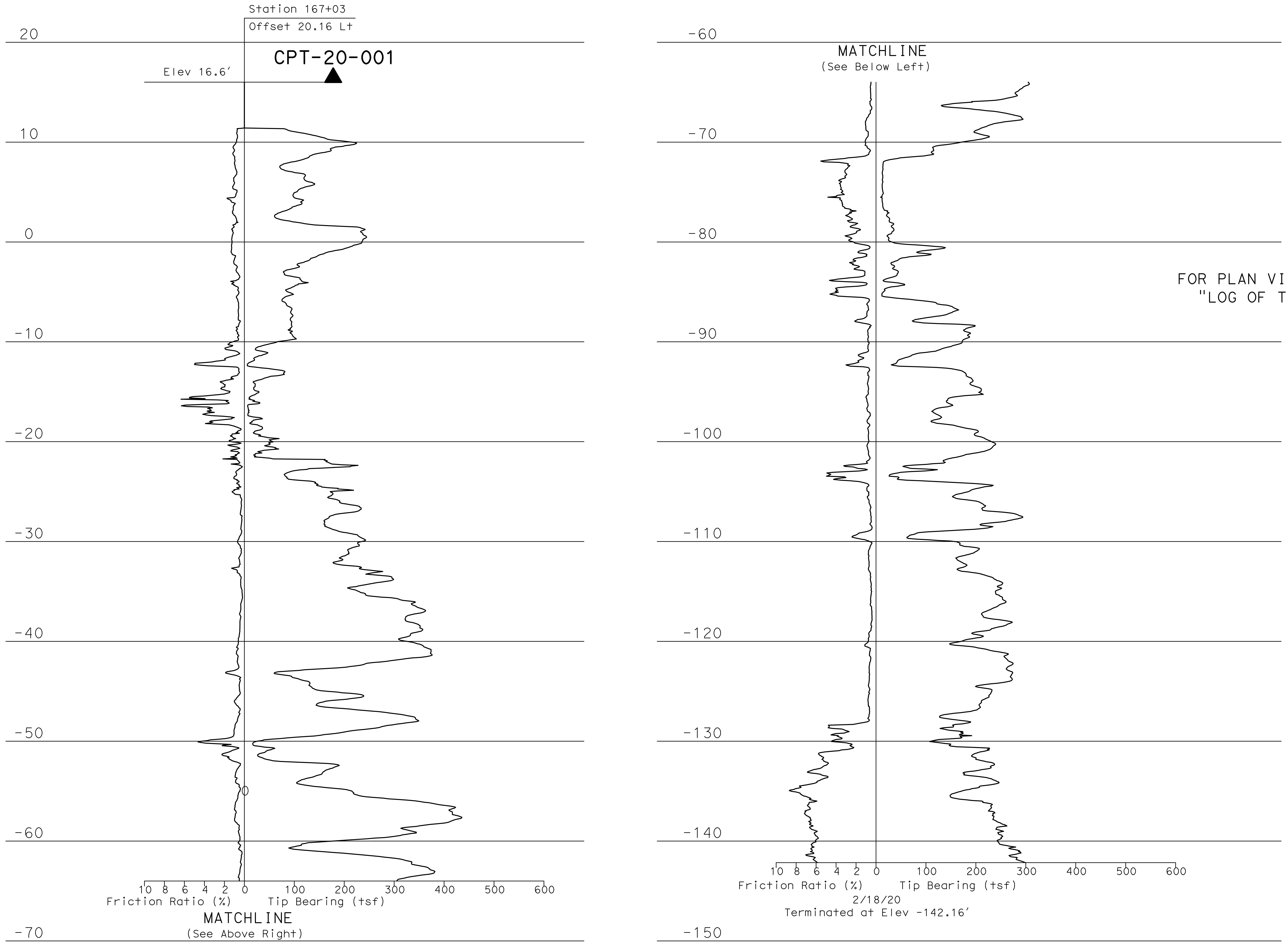
APPROVED

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
5
OF 8 SHEETS



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

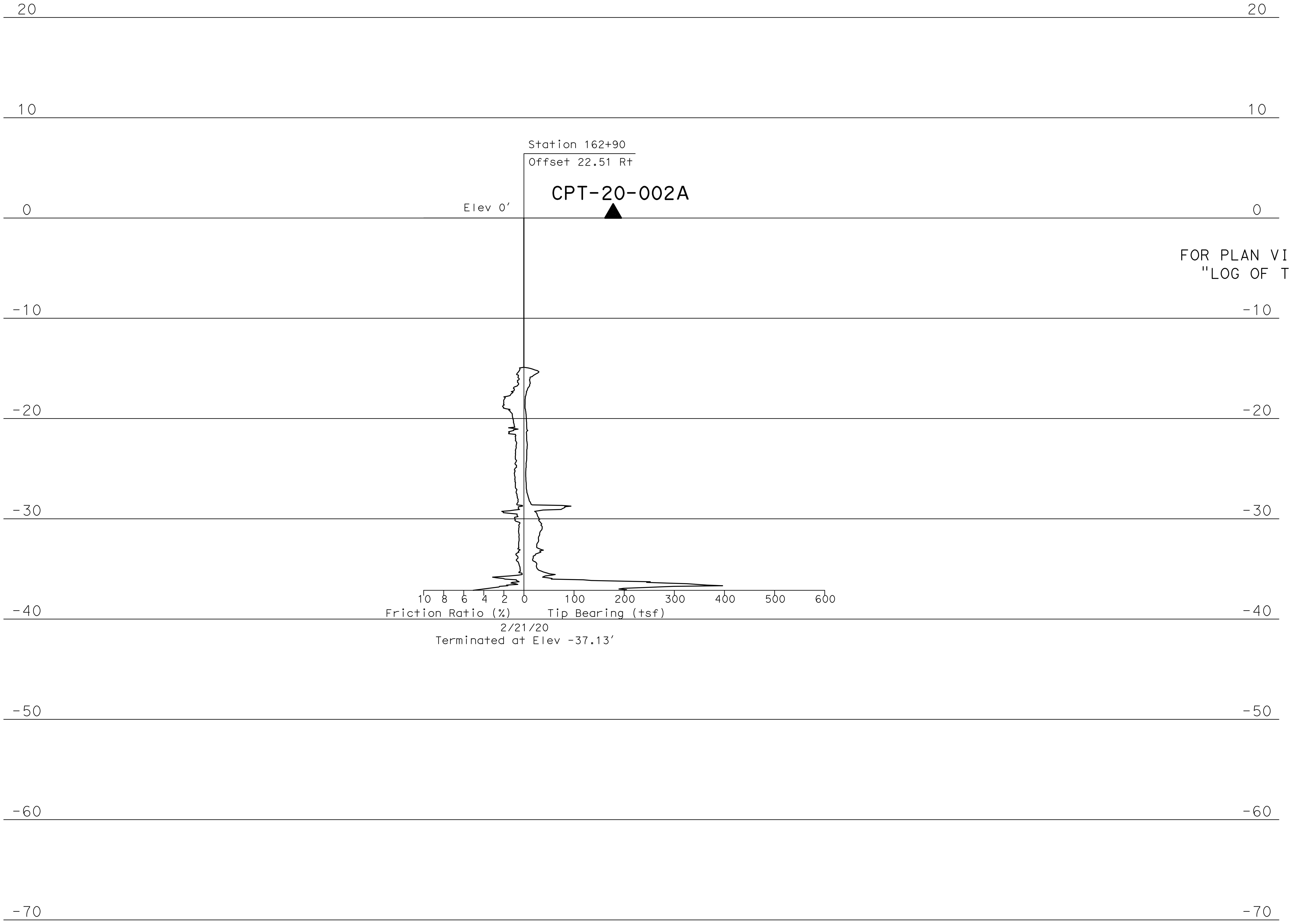
KLEINFELDER
Bright people. Right Solutions.
550 WEST C STREET, SUITE 1200 SAN DIEGO, CA. 92101 (619) 831-4600

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 6 OF 8 SHEETS

ORIGINAL SCALE: 0 1 2 3 4 INCHES



2/21/20
Terminated at Elev -37.13'

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

ORIGINAL SCALE: 0 1 2 3 4 INCHES

REVISION	DESCRIPTION	APPROVED	DATE

SCALE: AS SHOWN
ACAD FILE NO.
PROJECT NO.

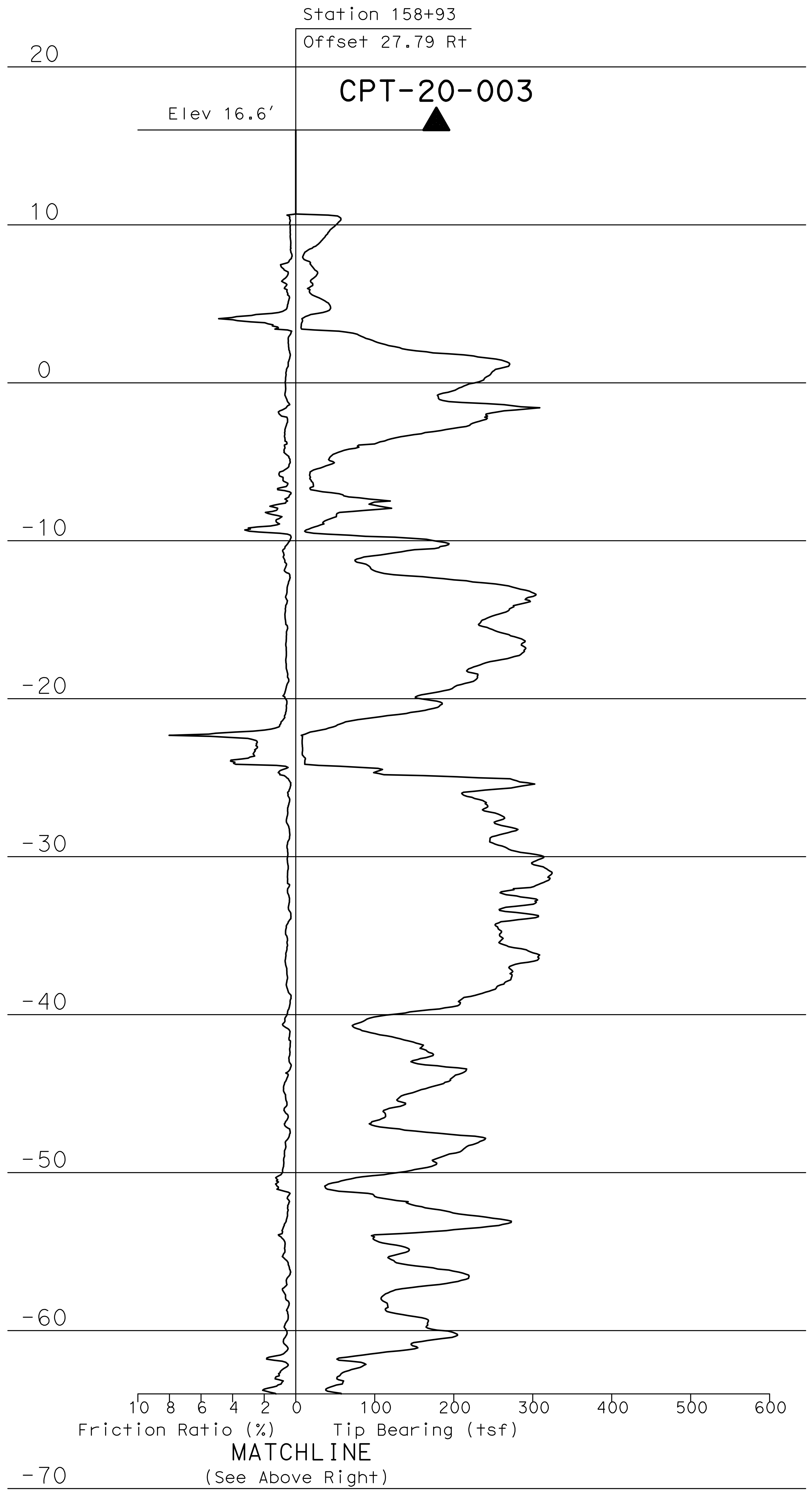
DESIGNED: D. Fahrney
DRAWN: D. Fahrney
CHECKED: J. Bonfiglio
ENGINEER OF WORK
R.C.E. NO.
DATE

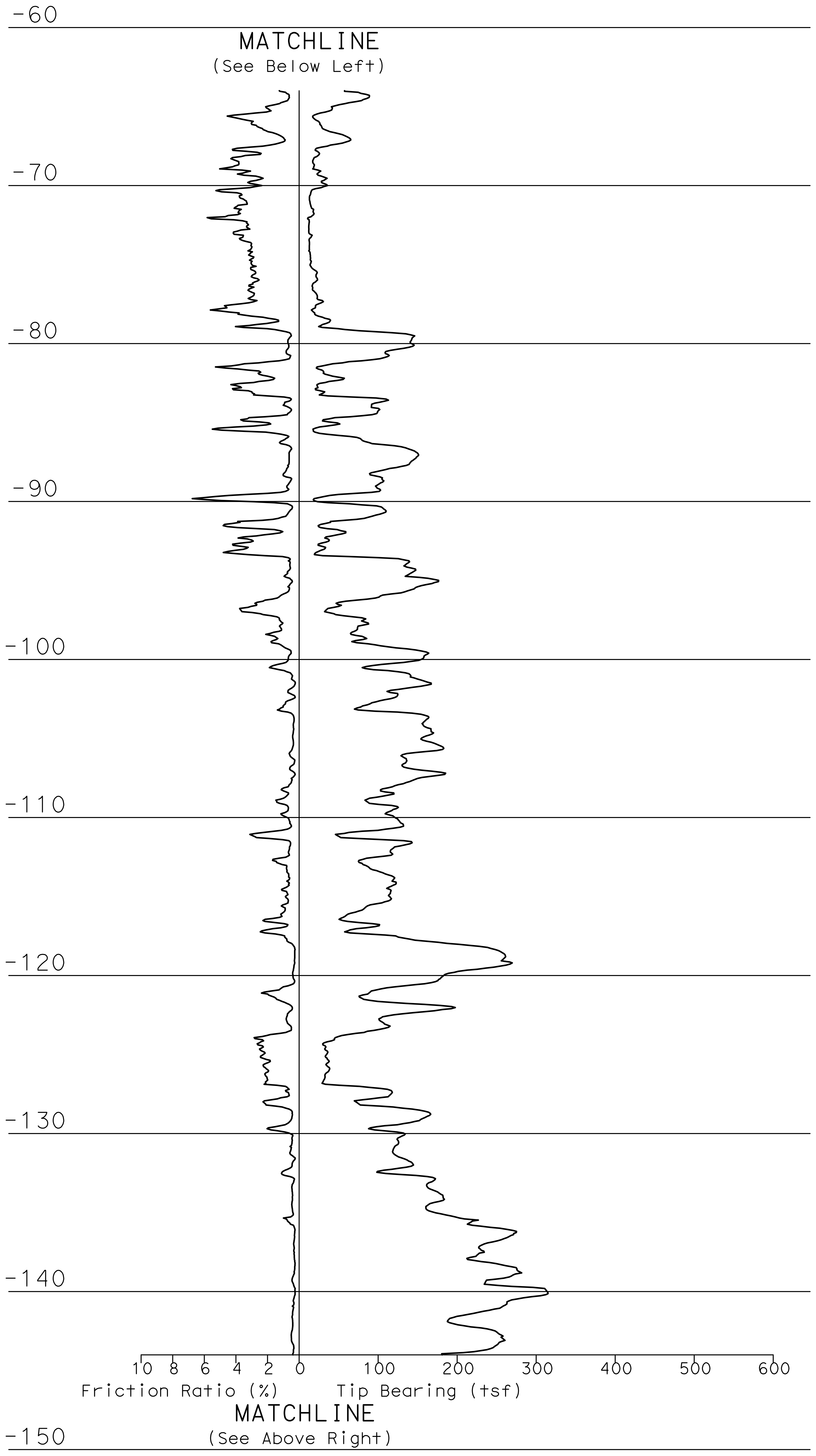
KLEINFELDER
Bright people. Right Solutions.
550 WEST C STREET, SUITE 1200 SAN DIEGO, CA. 92101 (619) 831-4600

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

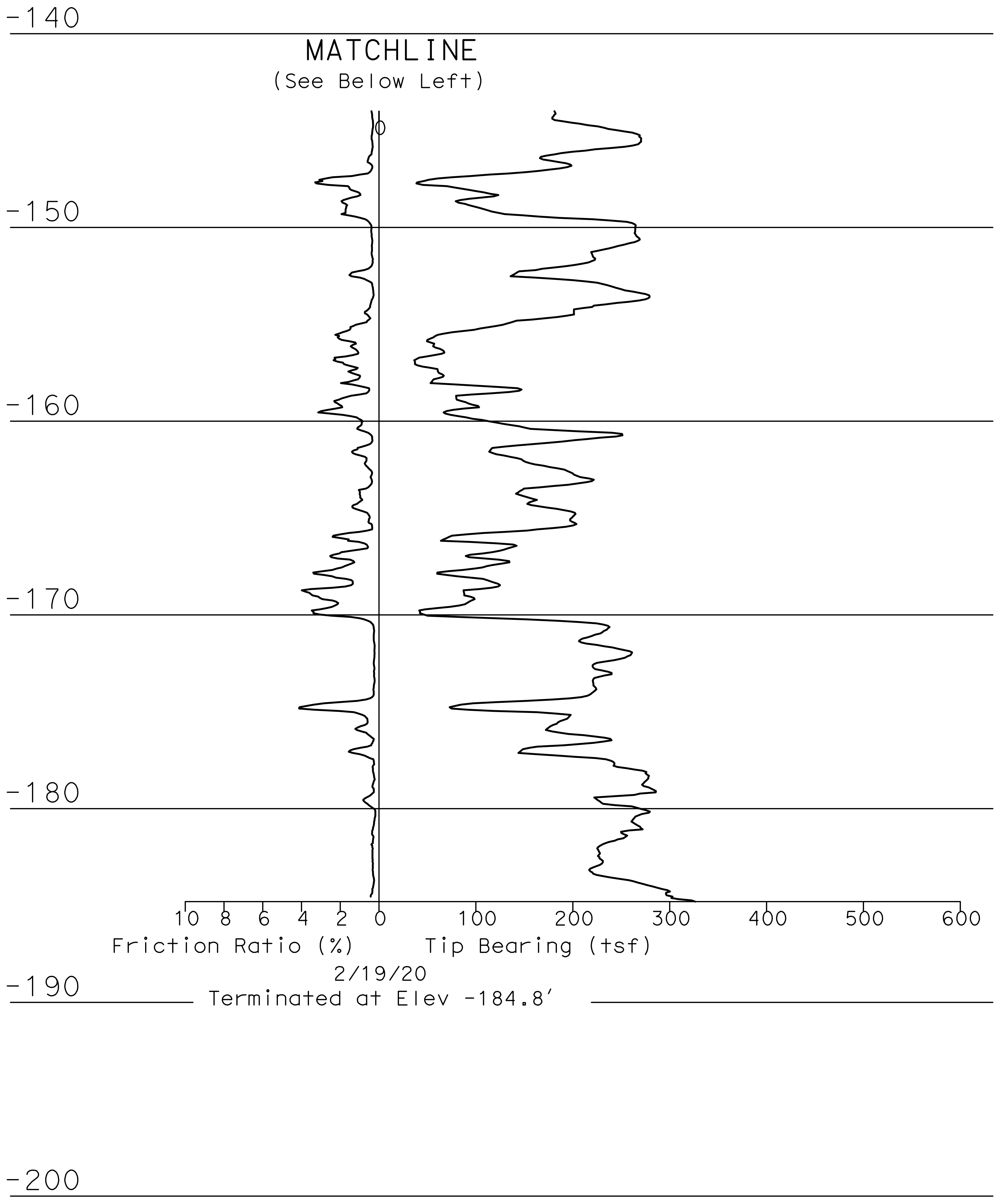
SHEET 7 OF 8 SHEETS





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

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
8
OF 8 SHEETS

ORIGINAL SCALE: INCHES 0 1 2 3 4

APPENDIX E

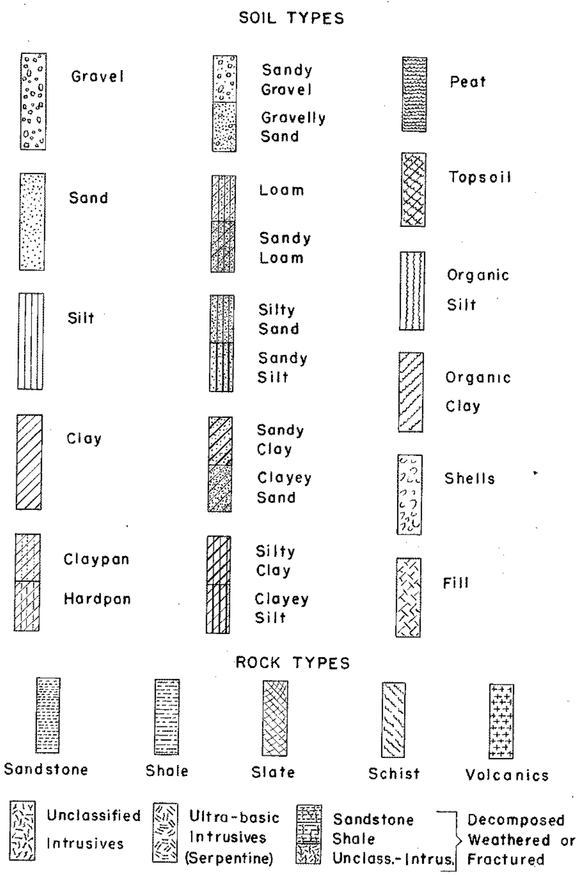
PREVIOUS RELEVANT GEOTECHNICAL INFORMATION BY OTHERS

APPENDIX E
PREVIOUS RELEVANT GEOTECHNICAL INFORMATION BY OTHERS

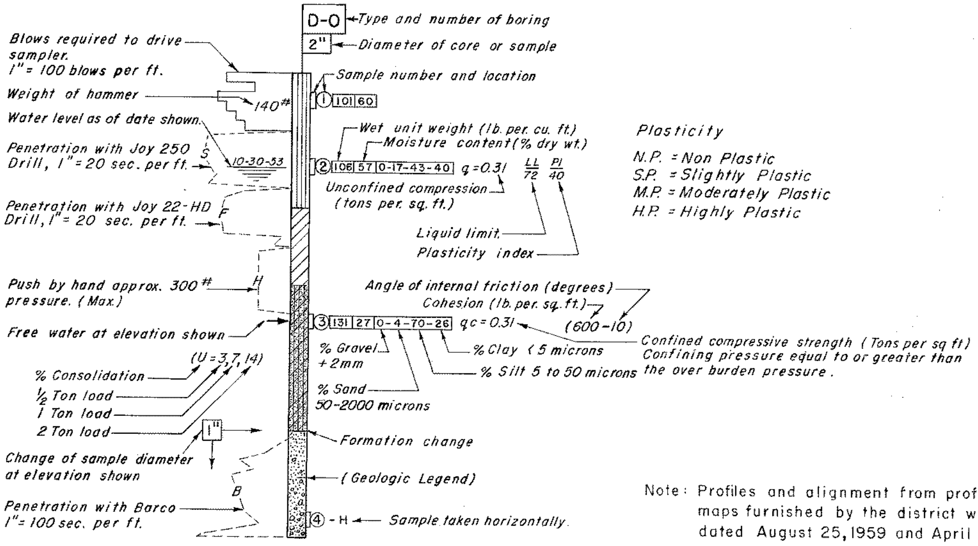
E.1 California Department of Public Works 1960 LOTBs
E.2 Ninyo & Moore 2018 Exploration Logs
E.3 Ninyo & Moore 2018 Laboratory Test Results

E.1 California Department of Public Works 1960 LOTBs

GEOLOGIC LEGEND



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

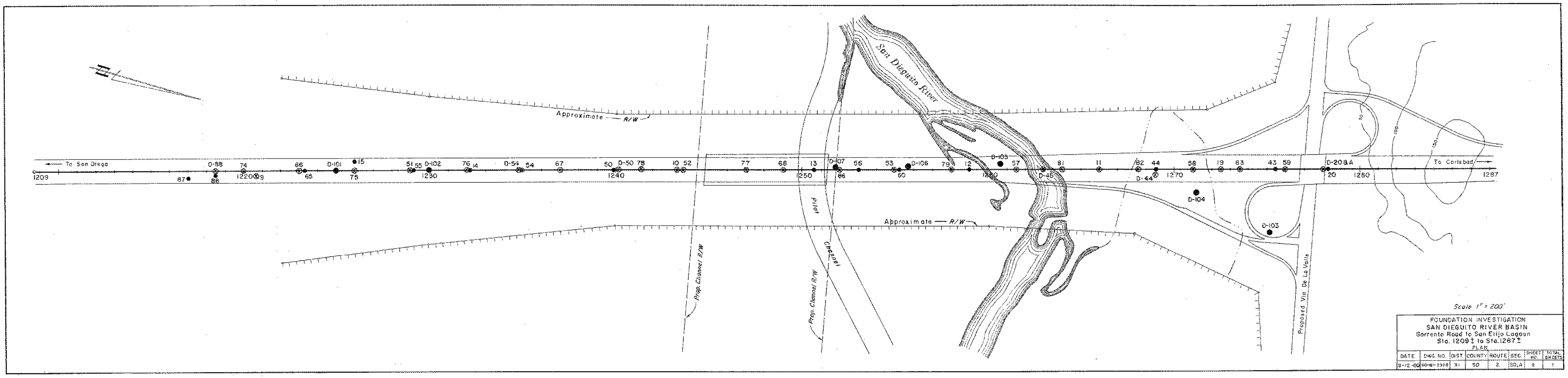
BORING LEGEND - PLAN SHEETS

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

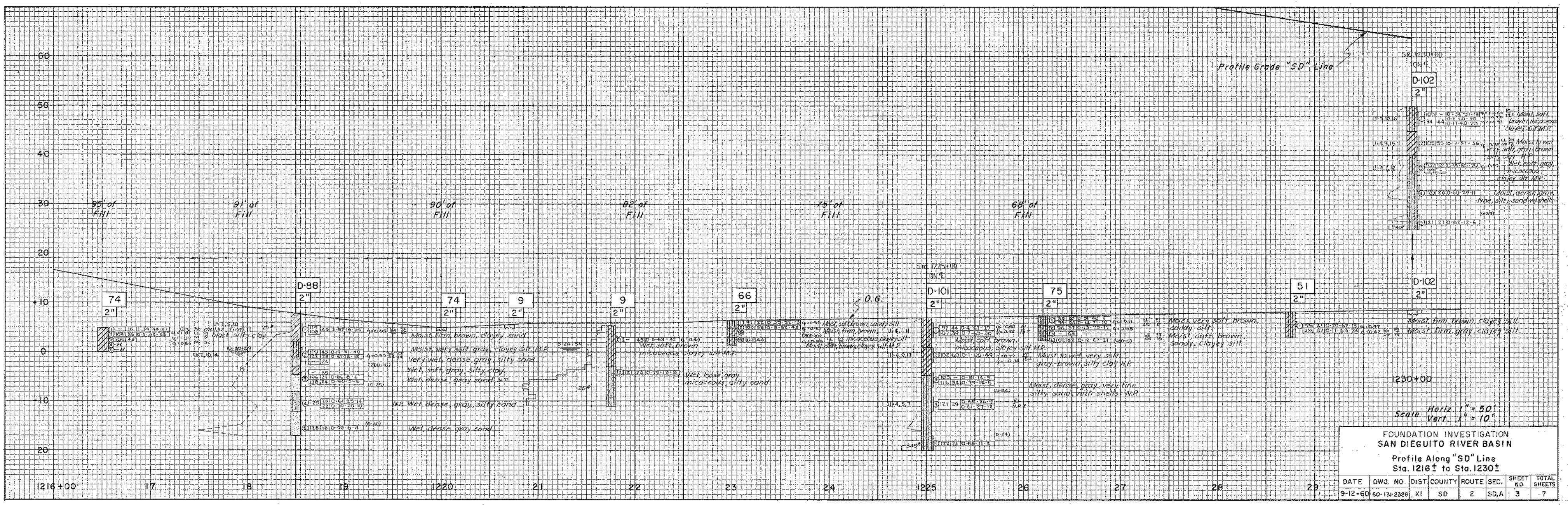
DWG. NO. 60-130-2328	SUBMITTED BY: a W Root SUPV. MATERIALS & RESEARCH ENG'R.	DATE 9-12-60
DRAWN BY R. P.	APPROVED BY: F. N. Hveem MATERIALS & RESEARCH ENG'R.	SHEET NO. 1 OF 7 SHEETS
CHECKED BY W. F. K.		



Scale 1" = 200'

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

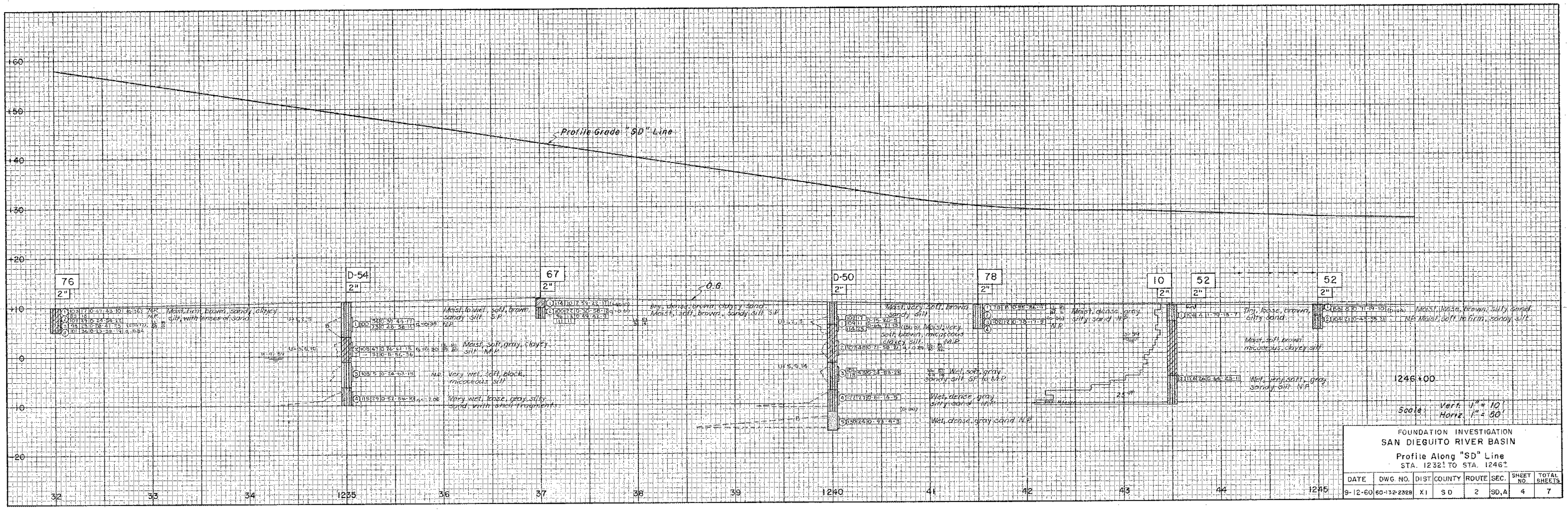
DATE	DWG. NO.	DIST.	COUNTY	ROUTE	SEC.	SHEET NO.	TOTAL SHEETS
9-12-60	60-161-2328	XI	SD	2	SD,A	2	7



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	60-131-2326	XI	SD	2	SD,A	3	7

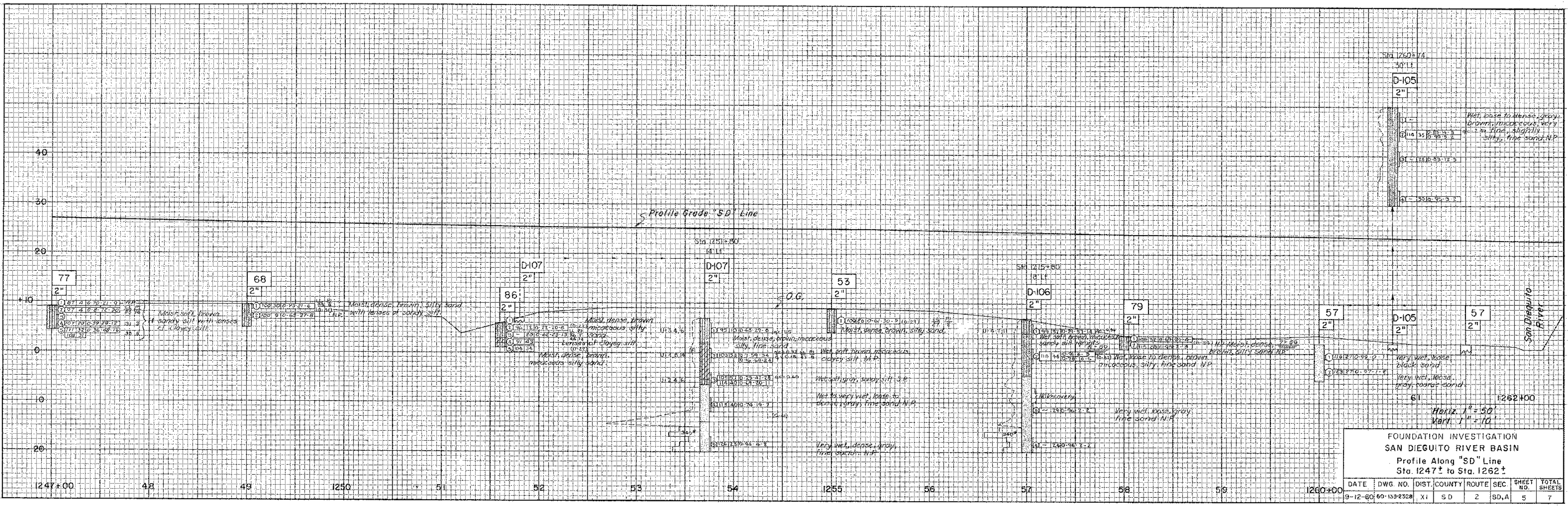


Scale: Vert. 1" = 10'
Horiz. 1" = 50'

FOUNDATION INVESTIGATION
SAN DIEGUITO RIVER BASIN

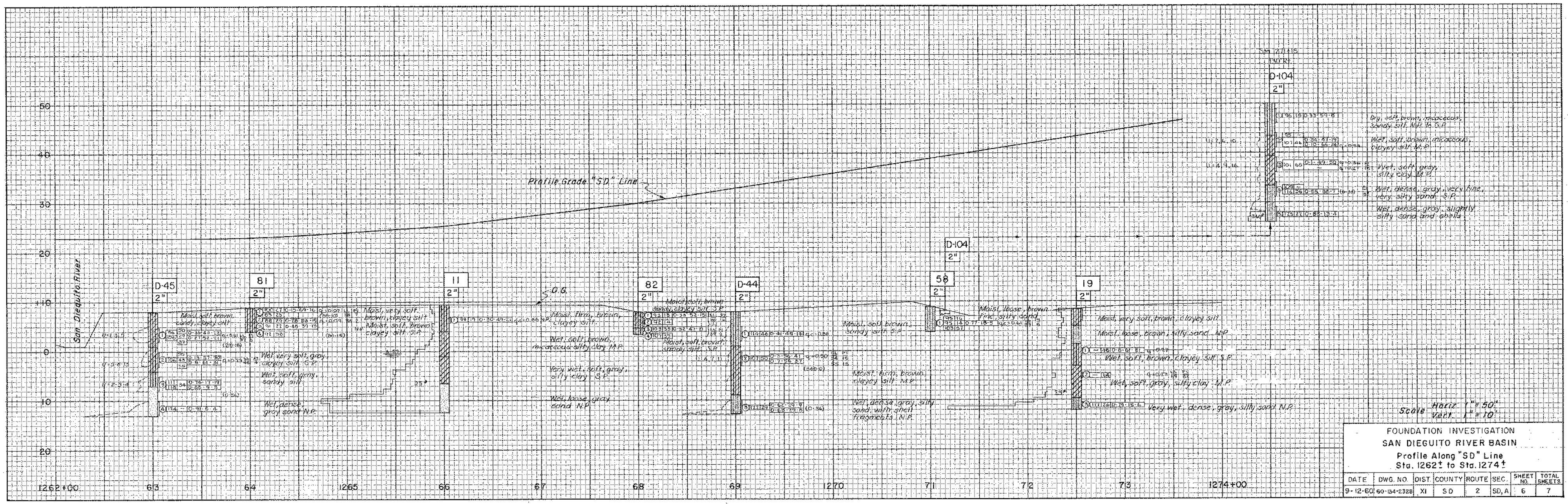
Profile Along "SD" Line
STA. 1232± TO STA. 1246±

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



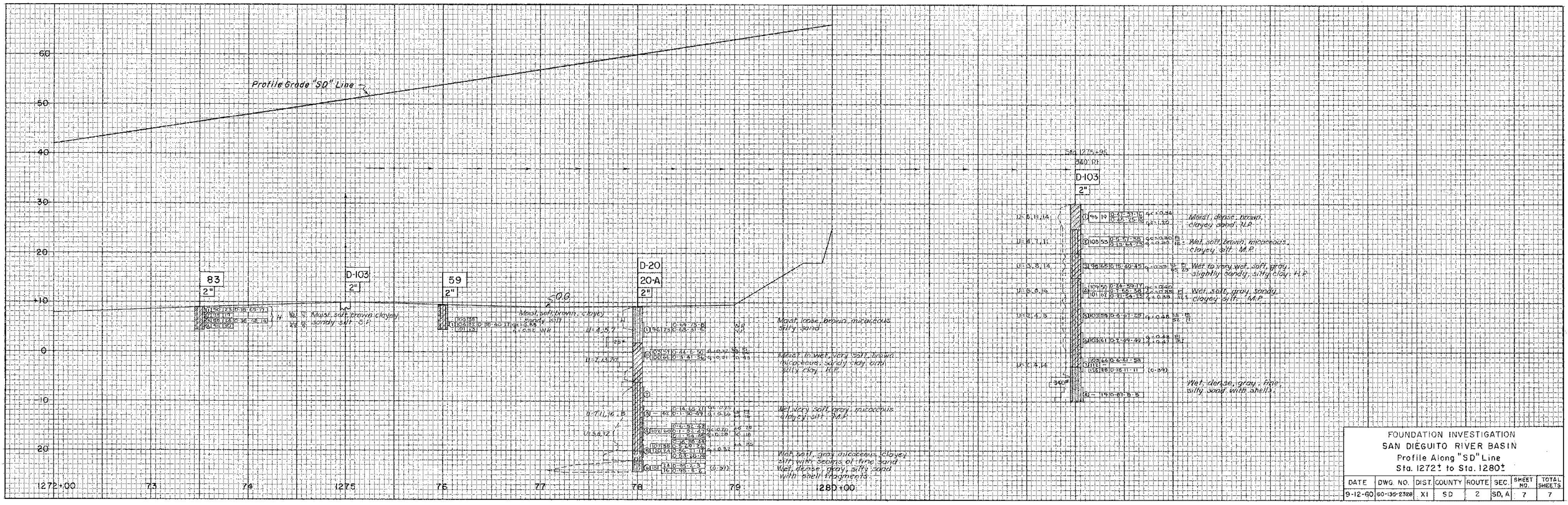
FOUNDATION INVESTIGATION
SAN DIEGUITO RIVER BASIN
Profile Along "SD" Line
Sta. 1247± to Sta. 1262±

DATE	DWG. NO.	DIST.	COUNTY	ROUTE	SEC.	SHEET NO.	TOTAL SHEETS
9-12-60	60-133-2328	XI	SD	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" = 50 ft.
Vert 1" = 10 ft

LEGEND

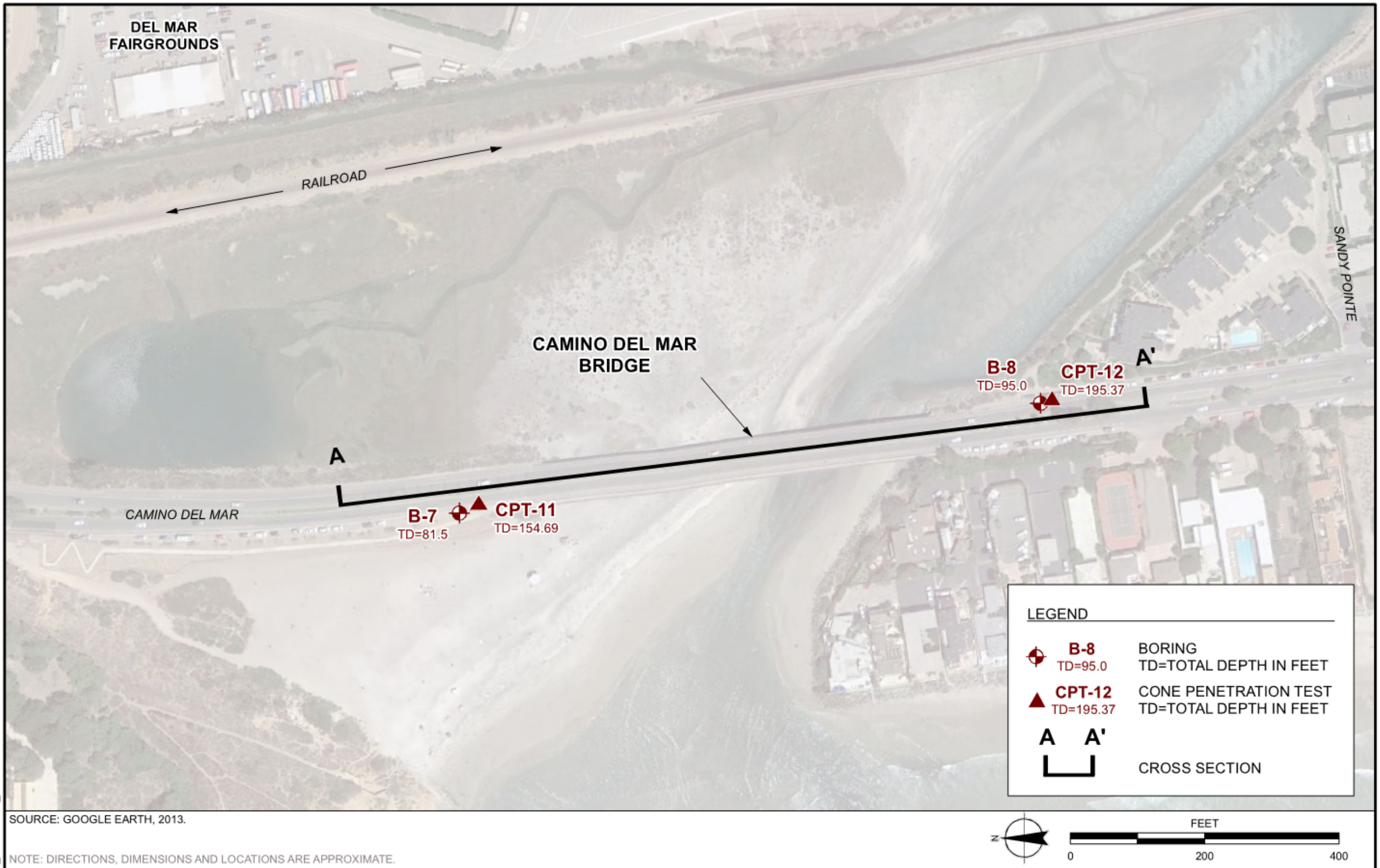
▨ CLAY

☐ 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

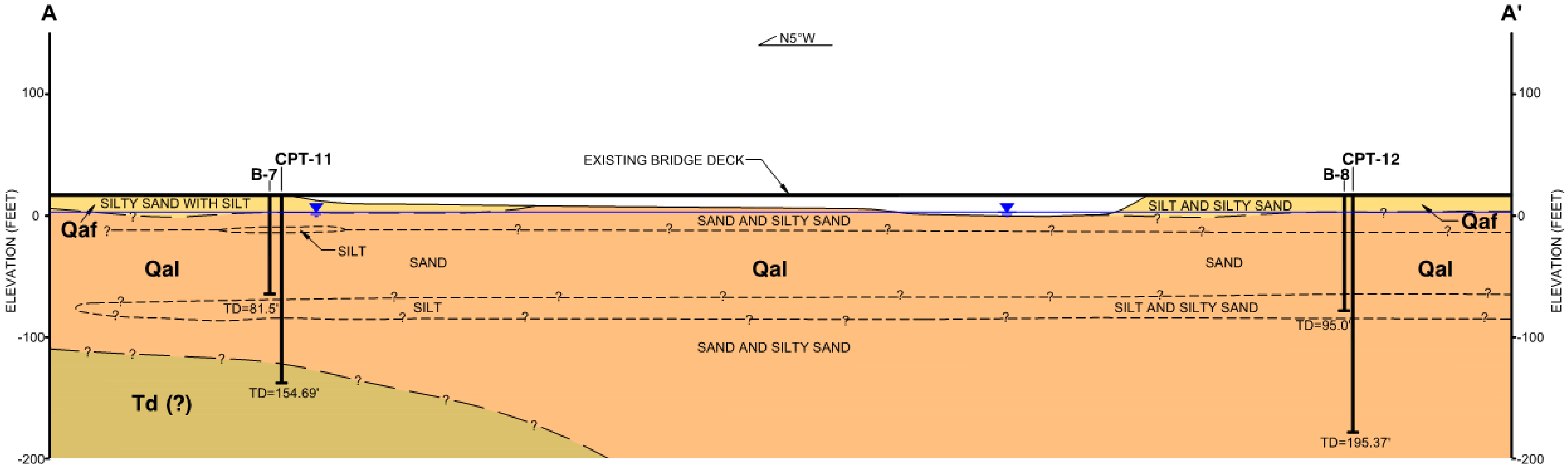
FIGURE 2

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

--

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'

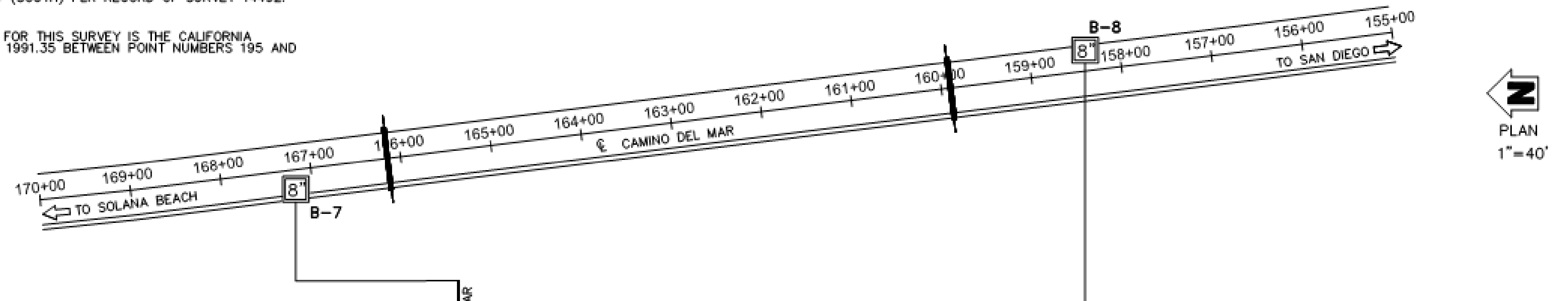
BASIS OF BEARINGS: THE BASIS OF BEARINGS FOR THIS SURVEY IS THE CALIFORNIA COORDINATE SYSTEM NAD 83, ZONE 6, EPOCH 1991.35 BETWEEN POINT NUMBERS 195 AND 459 AS SHOWN ON ROS 14492.

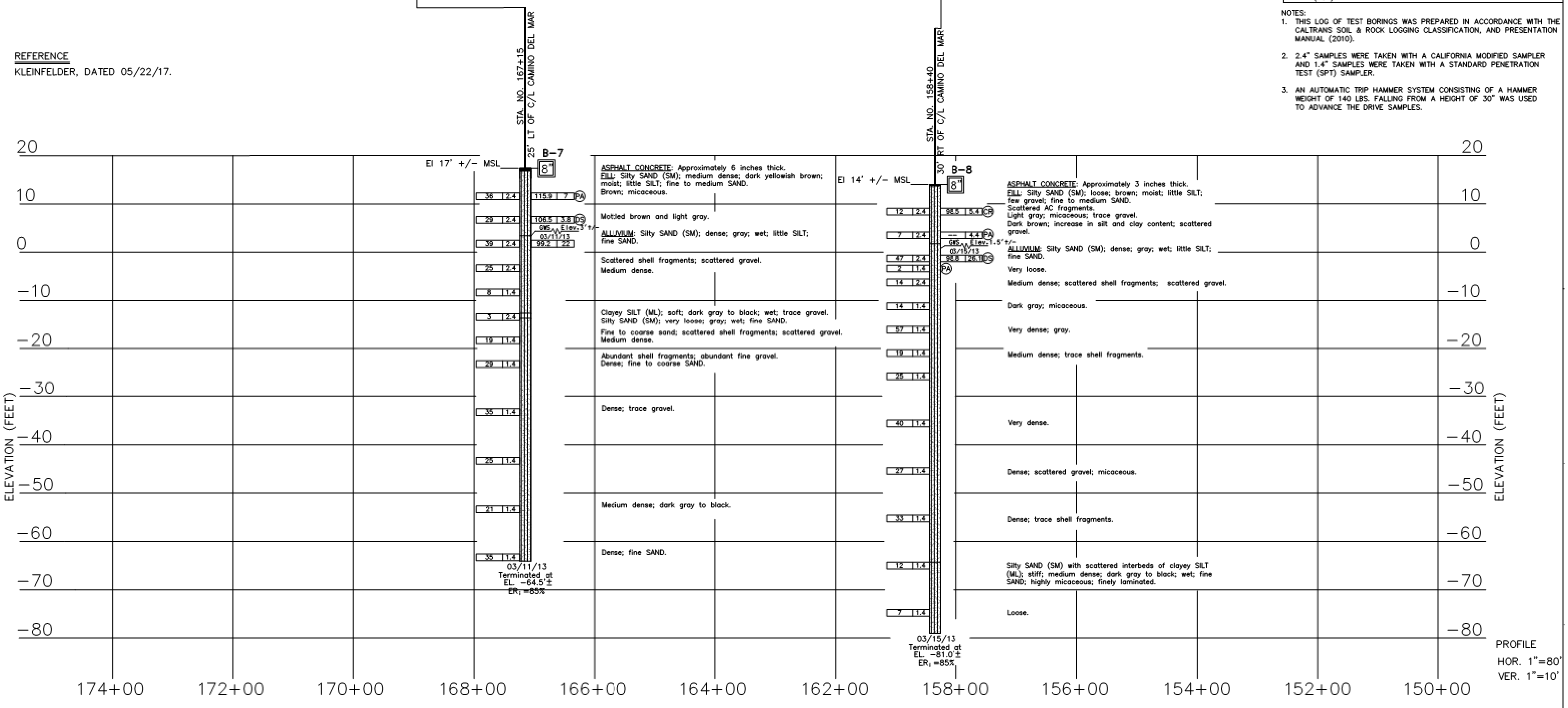
REFERENCE

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

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

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

--

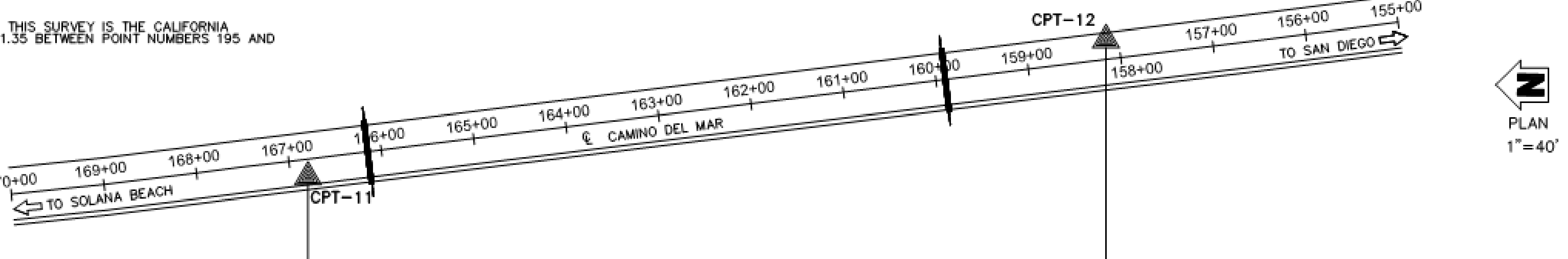
NINYO & MOORE
5710 Ruffin Road
San Diego, CA 92123
Phone (858) 576-1000

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'

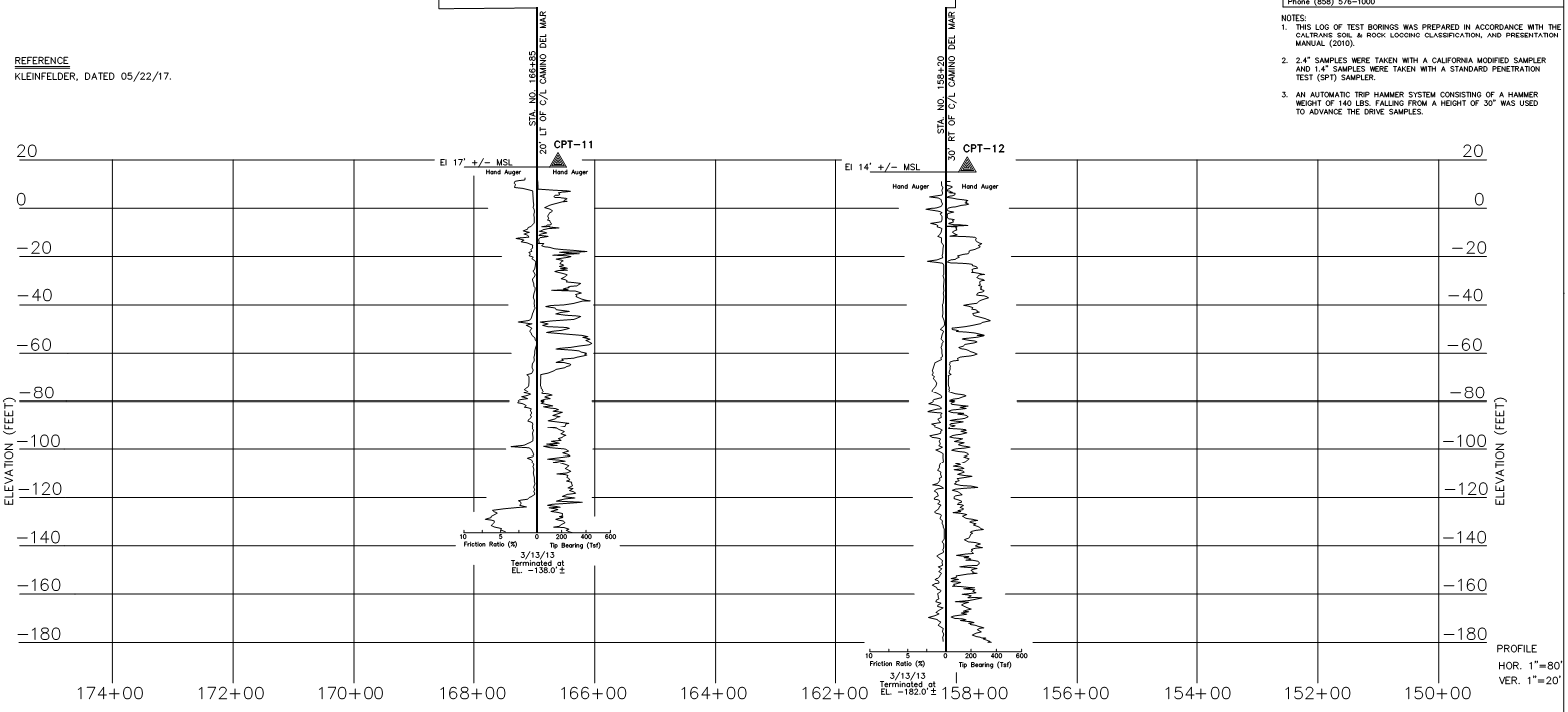
BASIS OF BEARINGS: THE BASIS OF BEARINGS FOR THIS SURVEY IS THE CALIFORNIA COORDINATE SYSTEM NAD 83, ZONE 6, EPOCH 1991.35 BETWEEN POINT NUMBERS 195 AND 459 AS SHOWN ON ROS 14492.



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.



-- 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

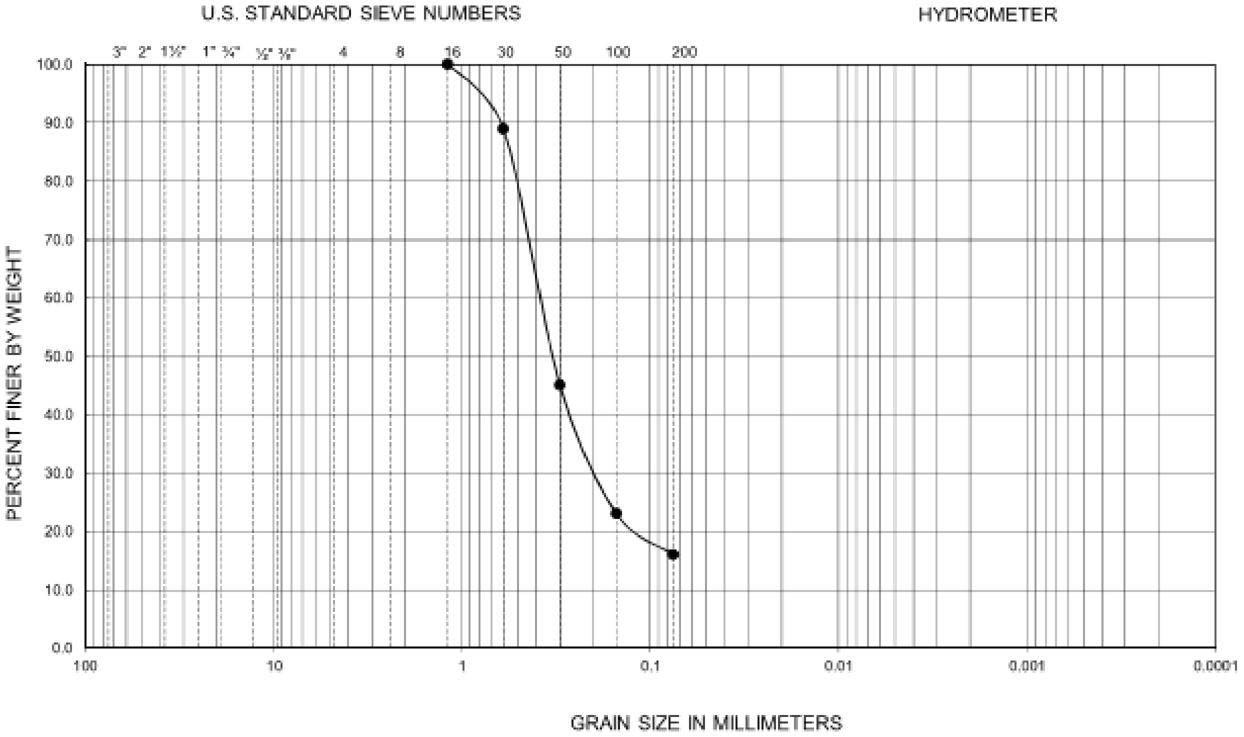
LOG OF TEST BORINGS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES				SHEET	OF
1				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

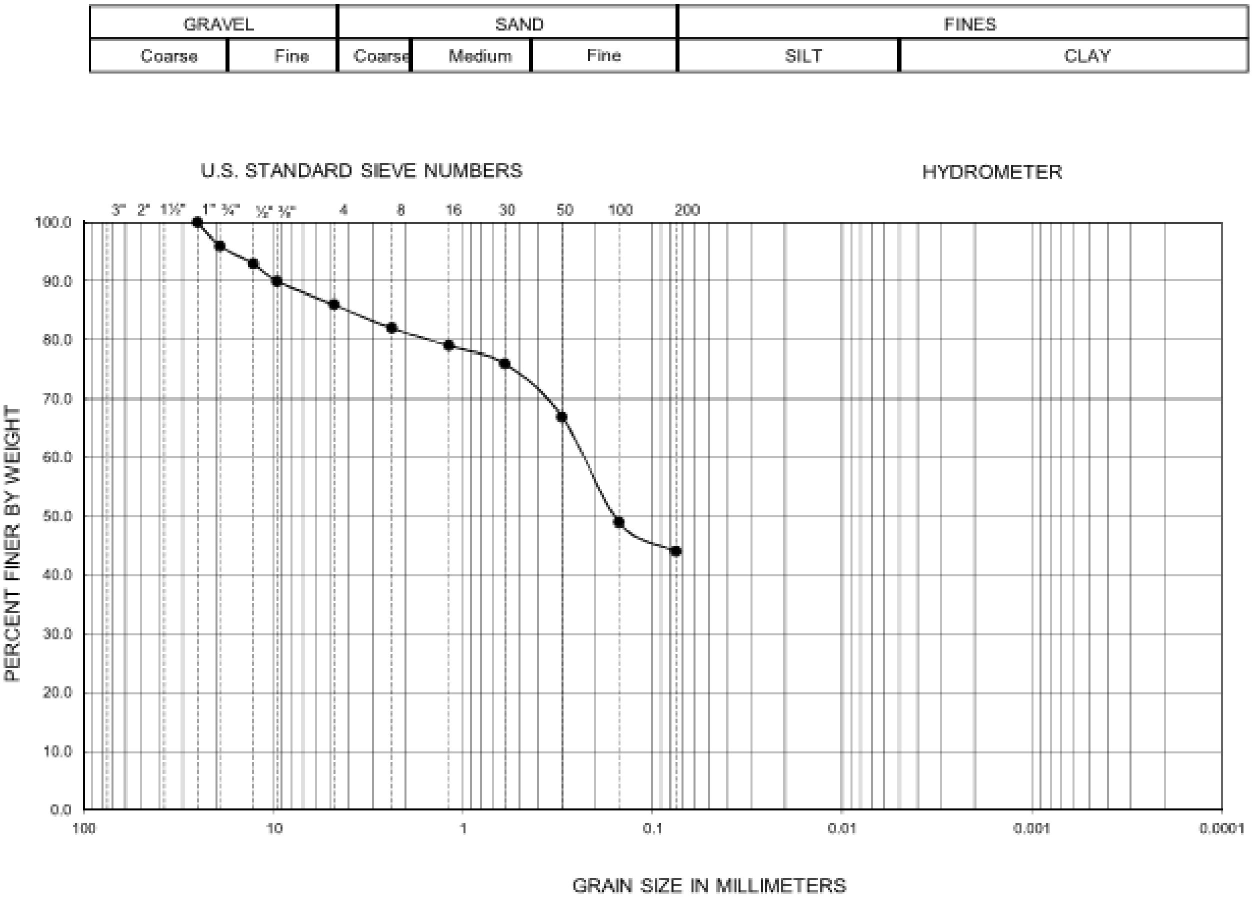
GRADATION TEST RESULTS

FIGURE

C-1

PROJECT NO.	DATE
108449001	7/18

CAMINO DEL MAR BRIDGE REPLACEMENT
DEL MAR, CALIFORNIA



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

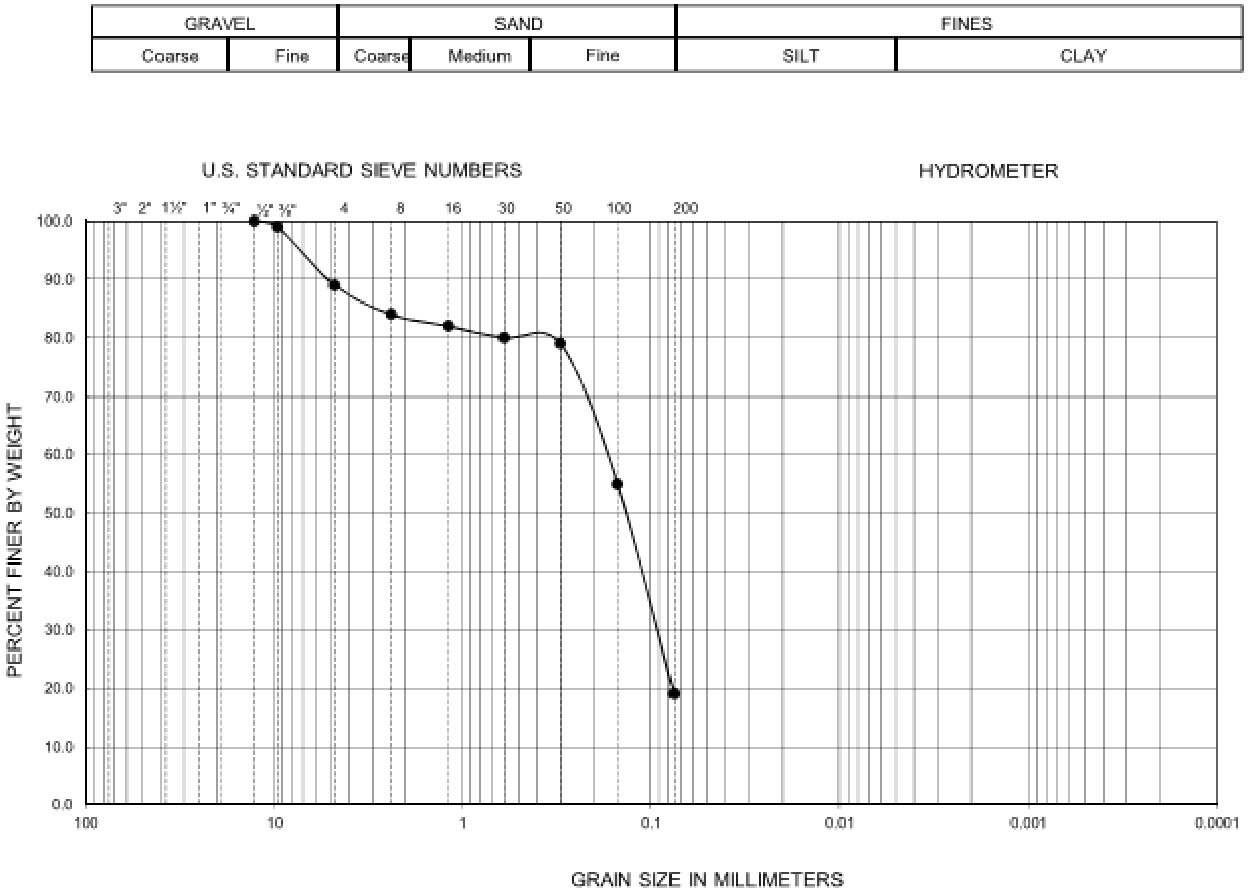
GRADATION TEST RESULTS

FIGURE

C-2

PROJECT NO.	DATE
108449001	7/18

CAMINO DEL MAR BRIDGE REPLACEMENT
DEL MAR, CALIFORNIA



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

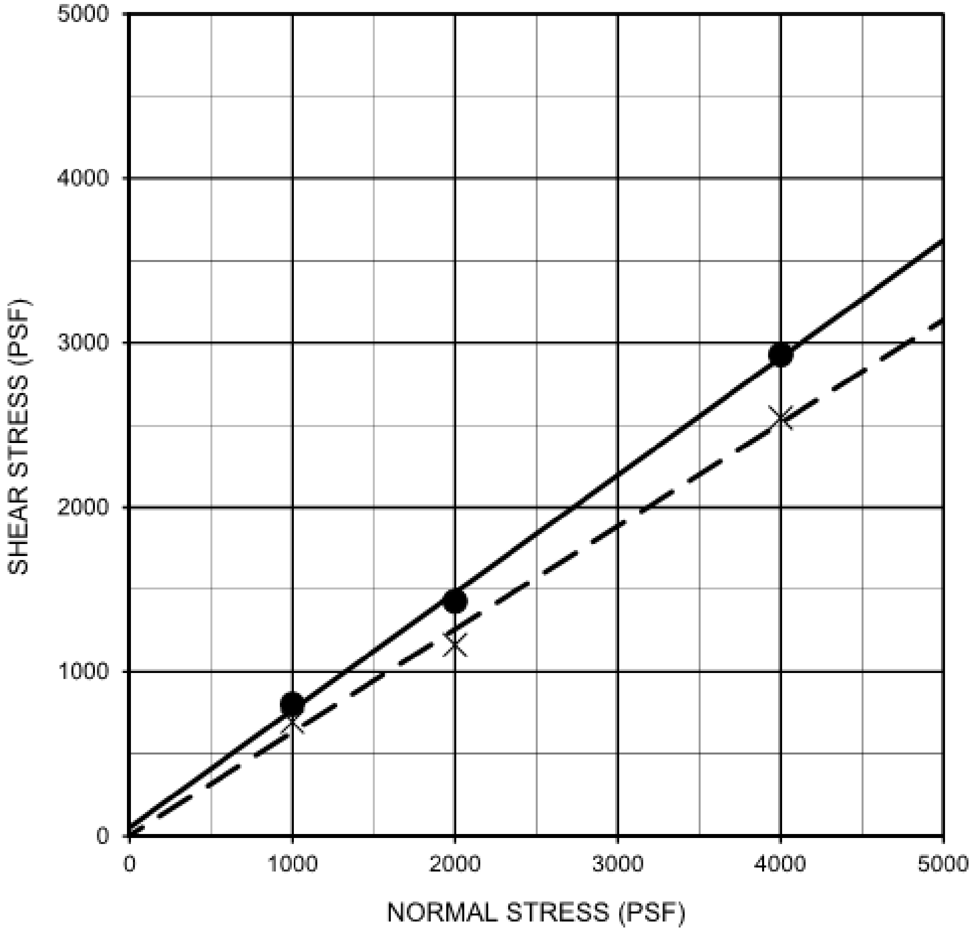
GRADATION TEST RESULTS

FIGURE
C-3

PROJECT NO.
108449001

DATE
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

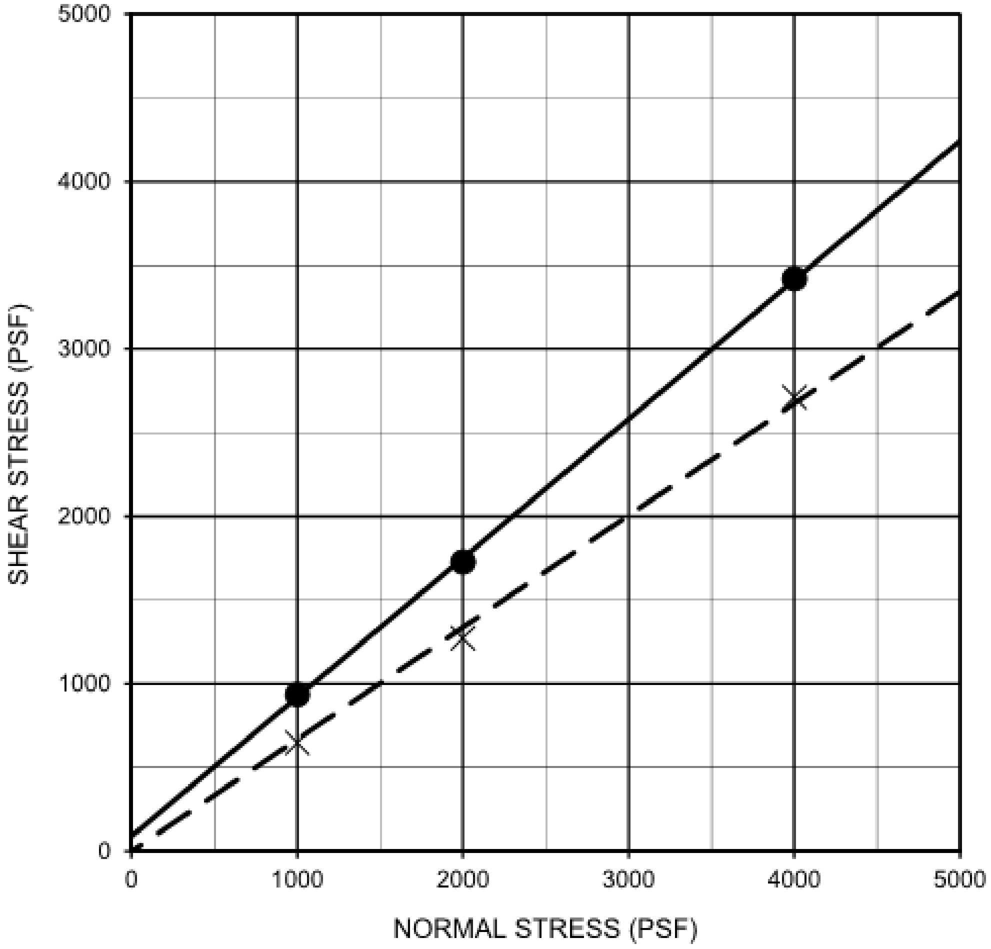
DIRECT SHEAR TEST RESULTS

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
PROJECT NO. 108449001 / DATE 7/18	CAMINO DEL MAR BRIDGE REPLACEMENT DEL MAR, CALIFORNIA	C-5

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 7 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

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
3		Sand (Loose)[1]	16	120	28	0.53	0
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

Table F-2 (Continued)

Caltrans ARS Online Target Response Spectrum

Period	Near Fault Amplification Factor	Probabilistic Spectral Acceleration (g)
0.5	1	0.92
0.75	1.1	0.78
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	$D_{5\text{-}95}$ (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; $D_{5\text{-}95}$ – 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 7 of this Preliminary Geotechnical Design Report and rely upon the results of the field investigation as presented in this report.

APPENDIX REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 2017, AASHTO LRFD Bridge Design Specifications, 8th Edition, September 2017.

Atik, L.A. and Abrahamson, N., 2010, An Improved Method for Nonstationary Spectral Matching, Earthquake Spectra, Vol. 26, No. 3, pp. 601-617, August 2010.

Bommer, J.J., Stafford, P.J., and Alarcon, J.E., 2009, Empirical Equations for Prediction of the Significant, Bracketed, and Uniform Duration of Earthquake Ground Motion, Bulletin of the Seismological Society of America, Vol. 99, No. 6, pp. 3217-3233, December 2009.

California Department of Transportation (Caltrans), 2019, California Amendments to the AASHTO LRFD bridge Design Specifications (2017 Eighth Edition), April 2019.

California Department of Transportation (Caltrans), 2019 “Caltrans Seismic Design Criteria,” Version 2.0, April 2019.

Darendeli, M. B. (2001). Development of a New Family of Normalized Modulus Reduction and Material Damping Curves, Department of Civil, Architectural and Environmental Engineering, The University of Texas, Austin, Texas.

Groholski, D., Hashash, Y, Musgrove, M., Harmon, J, and Kim, B., 2015, Evaluation of 1-D Non-linear Site Response Analysis using a General Quadratic/Hyperbolic Strength-Controlled Constitutive Model. 6th International Conference on Earthquake Geotechnical Engineering.

Harmon, J.A., 2017, Nonlinear Site Amplification Functions for Central and Eastern North America, University of Illinois at Urbana-Champaign Dissertation, 2017.

Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Ilhan, O., Xing, G., Groholski, D.R., Phillips, C.A., and Park, D. (2020) “DEEPSOIL 7.0, User Manual”. Urbana, IL, Board of Trustees of University of Illinois at Urbana-Champaign.

Hayden, C.P., Bray, J.D., and Abrahamson, N.A., 2014, Selection of Near-Fault Pulse Motions, Journal of Geotechnical and Geoevironmental Engineering, March 2014.

Lilhanand, K., and Tseng, W. S., 1987. Generation of synthetic time histories compatible with multiple-damping response spectra, *SMiRT-9*, Lausanne, K2/10.

Lilhanand, K., and Tseng, W. S., 1988. Development and application of realistic earthquake time histories compatible with multiple damping response spectra, in *Ninth World Conference on Earthquake Engin*eering, Tokyo, Japan, Vol 2, 819–824.

Musgrove, M., Harmon, J., Hashash, Y. M., & Rathje, E. (2017). Evaluation of the DEEPSOIL Software on the DesignSafe Cyberinfrastructure. Journal of Geotechnical and Geoenvironmental Engineering, 143(9), 02817005.

Pacific Earthquake Engineering Research Institute (PEER), 2014, PEER NGA-West2 Database (PEER Report 2013/03), by: Timothy D. Ancheta, Robert B. Darragh, Jonathan P. Stewart, Emel Seyhan, Walter J. Silva, Brian S.J. Chiou, Katie E. Wooddell, Robert W. Graves, Albert R. Kottke, David M. Boore, Tadahiro Kishida, and Jennifer L. Donahue. Petersen, M., Moschetti, M., et al. (2014). Documentation for the 2014 Update of the United States National Seismic Hazard Maps. USGS Open File Report 2014-1091.

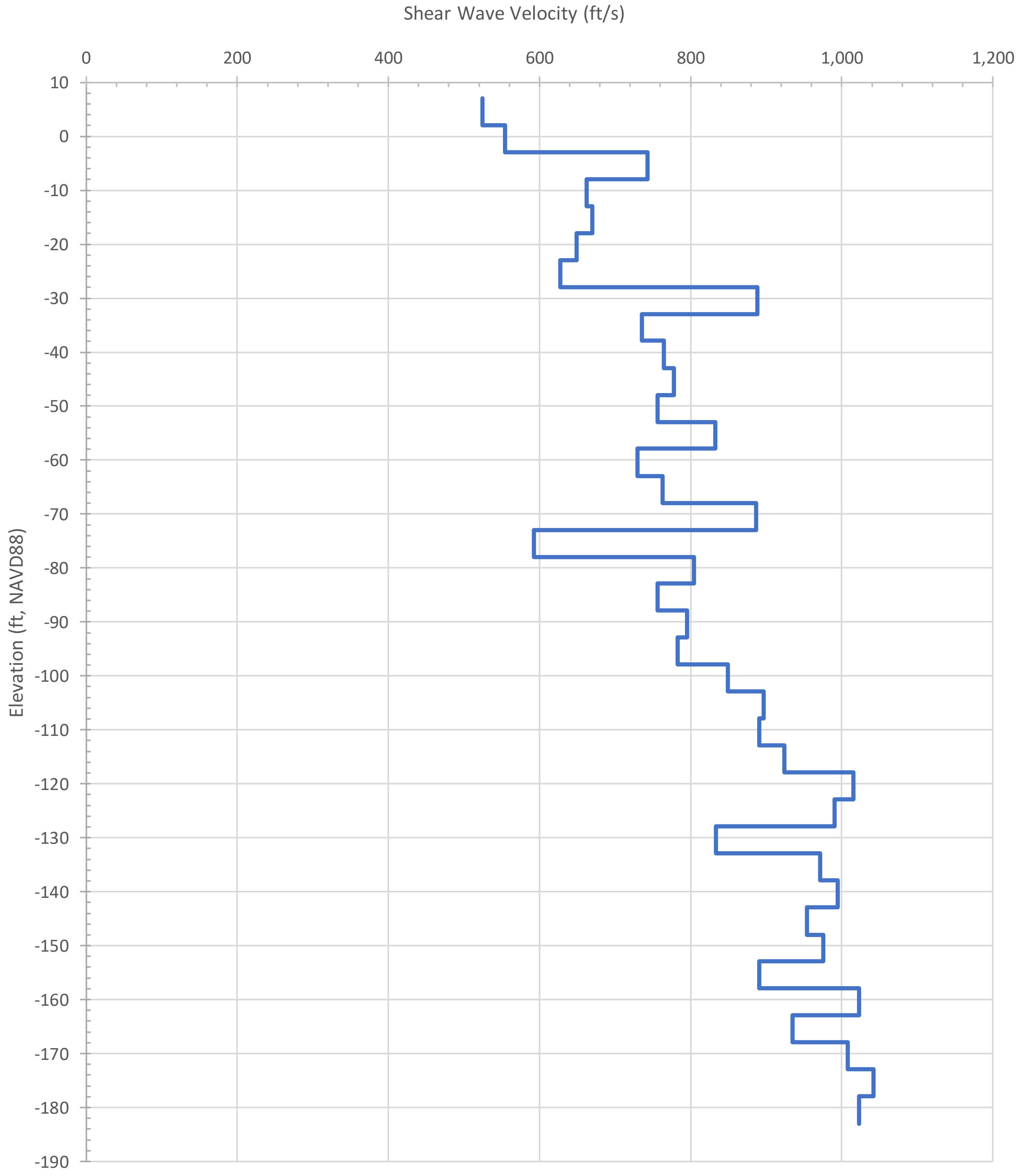
Petersen, M.D., Moschetti, M.P., Powers, P.M., Mueller, C.S., Haller, K.M., Frankel, A.D., Zeng, Yuehua, Rezaeian, Sanaz, Harmsen, S.C., Boyd, O.S., Field, Ned, Chen, Rui, Rukstales, K.S., Luco, Nico, Wheeler, R.L., Williams, R.A., and Olsen, A.H., 2014, Documentation for the 2014 update of the United States national seismic hazard maps: U.S. Geological Survey Open-File Report 2014–1091, 243 p., *https://dx.doi.org/10.3133/ofr20141091*.

Risk Engineering, Inc. (2018), EZ-FRISK^{TM} Online User's Manual, EZ-FRISK^{TM} Version 8. Risk Engineering, Inc., Boulder, Colorado.

Shahi, S.K. and Baker, J.W., 2014, An Efficient Algorithm to Identify Strong-Velocity Pulses in Multicomponent Ground Motions, Bulletin of the Seismological Society of America, Vol. 104, No. 5, pp. 2456–2466, October 2014.

Travasarou, T., Bray, J.D., and Abrahamson, N.A., 2003, Empirical Attenuation Relationship for Arias Intensity, Earthquake Engineering and Structural Dynamics, Vol. 32, pp. 1133-1155, 2003.

SCPT Shear Wave Velocity Results



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

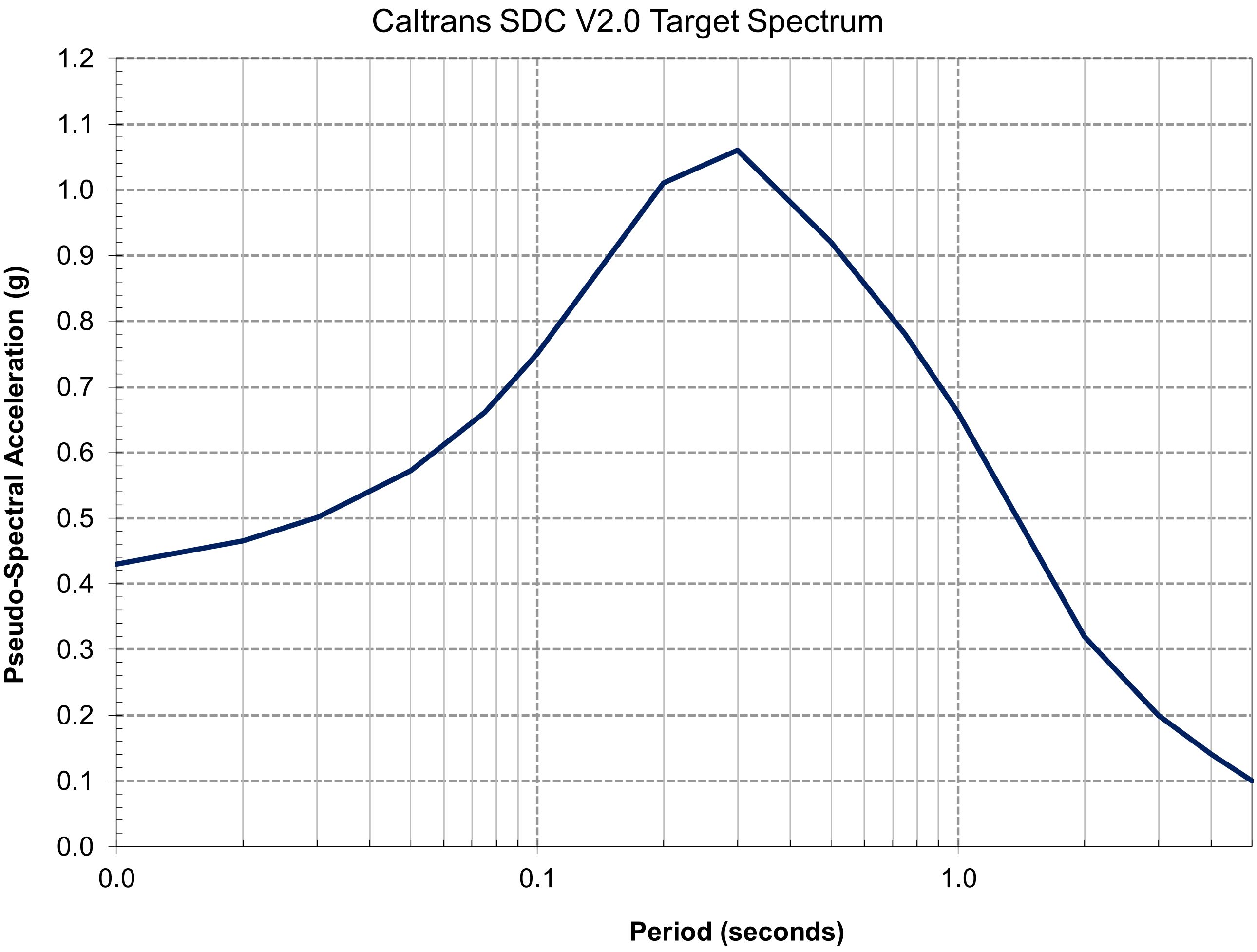
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	SCPT.ppt

SCPT SHEAR WAVE VELOCITY RESULTS

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

FIGURE

F-1



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

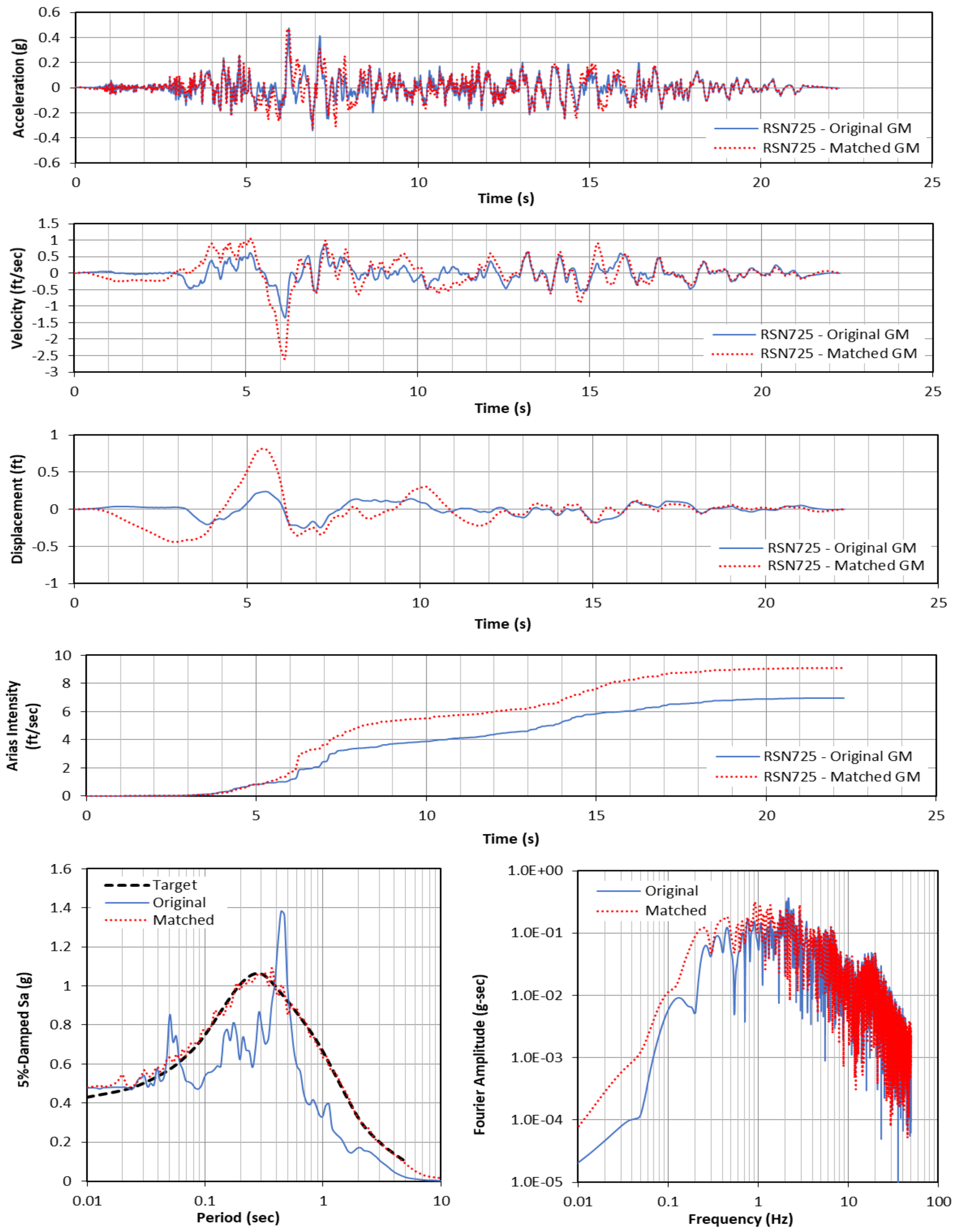
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	TargetSpectrum.ppt

TARGET SPECTRUM FOR SITE RESPONSE ANALYSIS

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

FIGURE

F-2



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

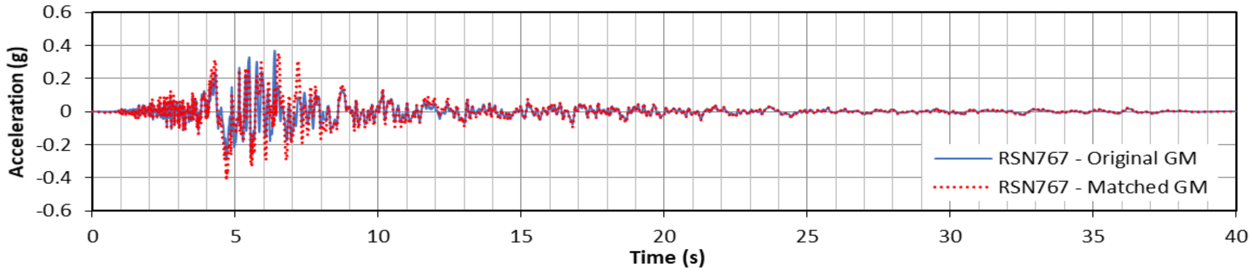
PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM1 Plots.ppt

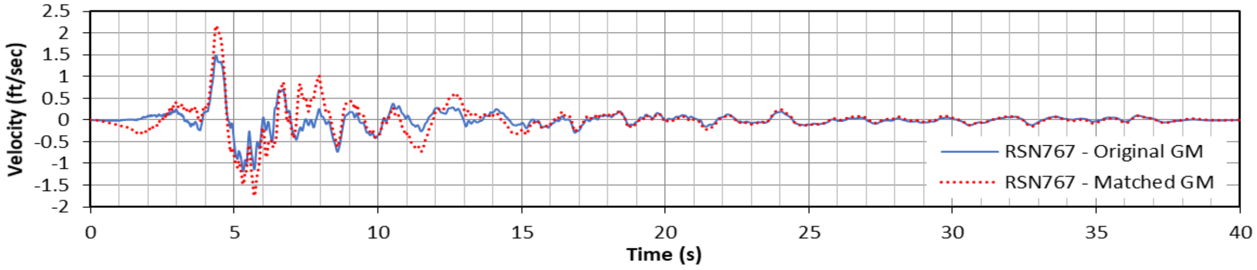
SPECTRAL MATCHING RESULTS
RSN No. 725 (270º)
SUPERSTITION HILLS 1987

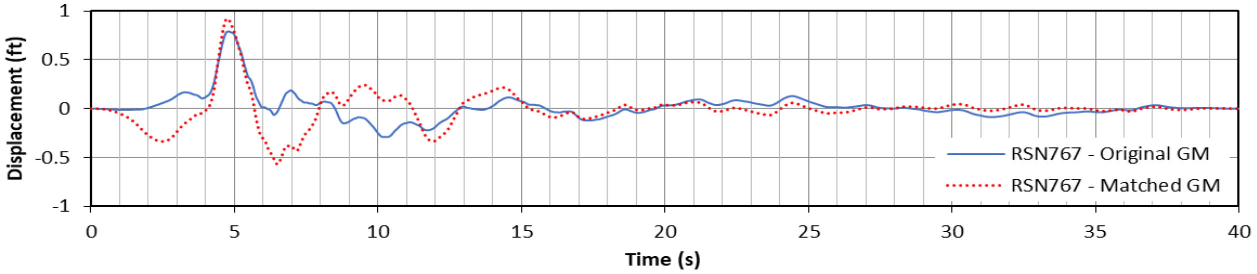
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

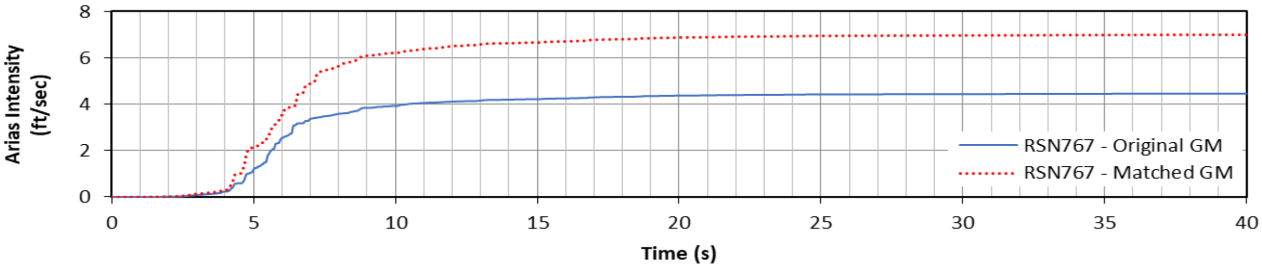
FIGURE

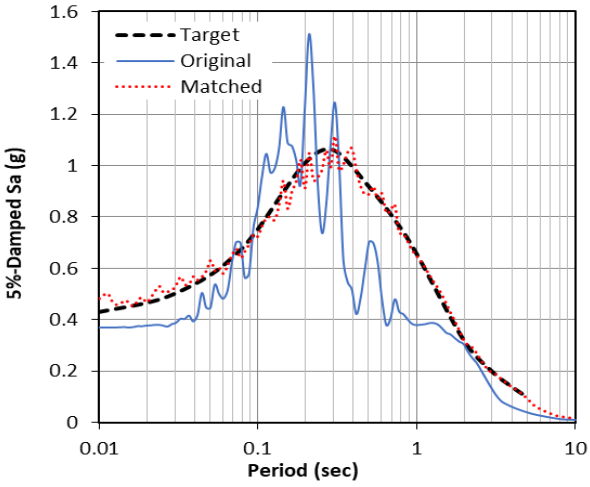
F-3

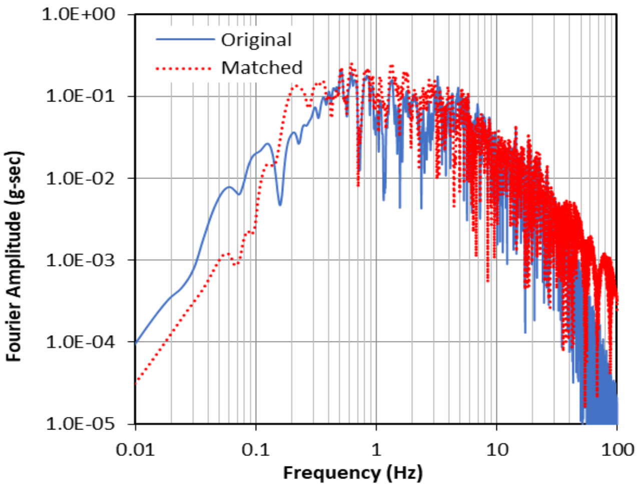












KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

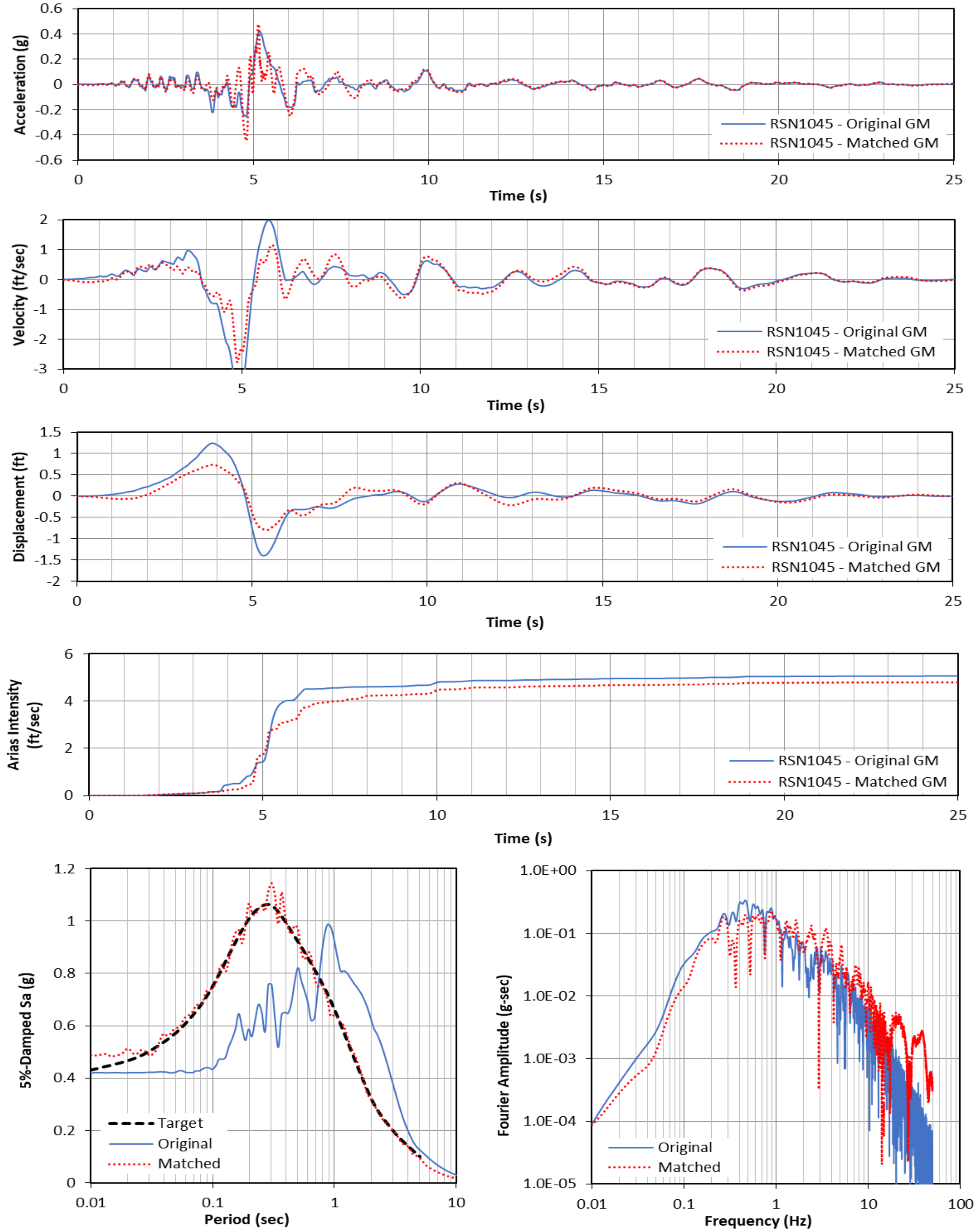
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	GM2 Plots.ppt

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



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

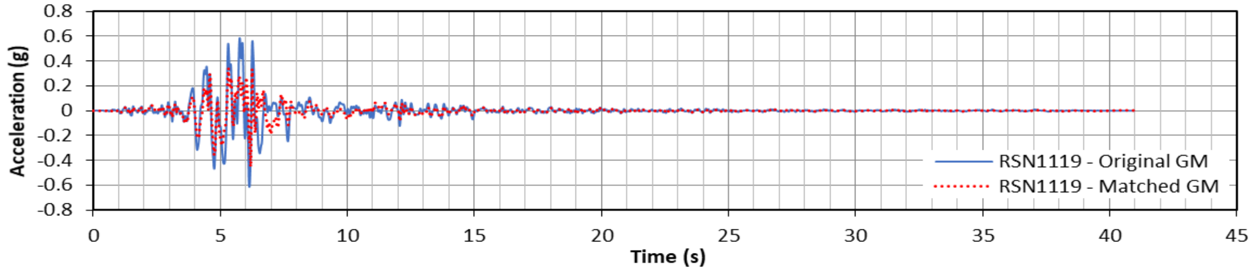
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	GM3 Plots.ppt

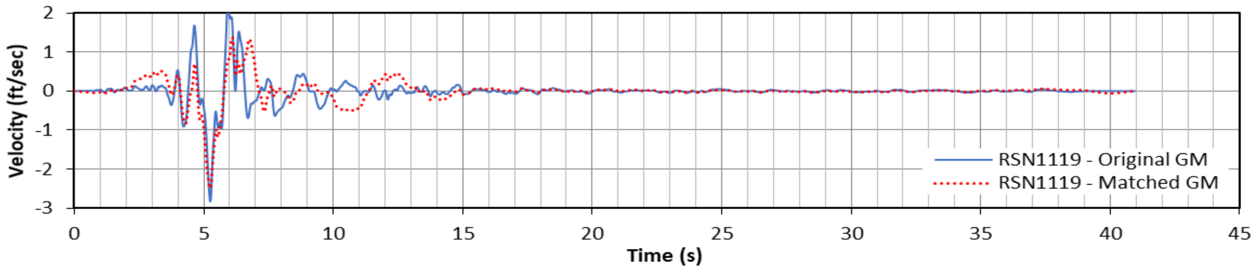
SPECTRAL MATCHING RESULTS
RSN No. 1045 (46º)
NORTHRIDGE 1994

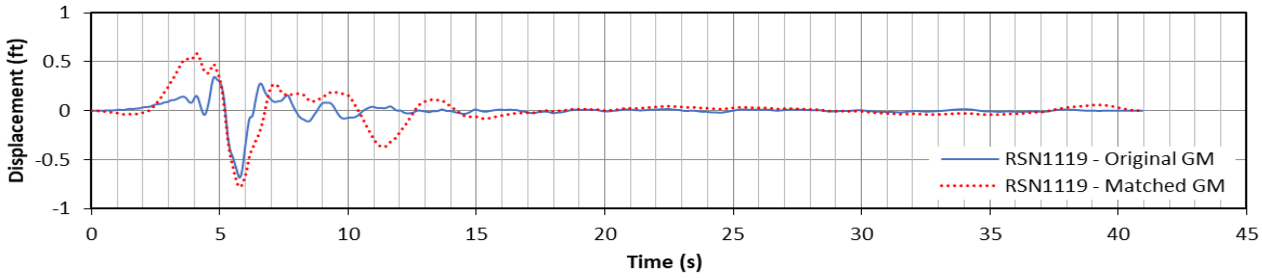
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

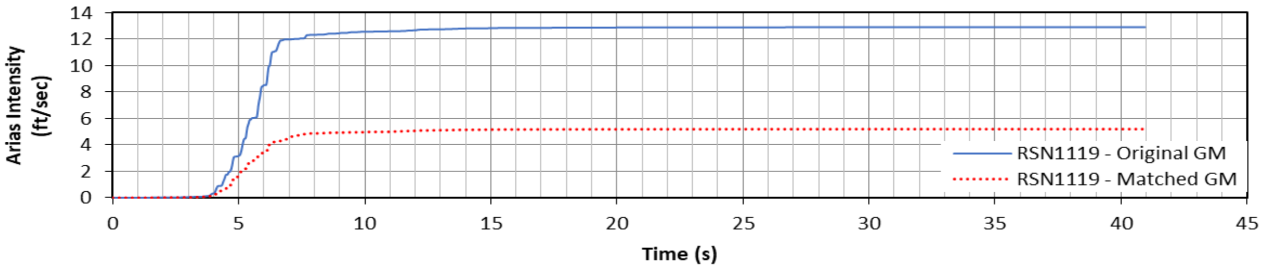
FIGURE

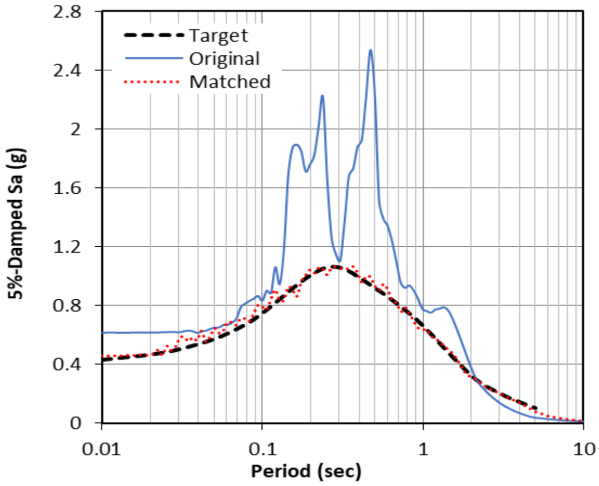
F-5

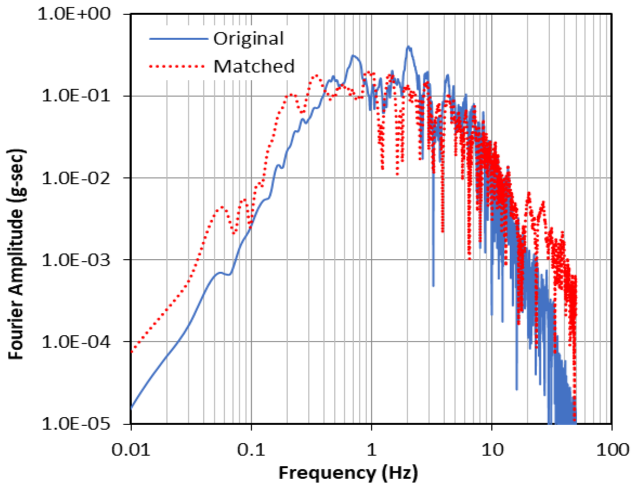












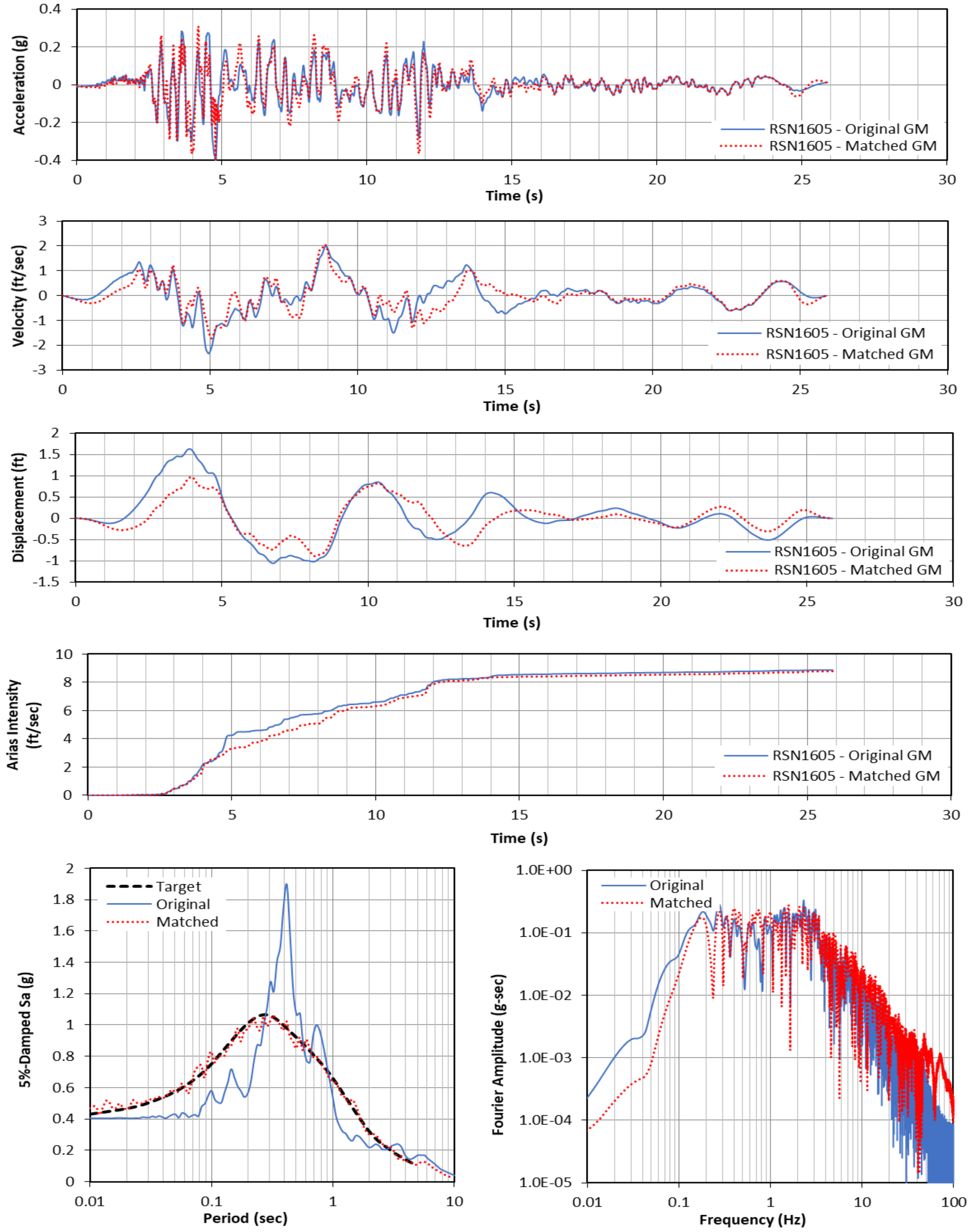
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM4 Plots.ppt

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



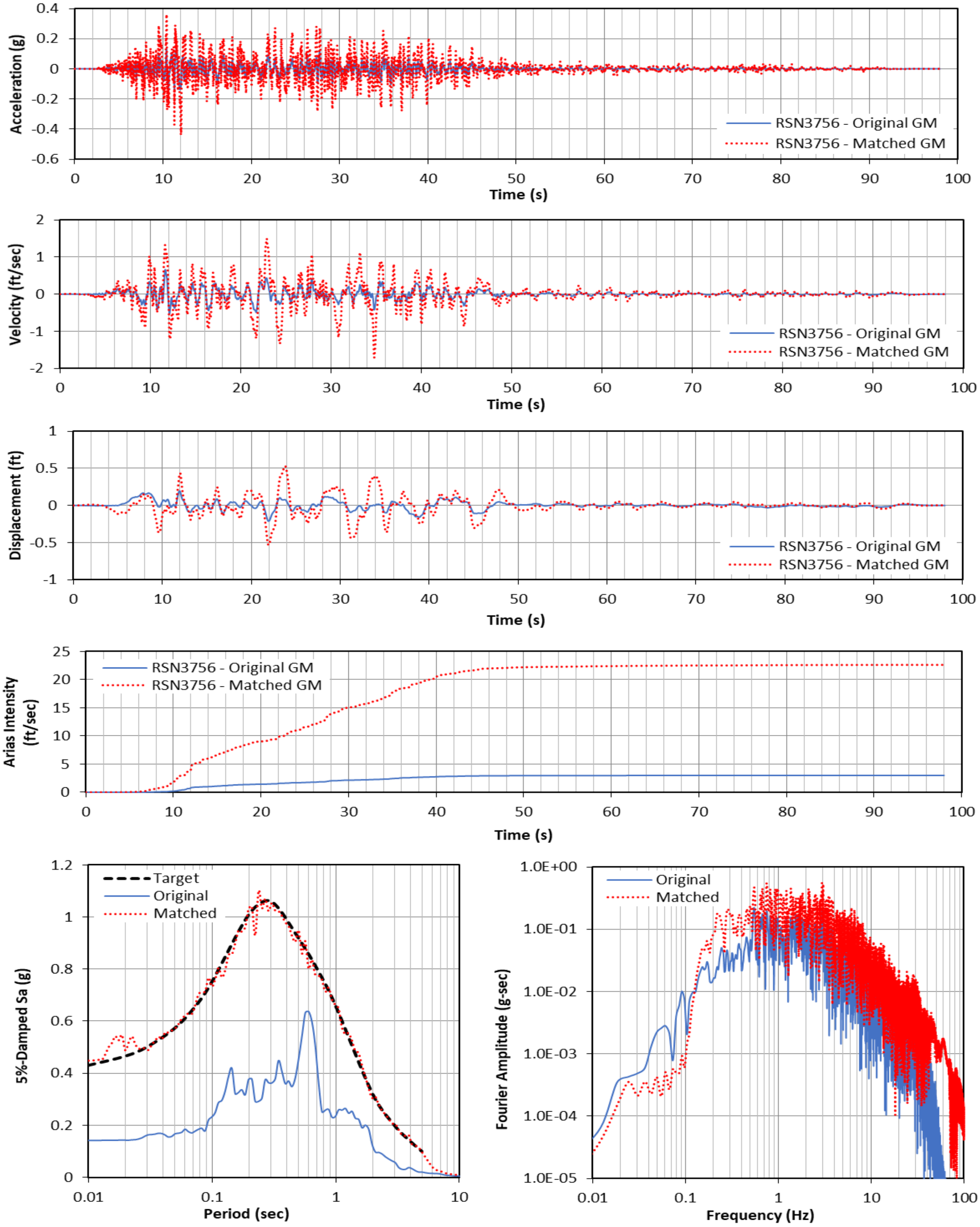
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM5 Plots.ppt

SPECTRAL MATCHING RESULTS
RSN No. 1605 (180^o)
DUZCE, TURKEY 1999

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

FIGURE
F-7



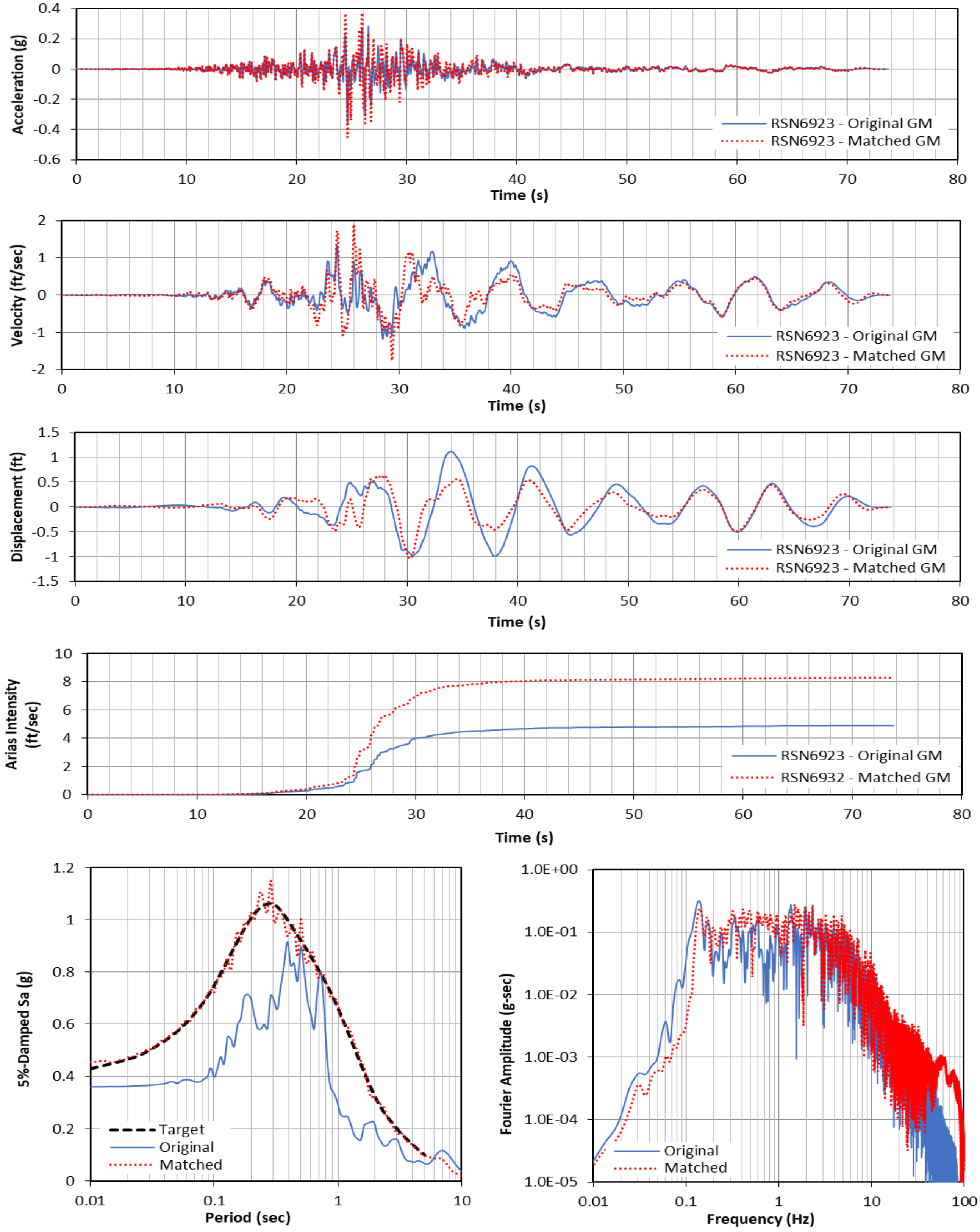
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM6 Plots.ppt

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



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

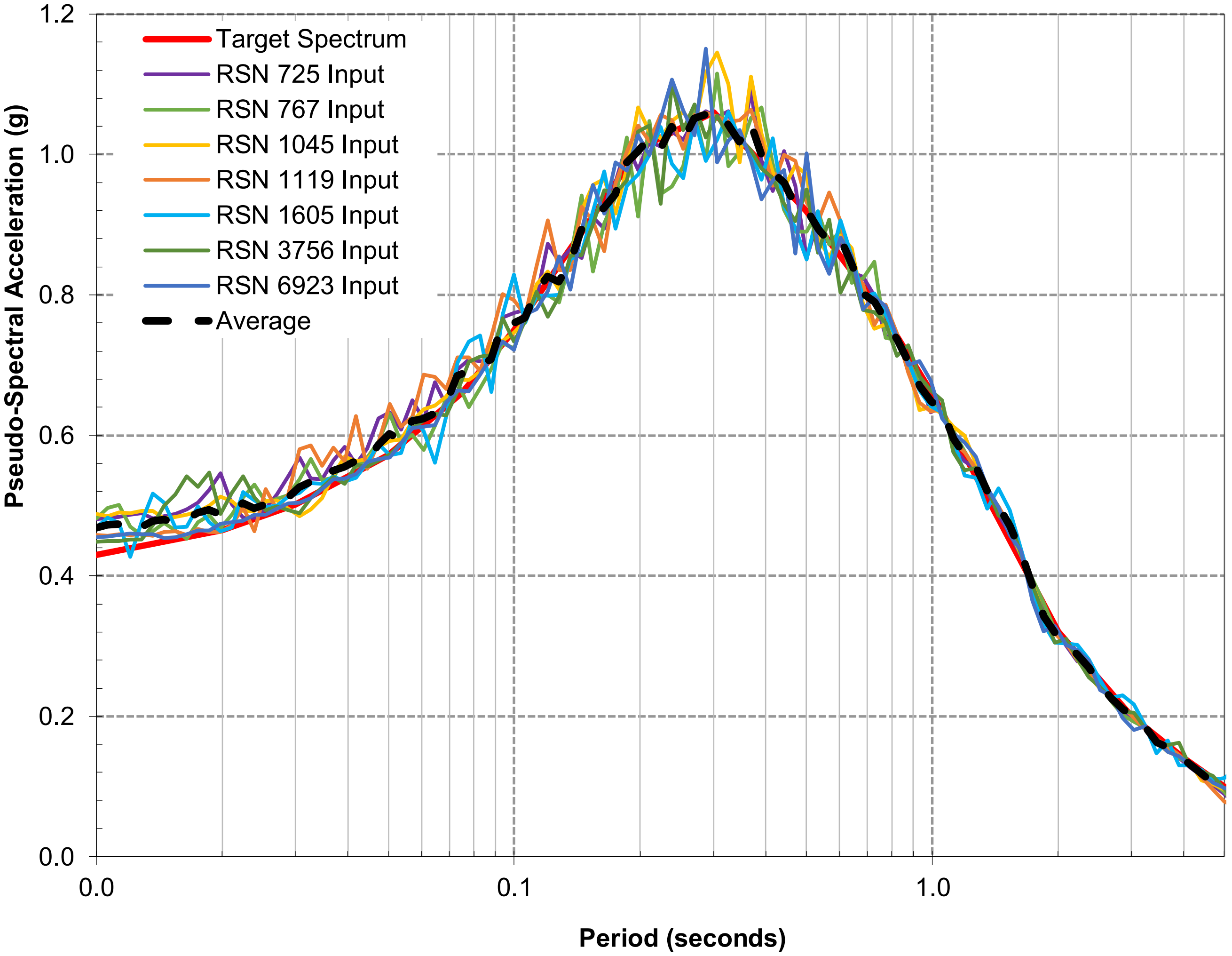
PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM7 Plots.ppt

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

Matched Spectra to Target Spectrum



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

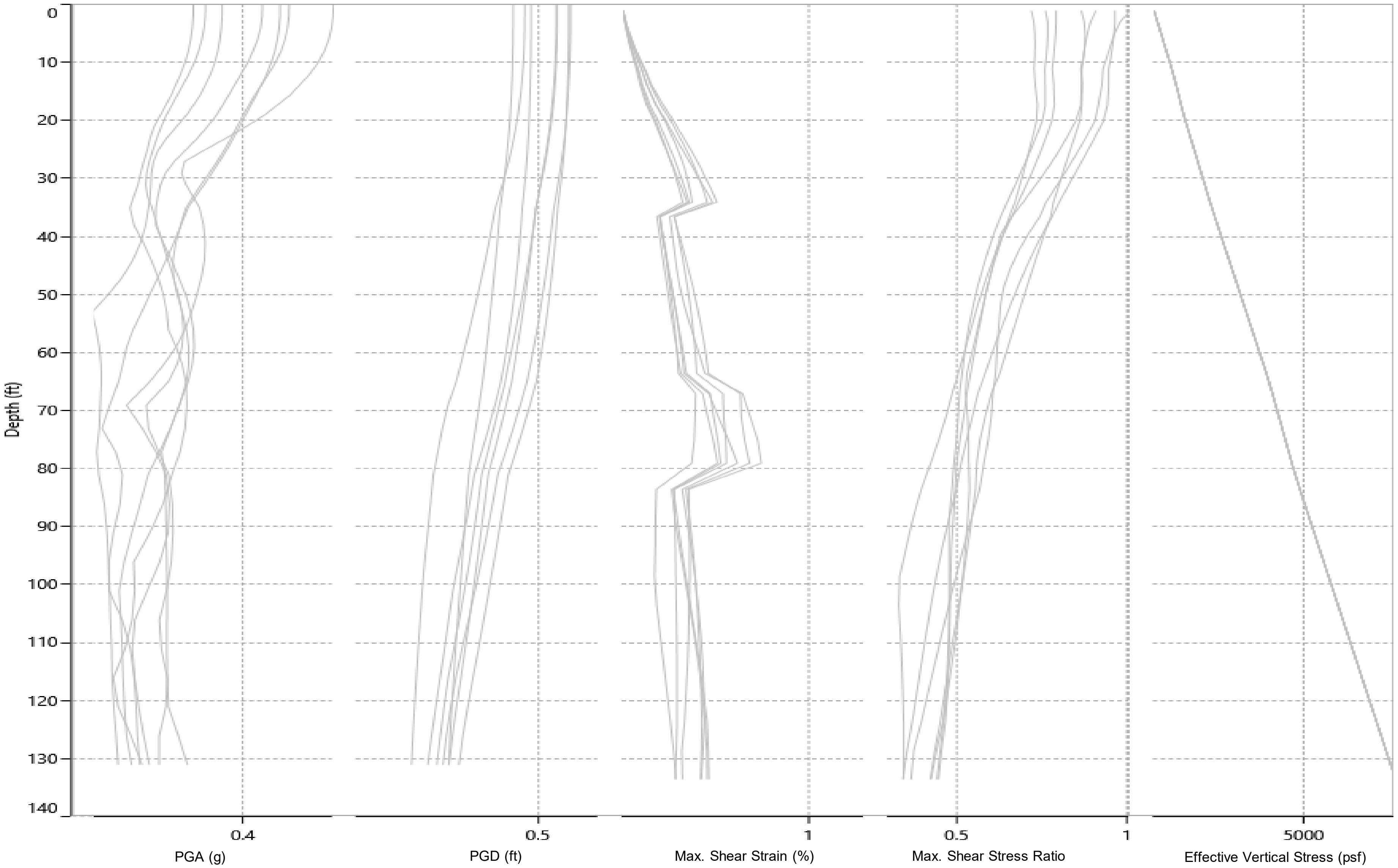
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	InputSpectra.ppt

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



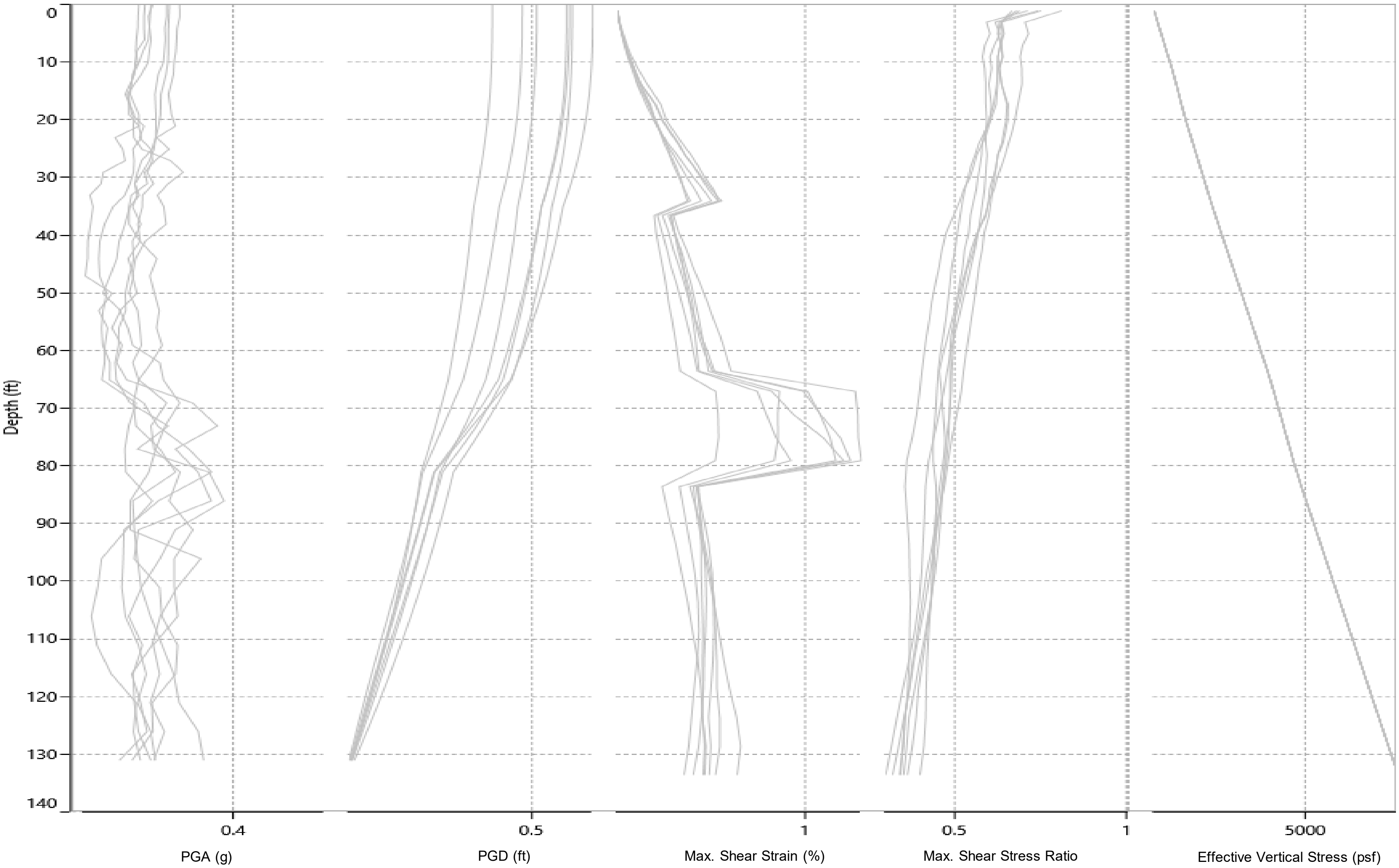
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
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



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

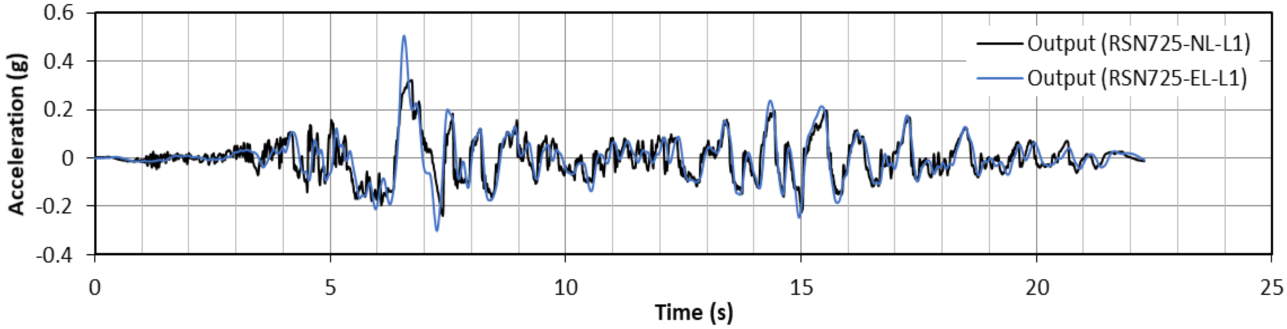
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	Results Prof Plots.ppt

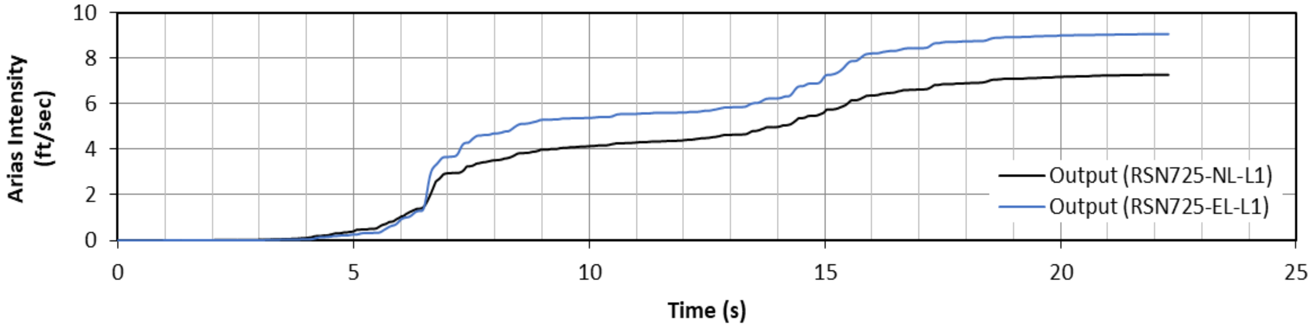
NON-LINEAR SITE RESPONSE RESULTS VS DEPTH GM1 TO GM7

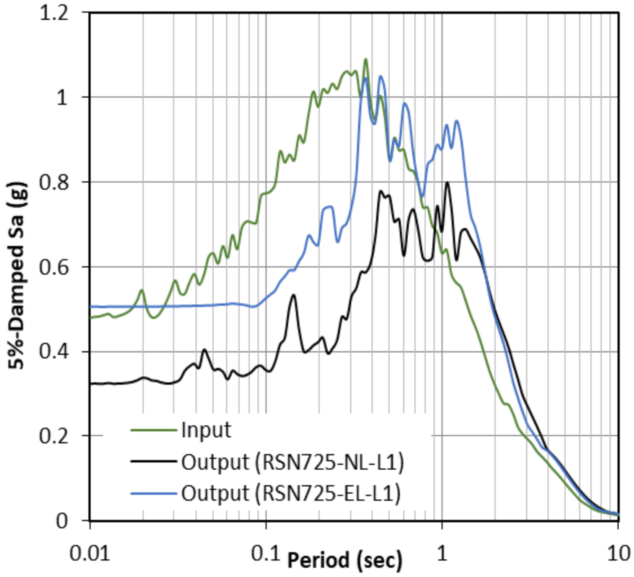
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER – PHASE 0
DEL MAR, CALIFORNIA

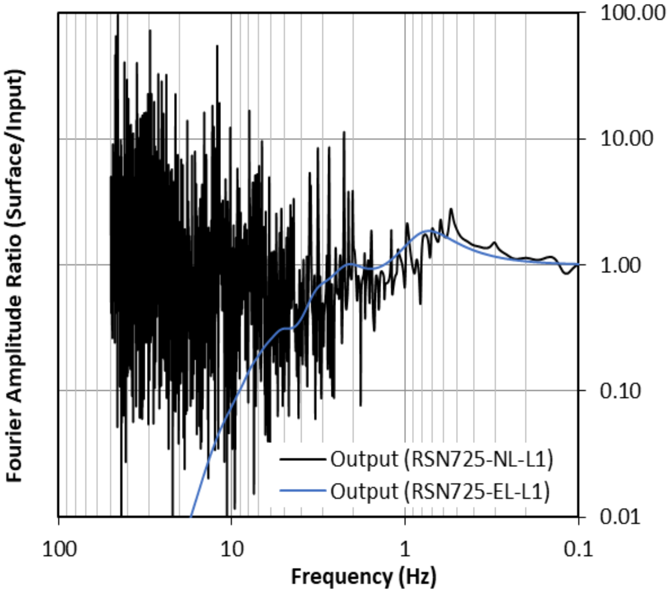
FIGURE

F-12









Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

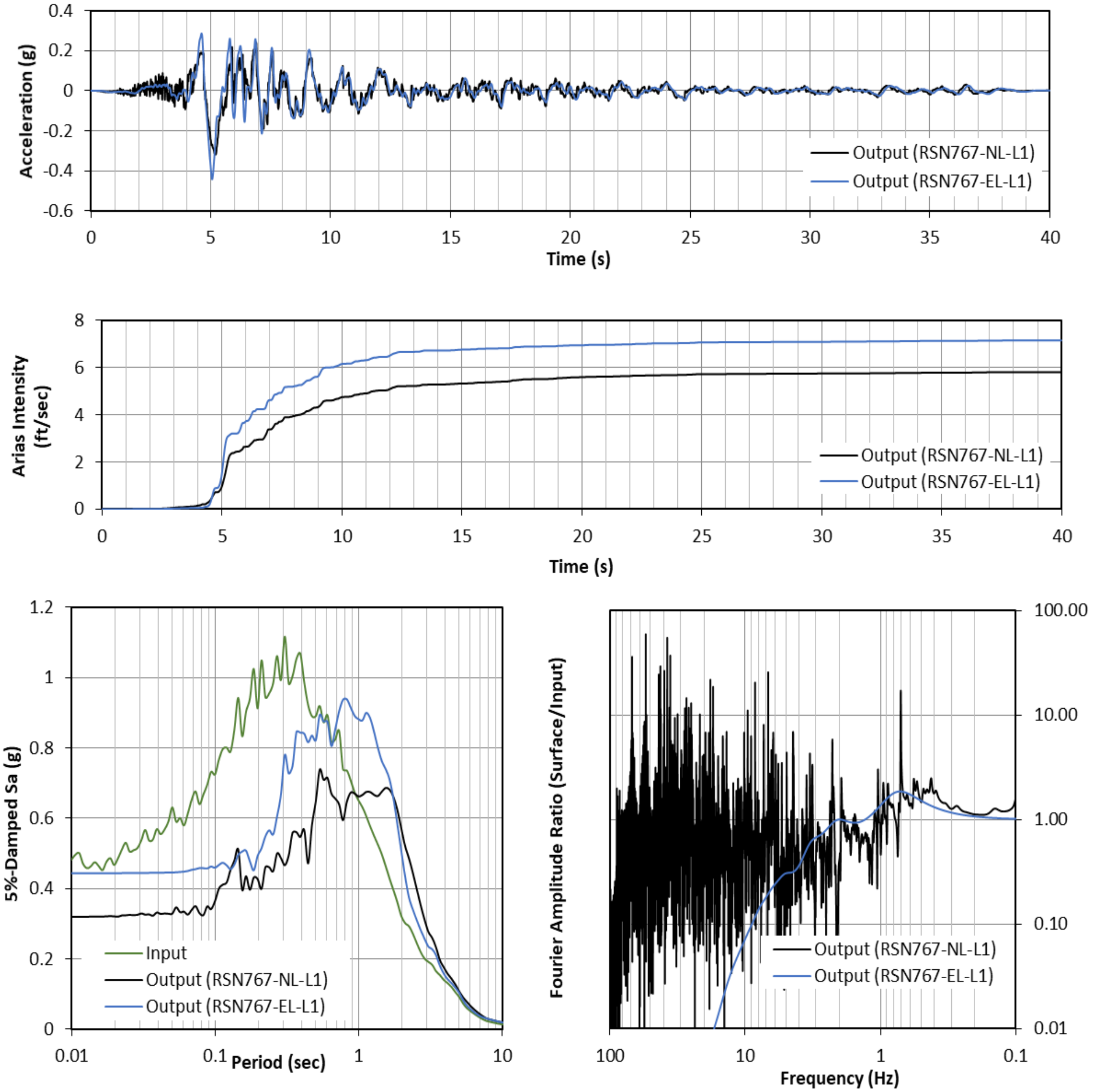
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

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

KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

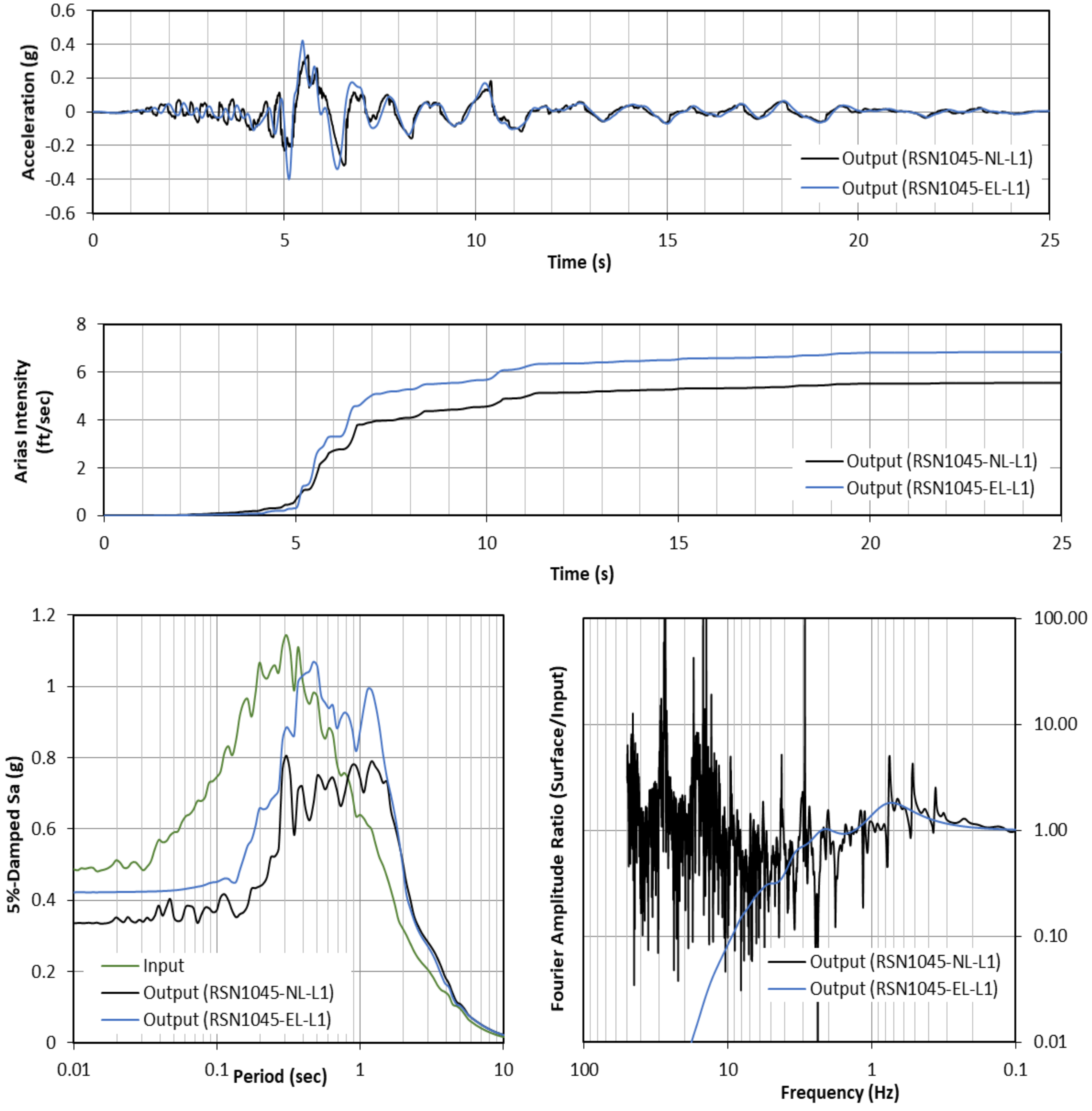
PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM2 Response.ppt

**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

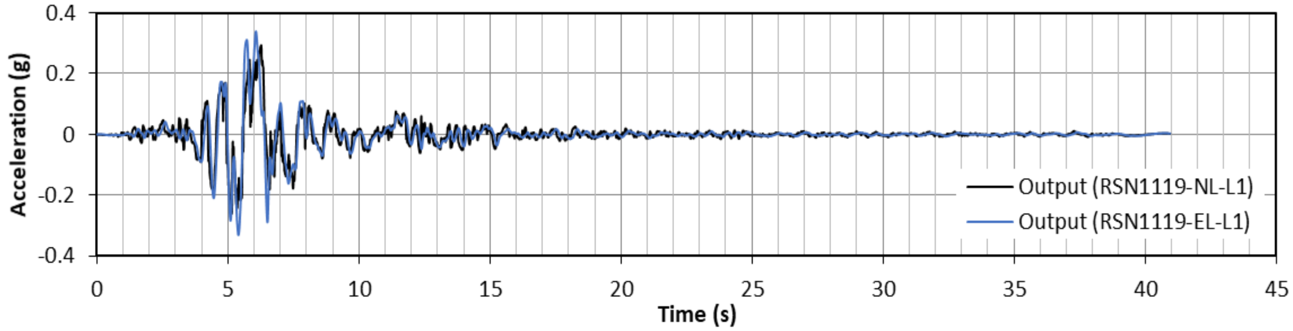
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

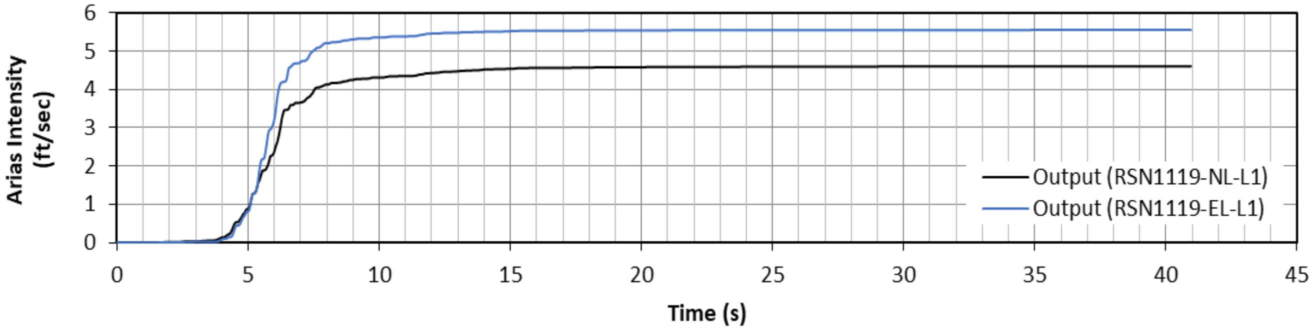
PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM3 Response.ppt

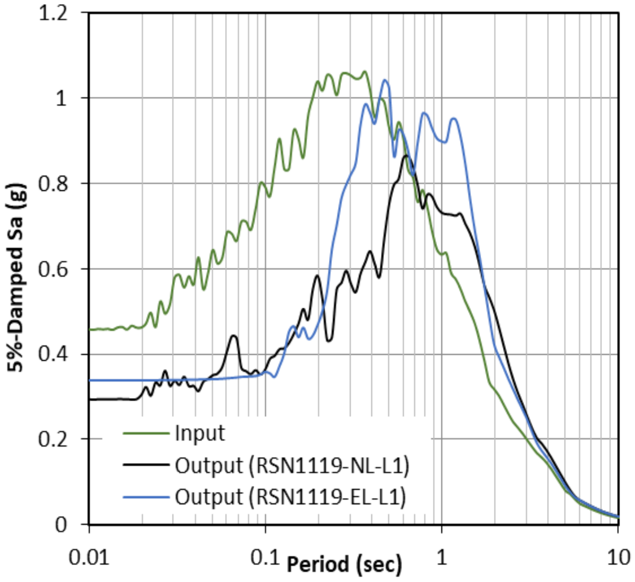
GROUND MOTION RSN1045
RESPONSE AT GROUND SURFACE

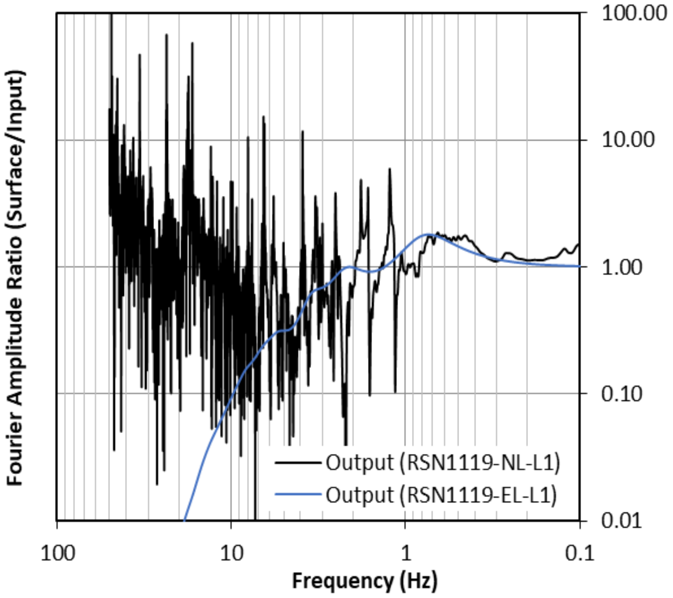
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-15









Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

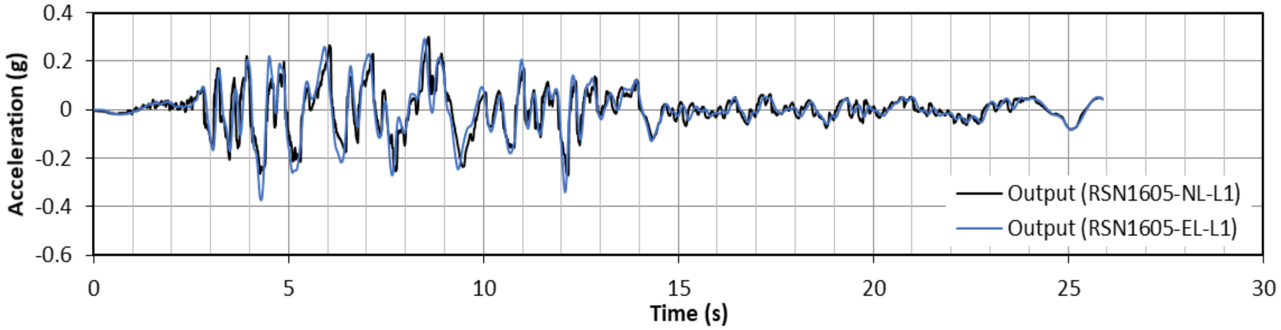
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

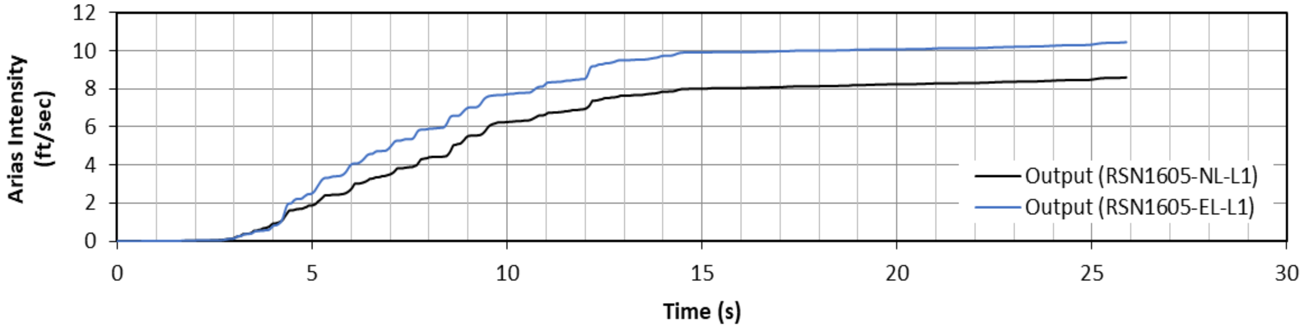
PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM4 Response.ppt

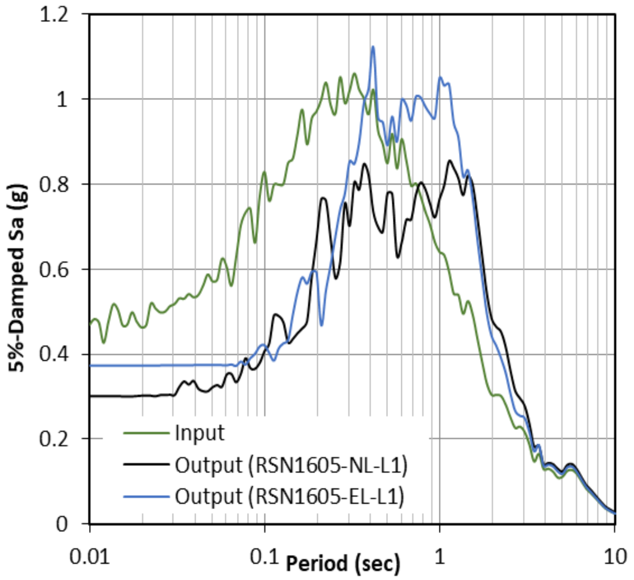
GROUND MOTION RSN1119
RESPONSE AT GROUND SURFACE

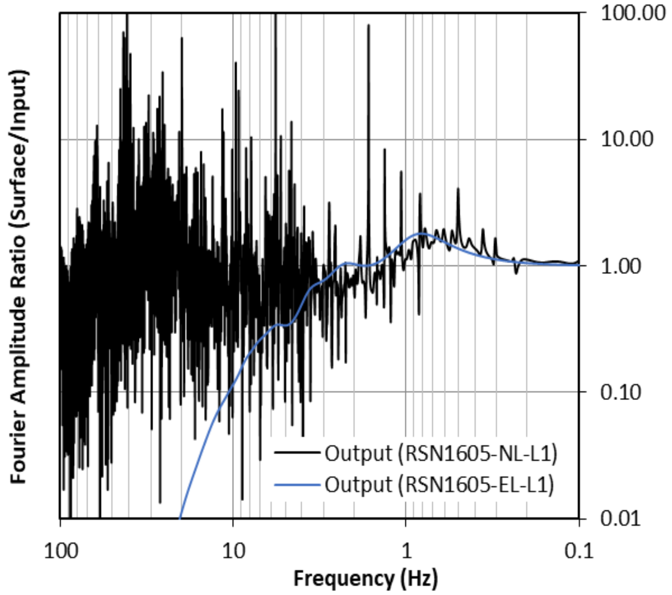
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

FIGURE
F-16









Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

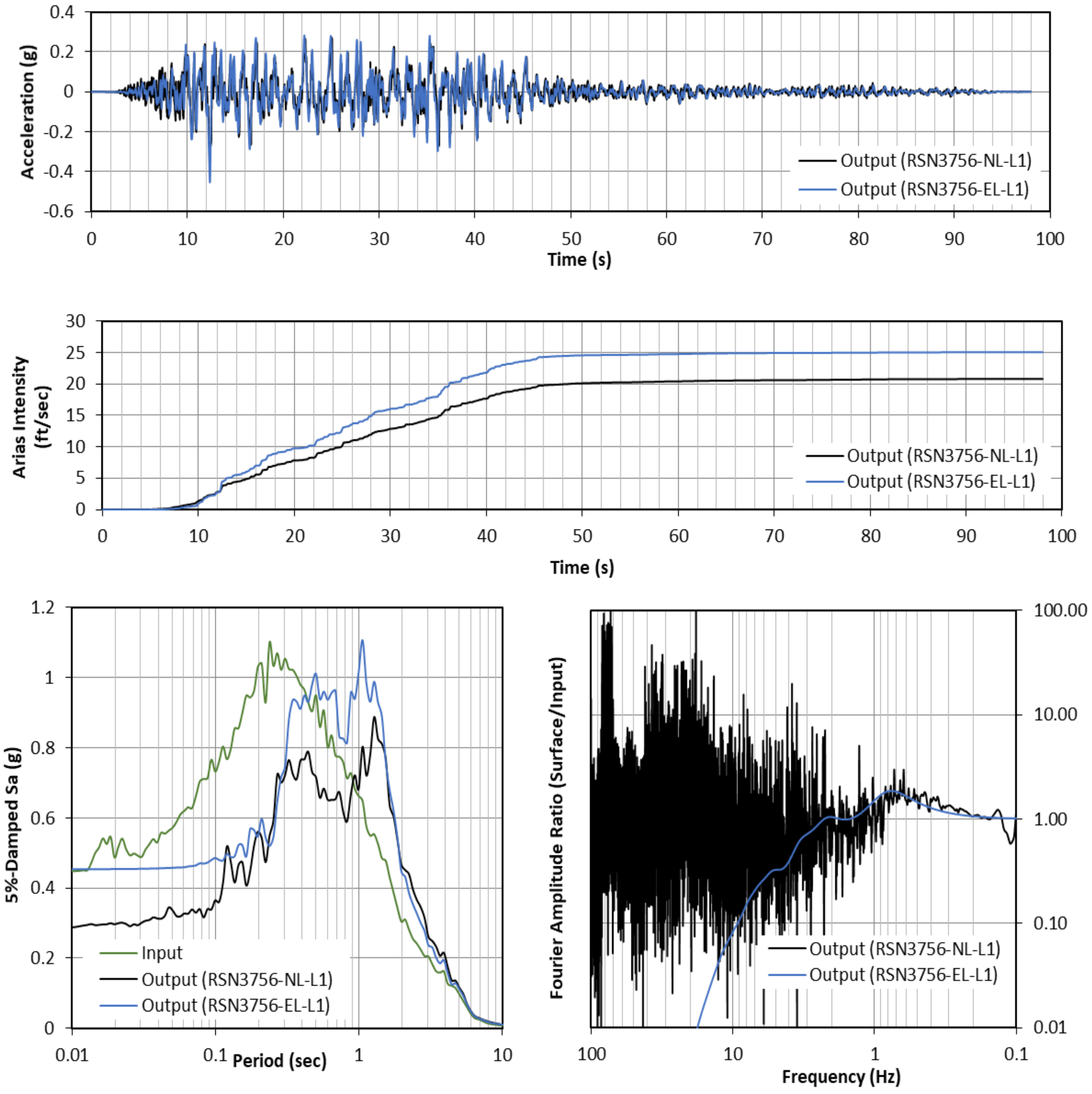
PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM5 Response.ppt

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

KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

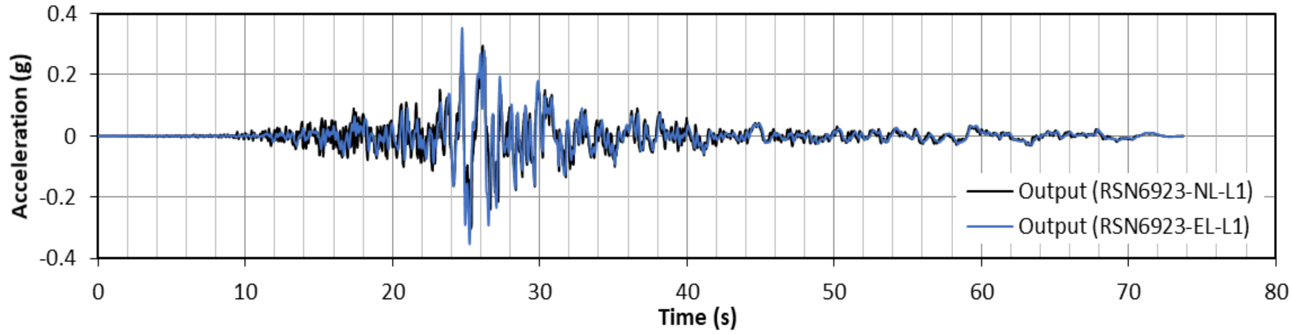
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	GM6 Response.ppt

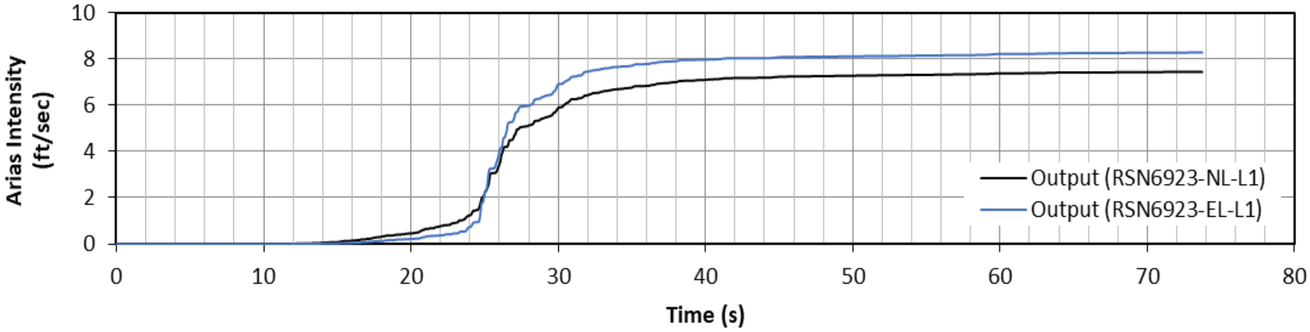
GROUND MOTION RSN3756
RESPONSE AT GROUND SURFACE

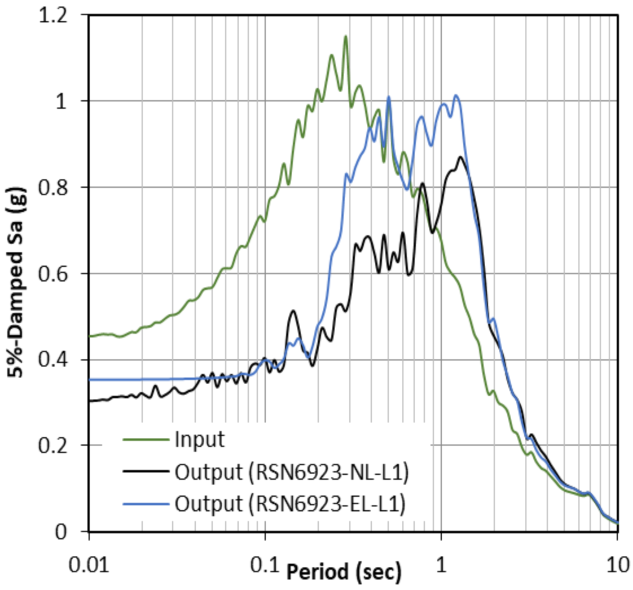
CAMINO DEL MAR BRIDGE REPLACEMENT
OVER SAN DIEGUITO RIVER - PHASE 0
DEL MAR, CALIFORNIA

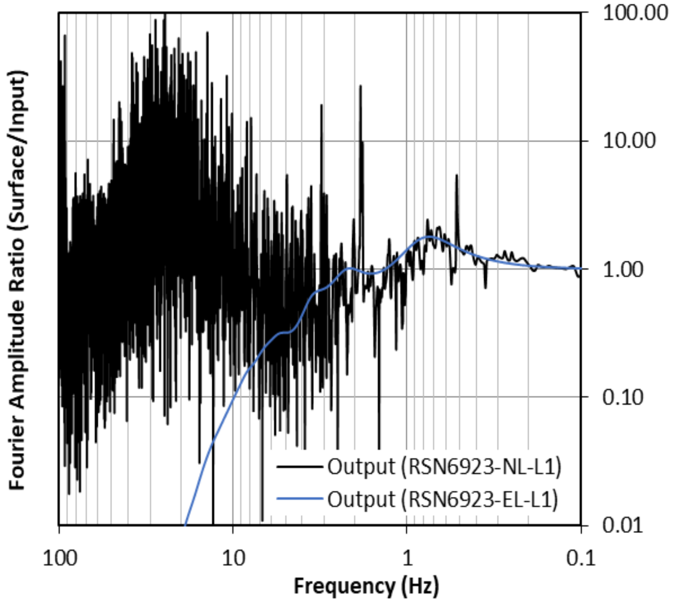
FIGURE

F-18









Notes: NL = Non-Linear; EL = Equivalent Linear; L1 = Layer 1

KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO. 20180876.001A
DATE: 06/2020
DRAWN: JLB
CHECKED BY: ZZ
File Name: GM7 Response.ppt

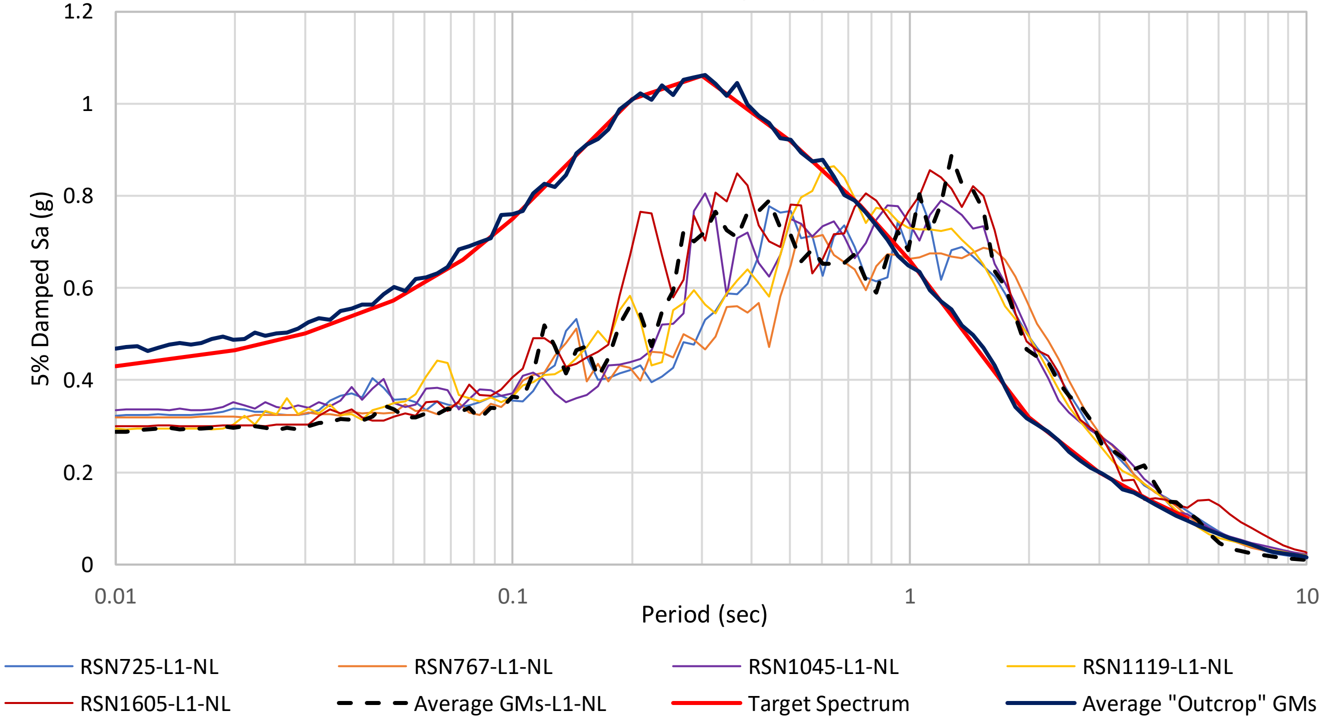
GROUND MOTION RSN6923
RESPONSE AT GROUND SURFACE

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

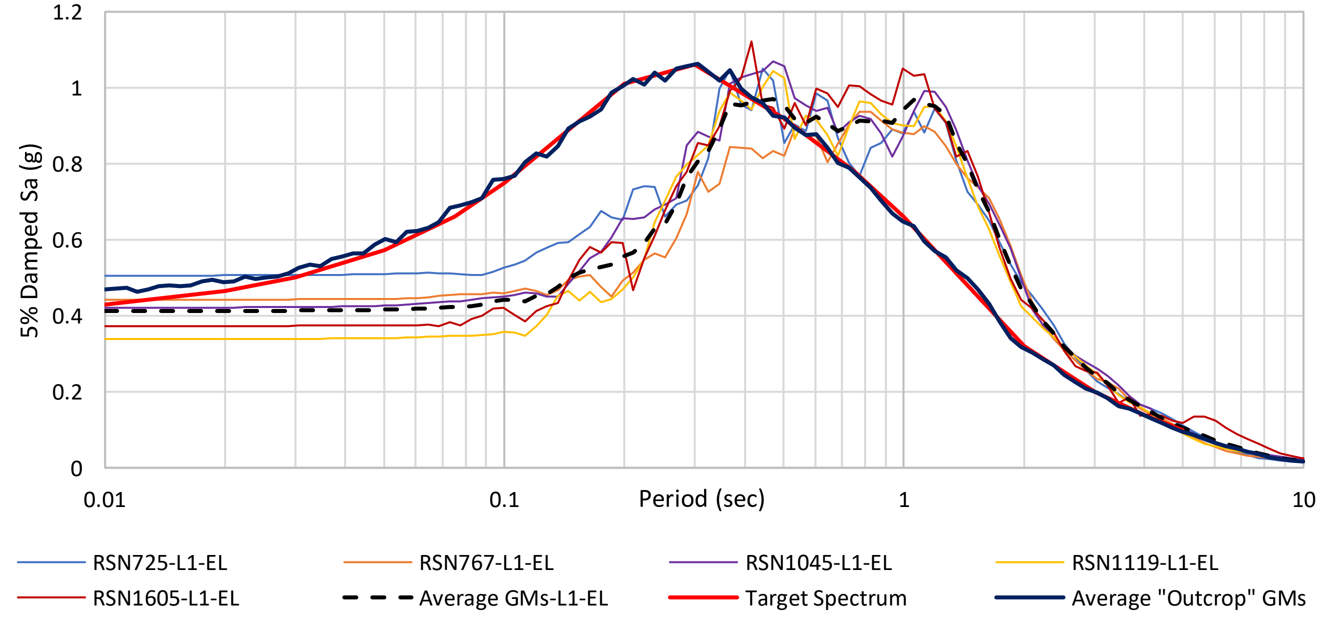
FIGURE

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



KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

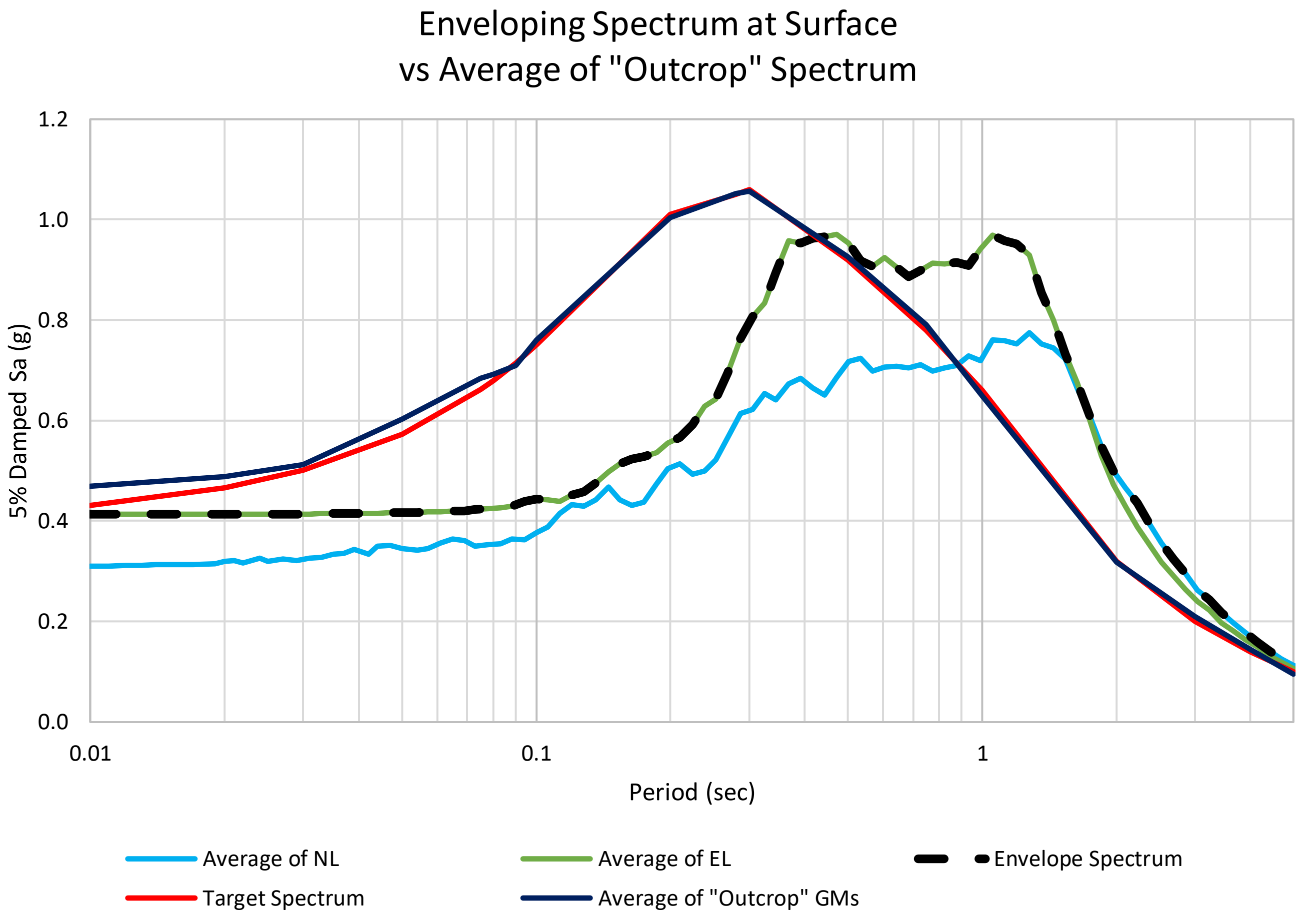
PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	SiteResults.ppt

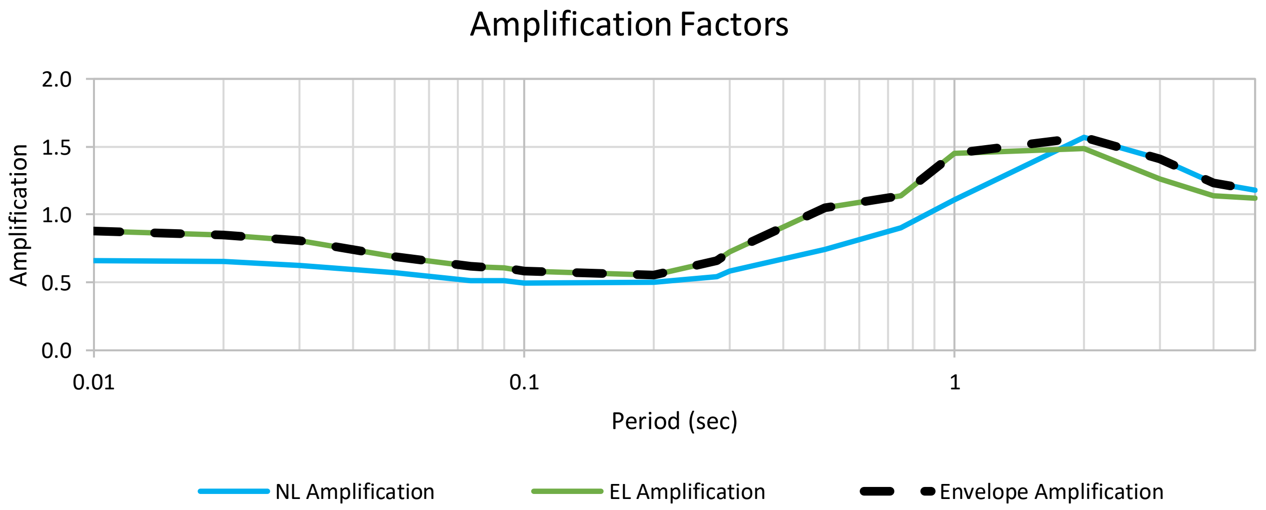
AVERAGE SITE RESPONSE RESULTS

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

FIGURE

F-20





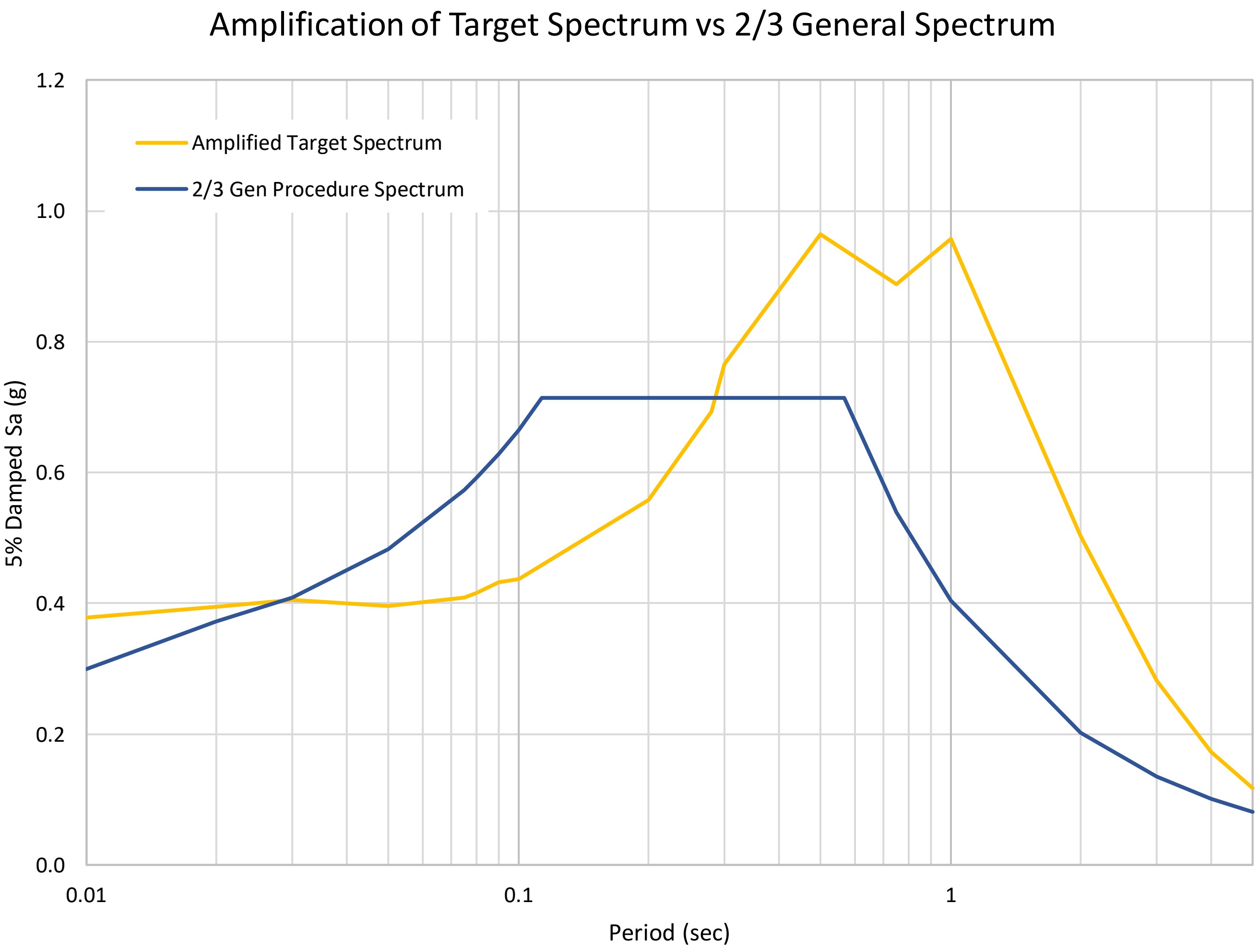
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO. 20180876.001A
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



PROJECT NO.	20180876.001A
DATE:	06/2020
DRAWN:	JLB
CHECKED BY:	ZZ
File Name:	Design Spectrum.ppt

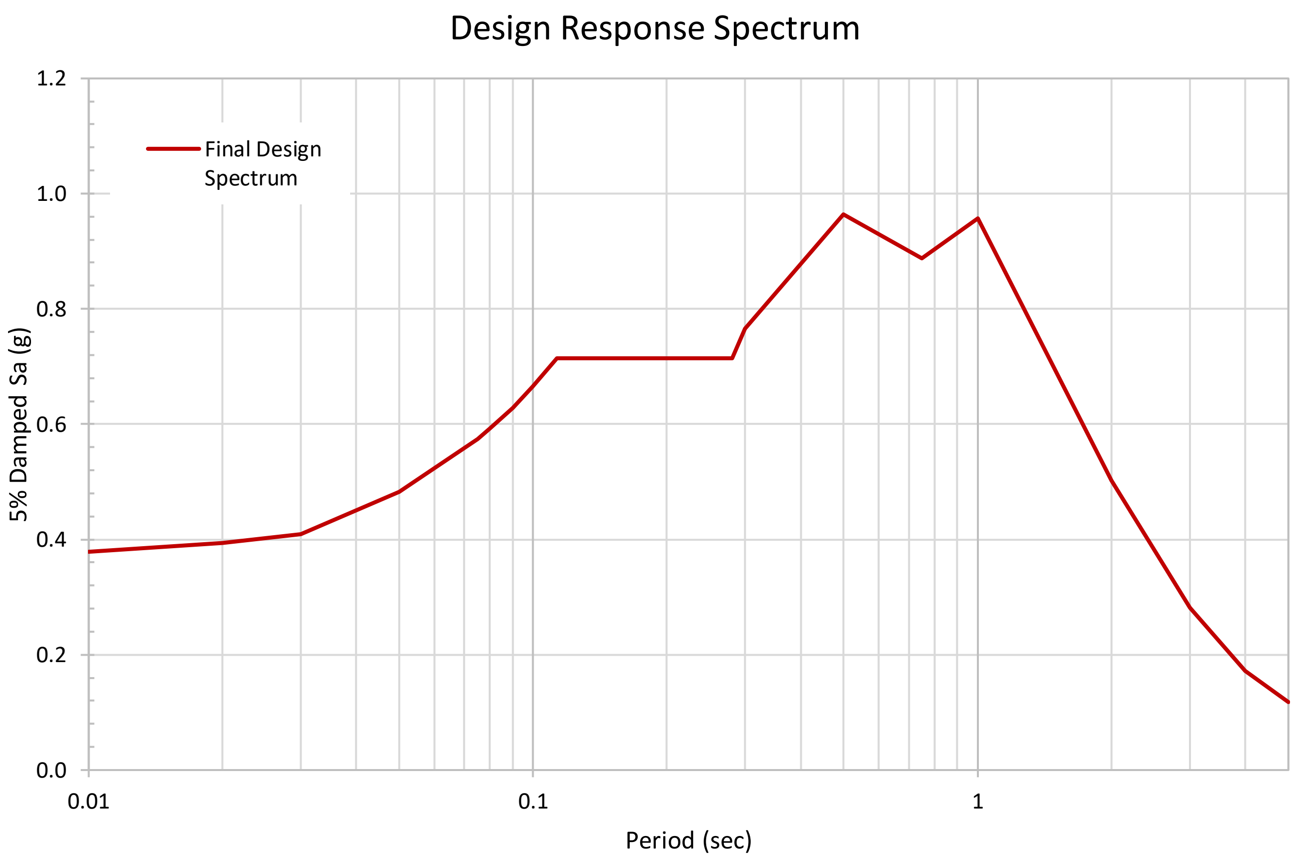
www.kleinfelder.com

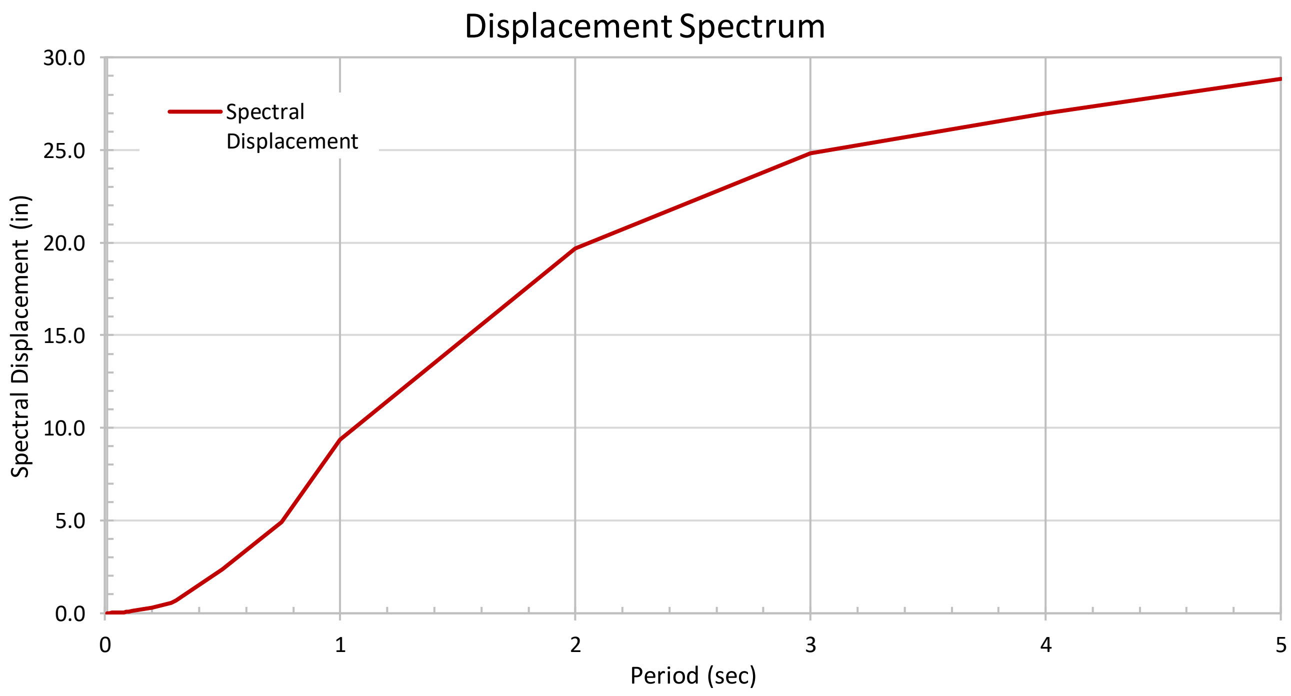
AMPLIFICATION SPECTRUM COMPARISON TO LOWER LIMIT

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

FIGURE

F-22





KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

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 Liquefaction and Seismic Settlement Calculations

G.1 LIQUEFACTION AND SEISMIC SETTLEMENT CALCULATIONS

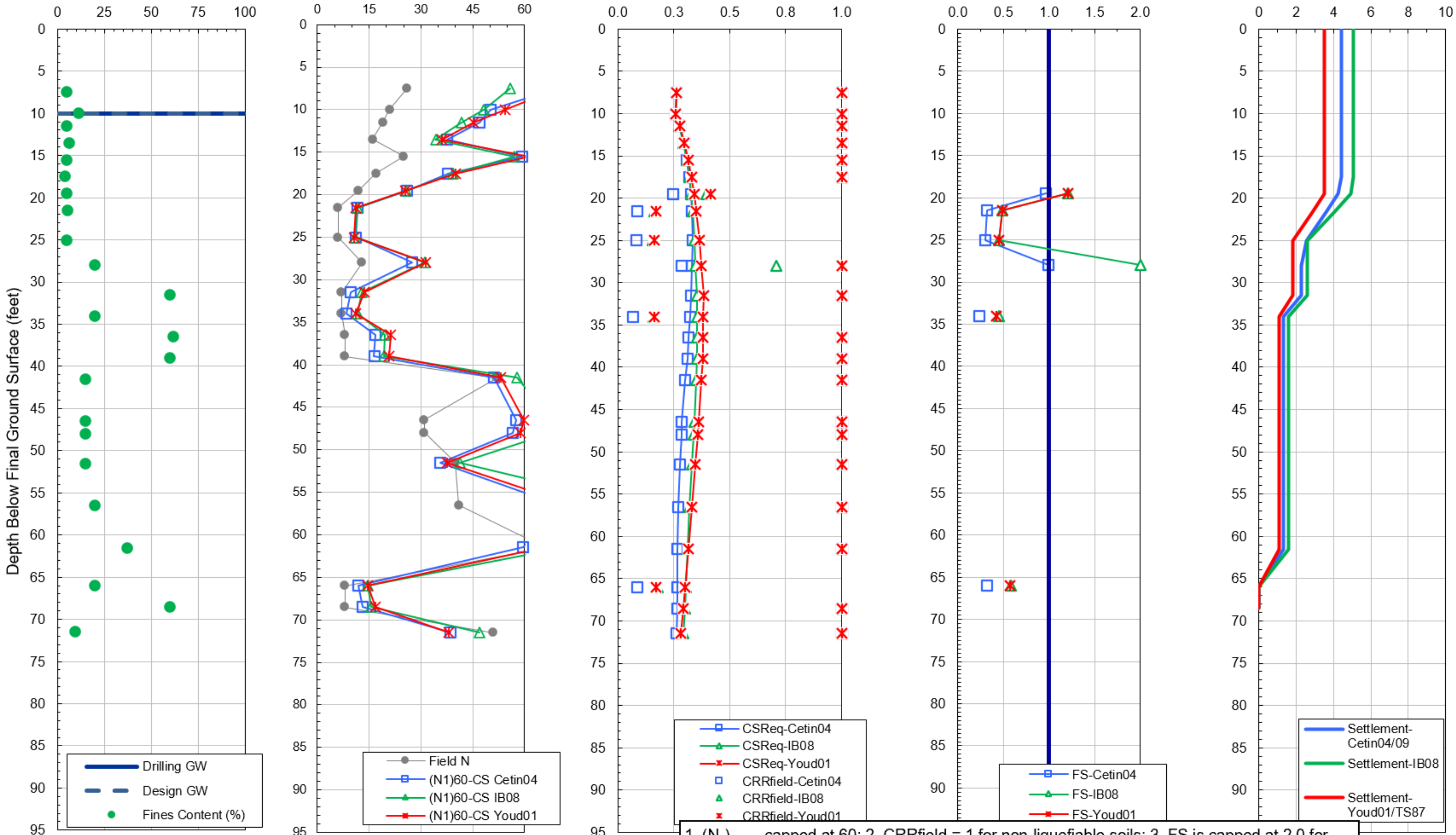
Boring ID: **R-20-001 (0 to 70 ft)**

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.

KLEINFELDER
Bright People. Right Solutions.

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

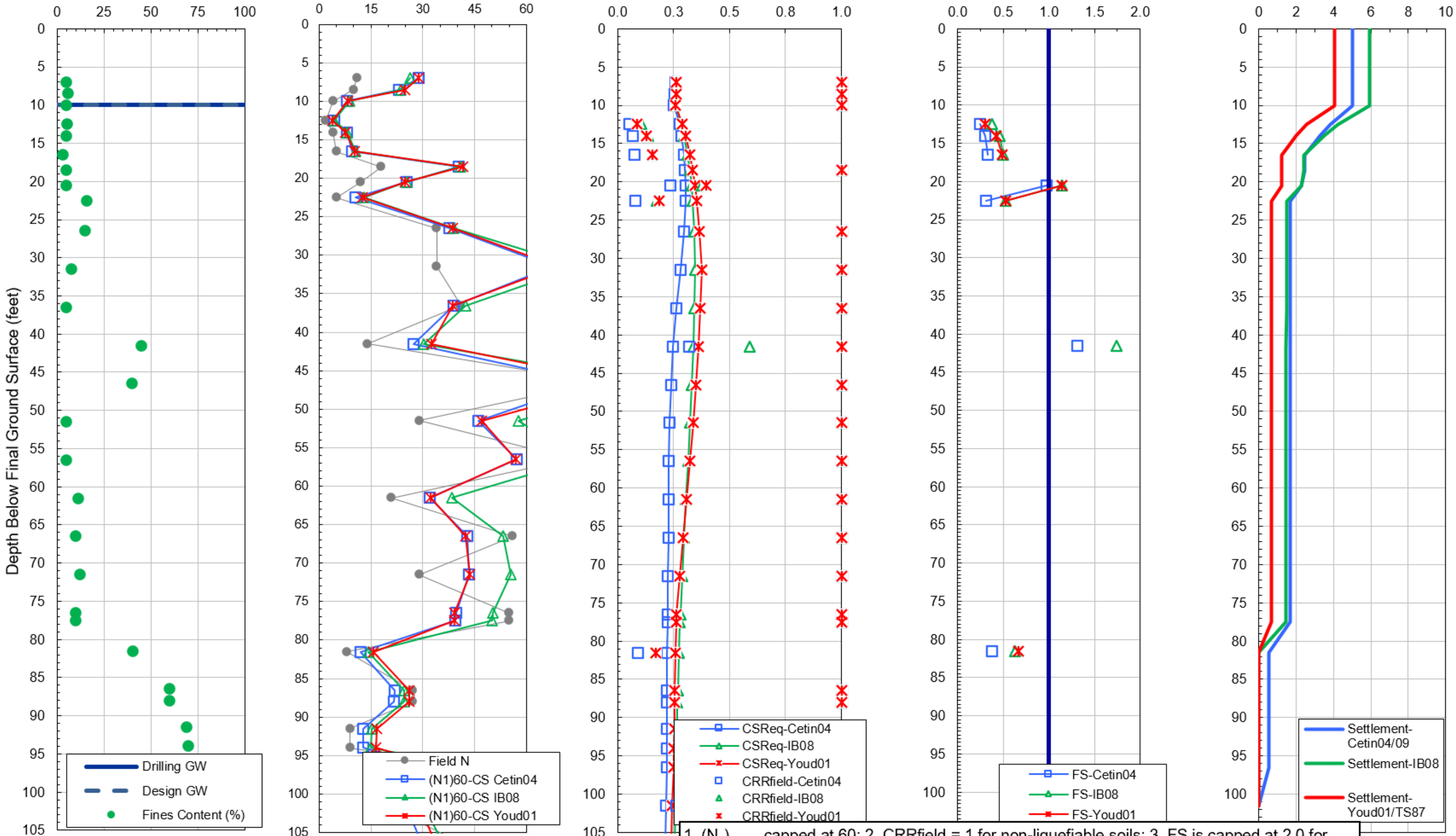
Boring ID: **R-20-002 (0 to 70 ft)**

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.

KLEINFELDER
Bright People. Right Solutions.

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)

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) (%)	Results 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)

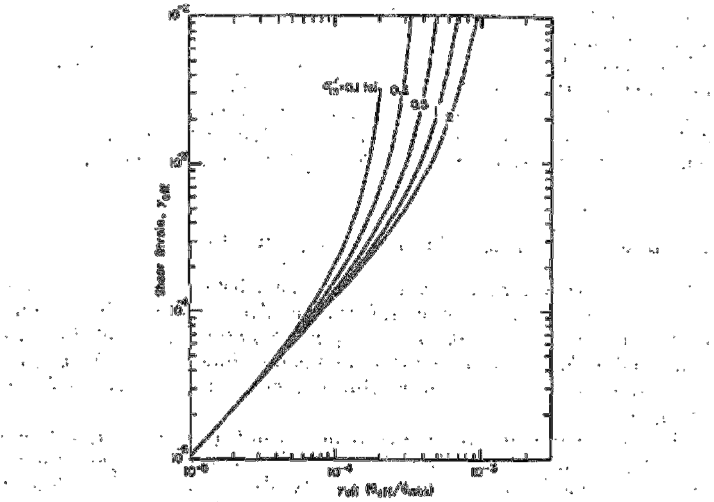


FIG. 11.—Plot for Determination of Induced Strain in Sand Deposits

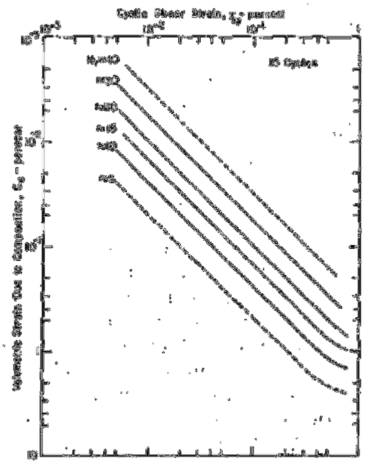


FIG. 13.—Relationship between Volumetric Strain, Shear Strain, and Penetration Resistance for Dry Sands .

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)

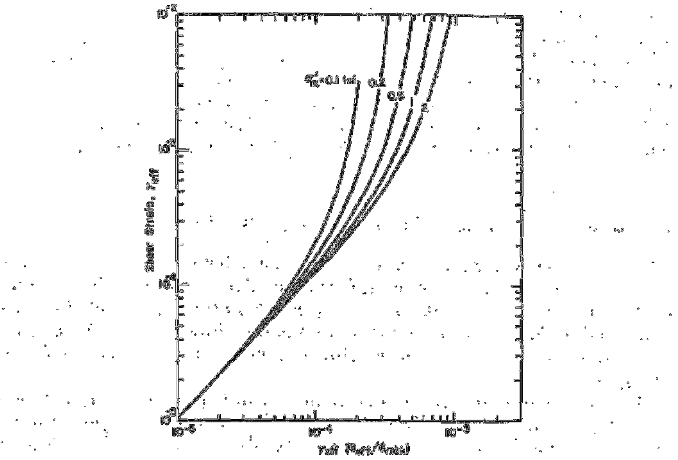


FIG. 11.—Plot for Determination of Induced Strain in Sand Deposits

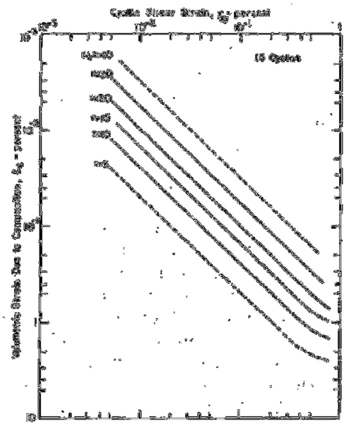
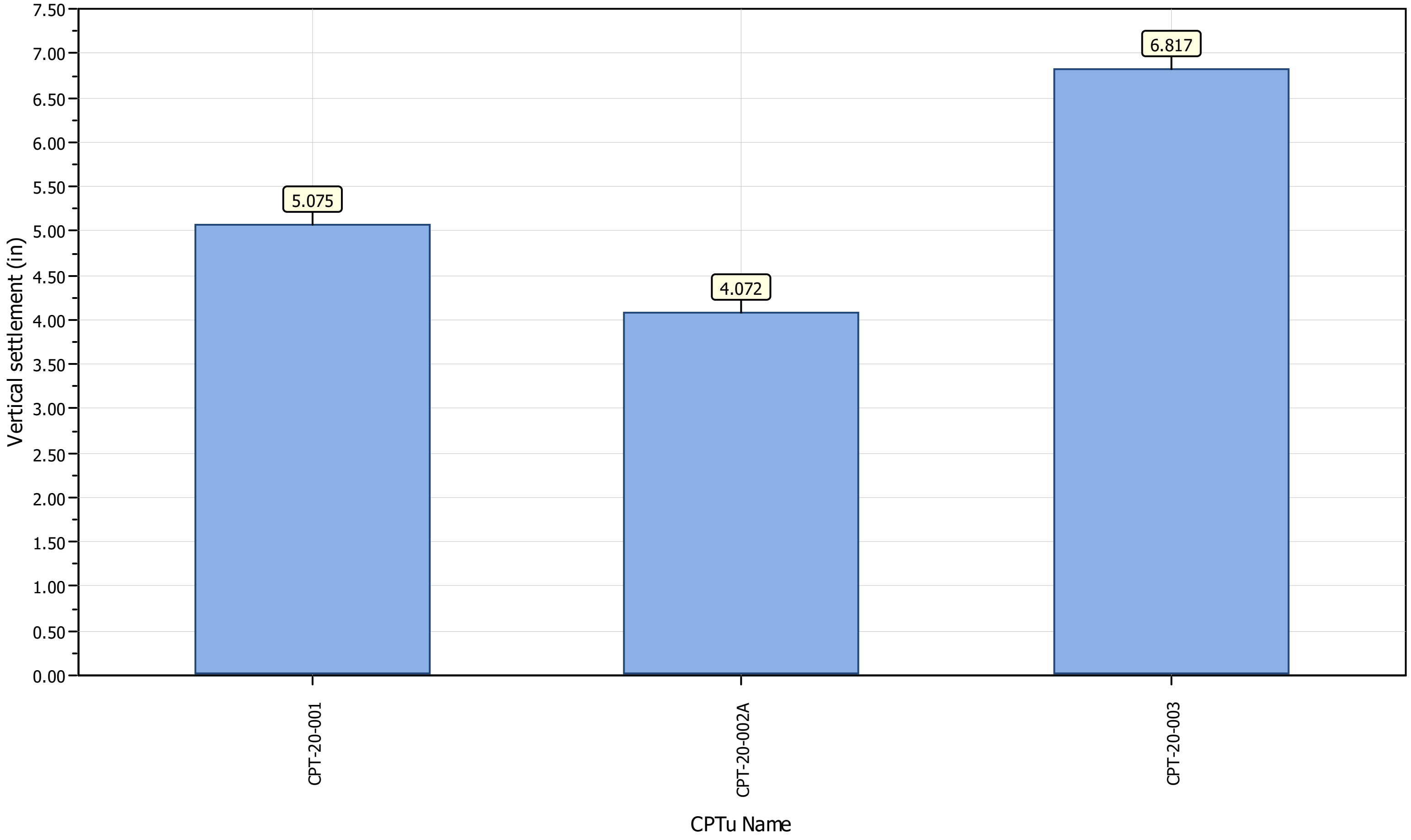


FIG. 13.—Relationship between Volumetric Strain, Shear Strain, and Penetration Resistance for Dry Sands

Project title : Camino Del Mar Bridge Replacement

Location : Del Mar, CA

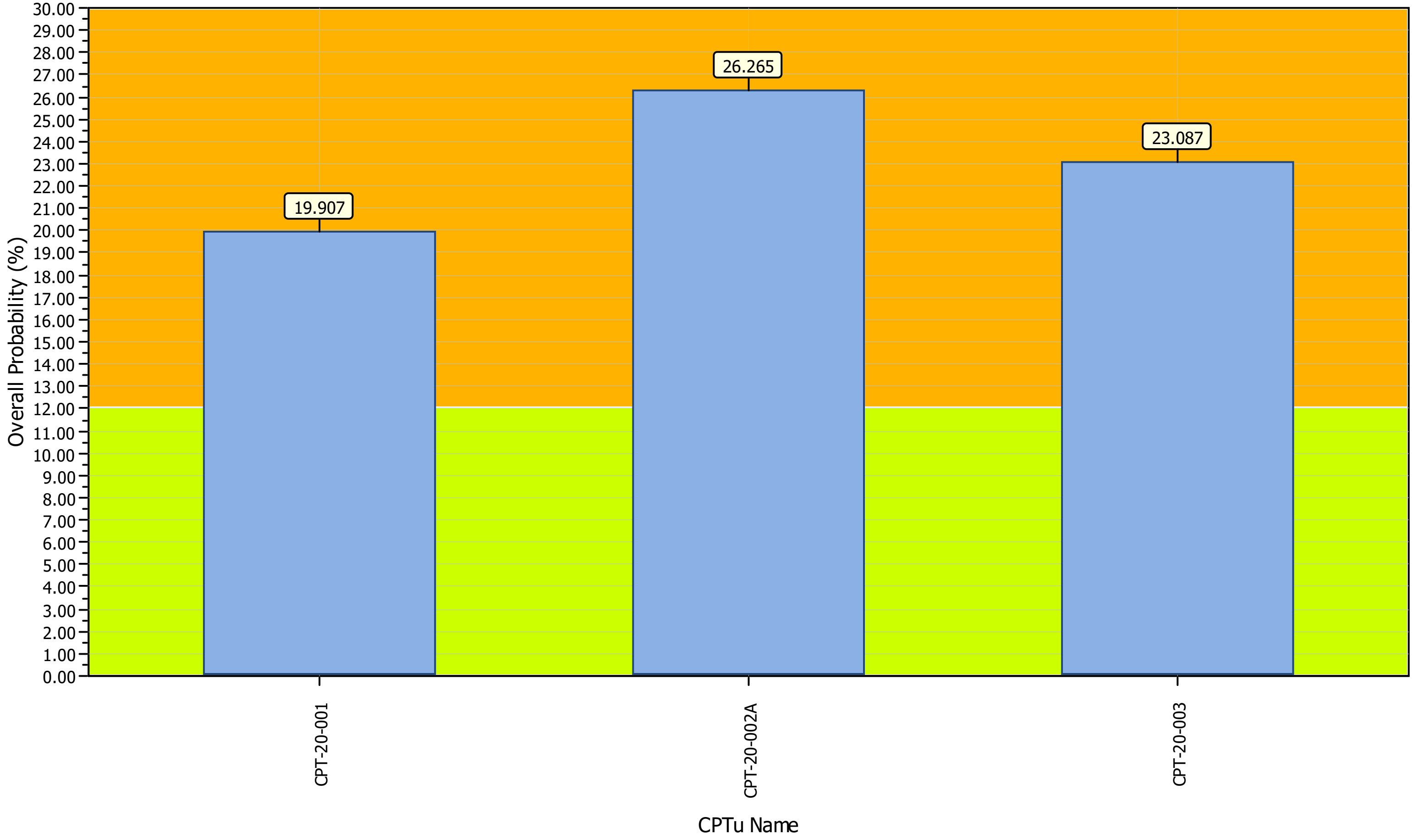
Overall vertical settlements report



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

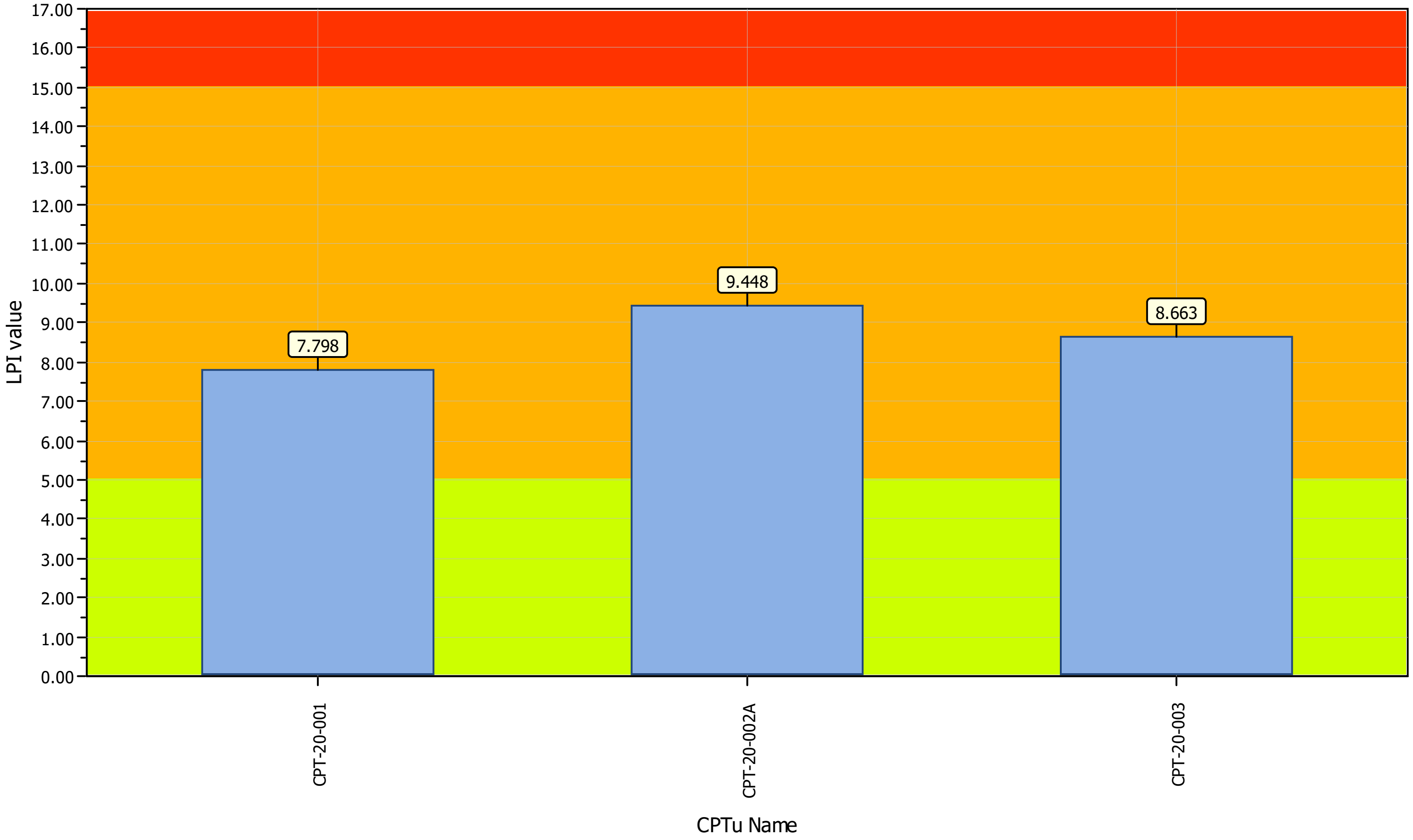
100% high probability

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

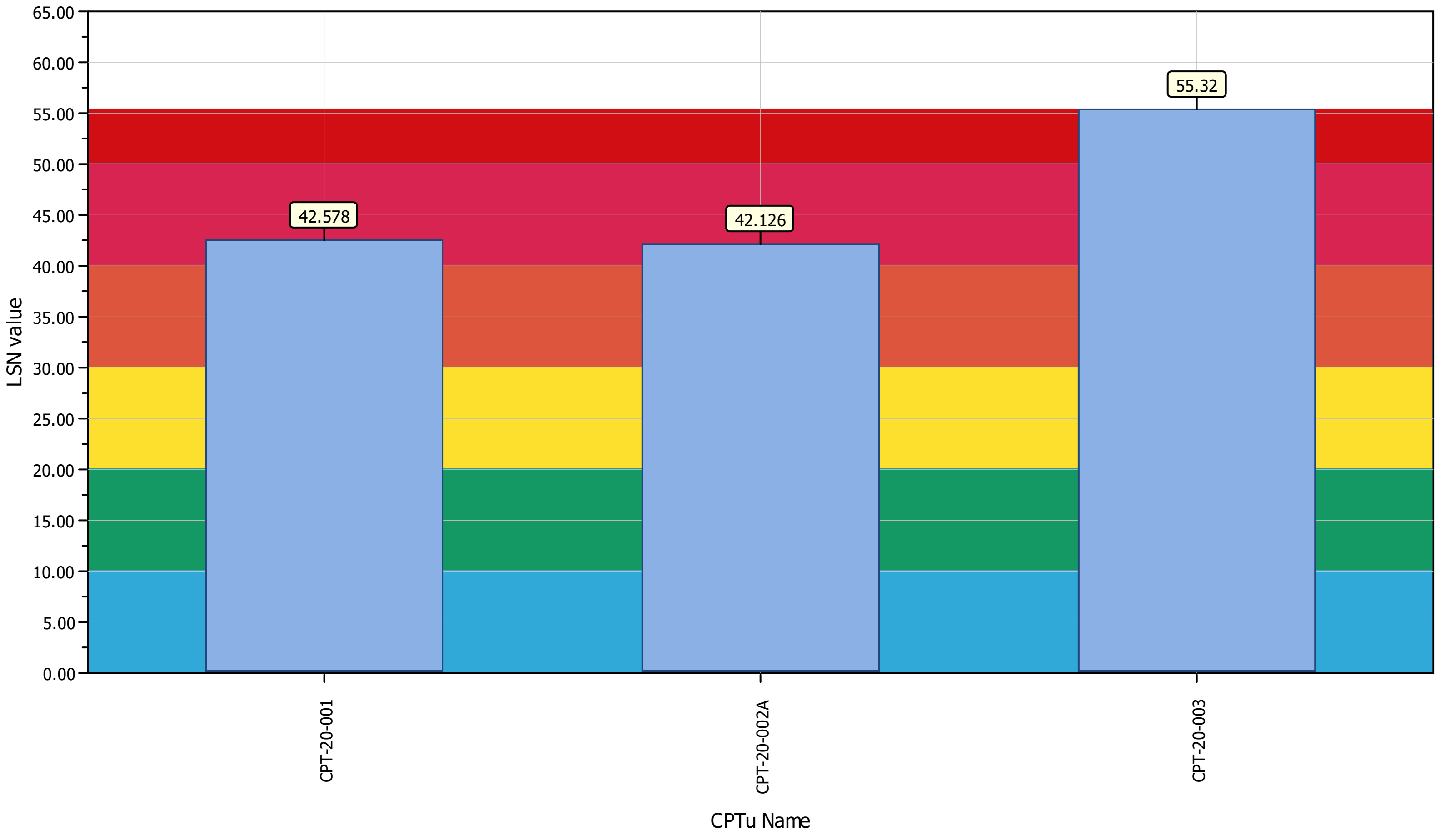
100% high risk

0% very high risk

Project title : Camino Del Mar Bridge Replacement

Location : Del Mar, CA

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

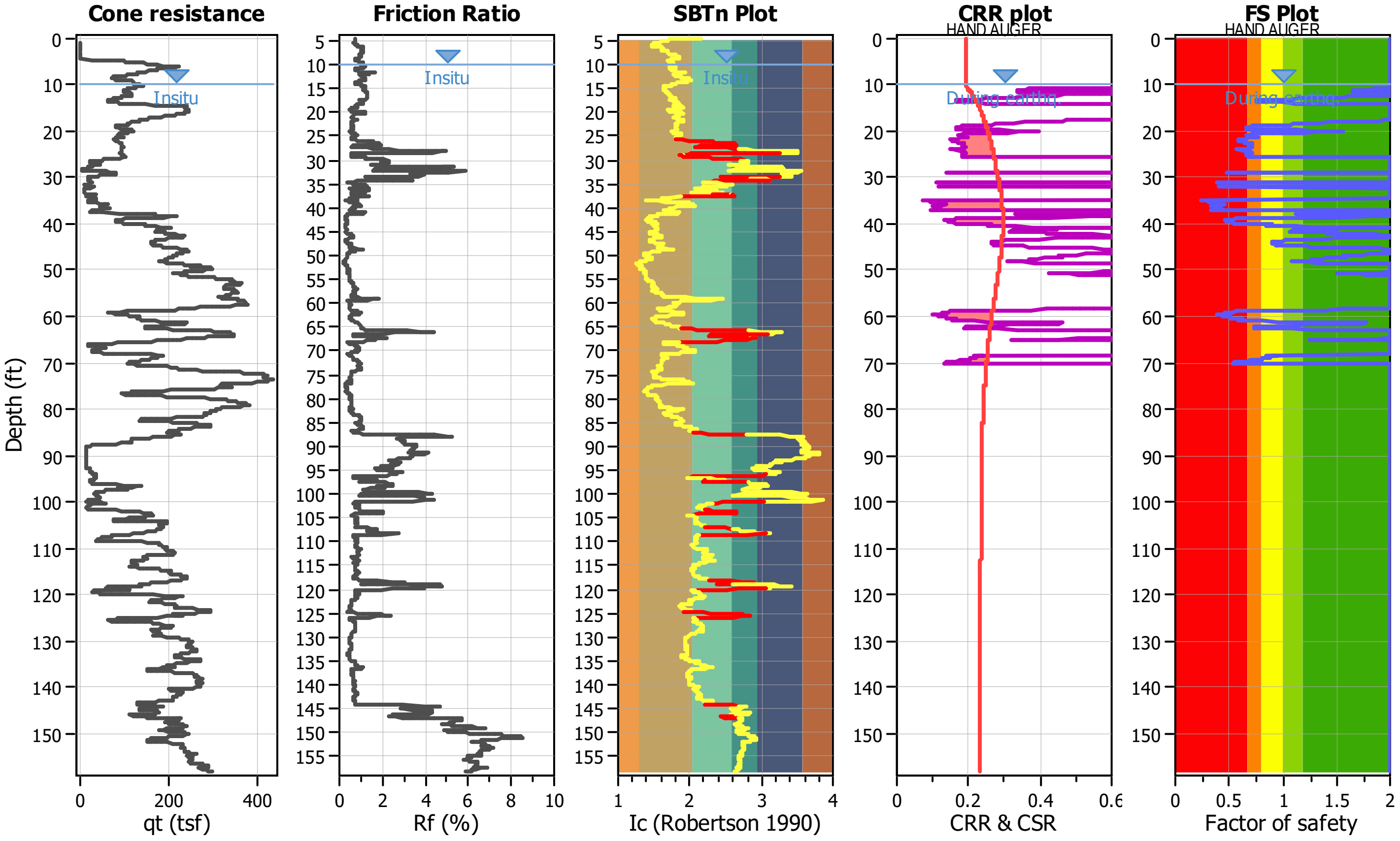
LIQUEFACTION ANALYSIS REPORT

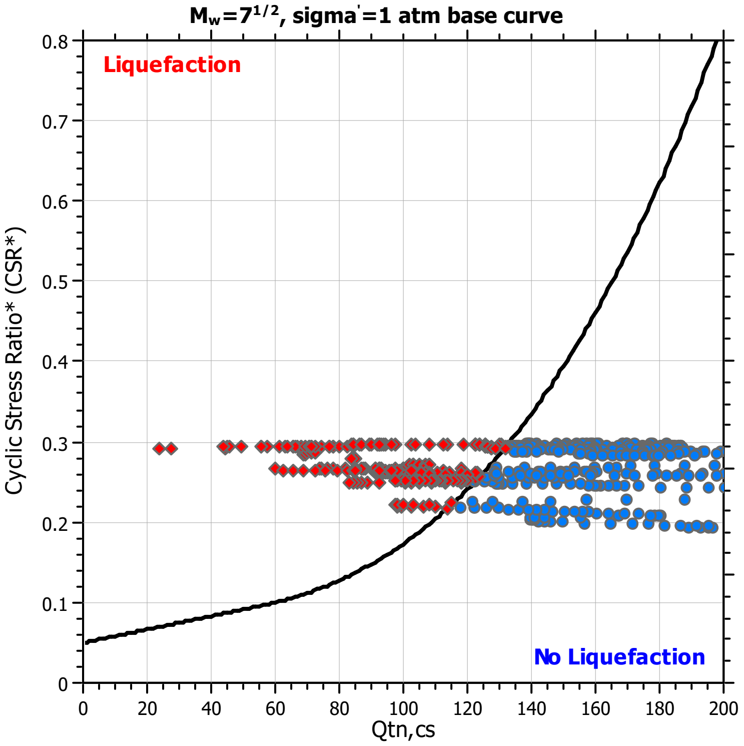
Project title : Camino Del Mar Bridge Replacement **Location : Del Mar, CA**

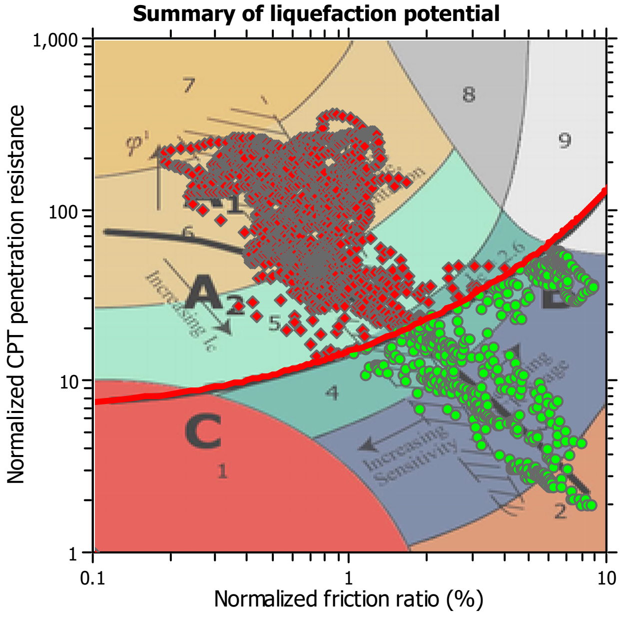
CPT file : CPT-20-001

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		

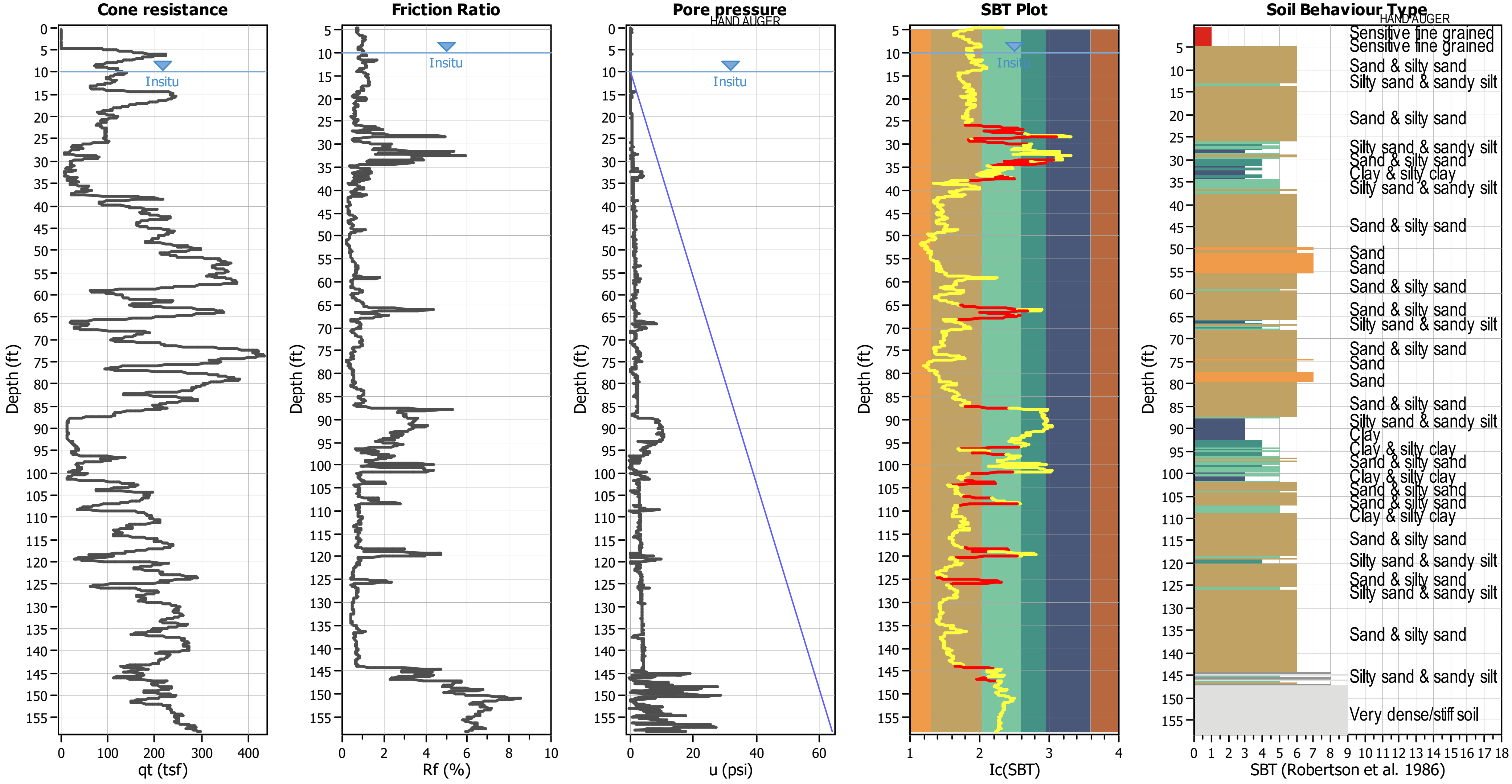






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



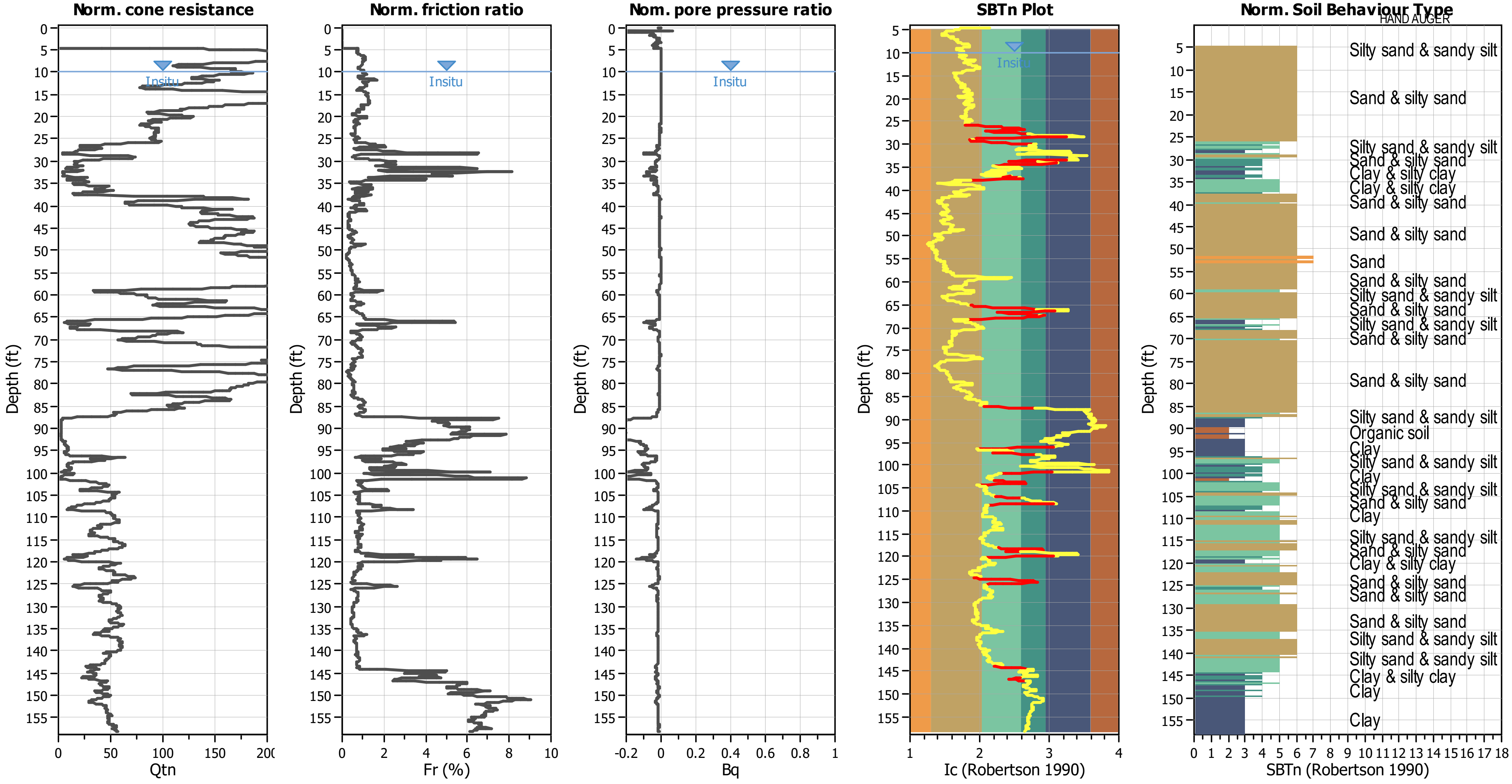
Input parameters and analysis data

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)



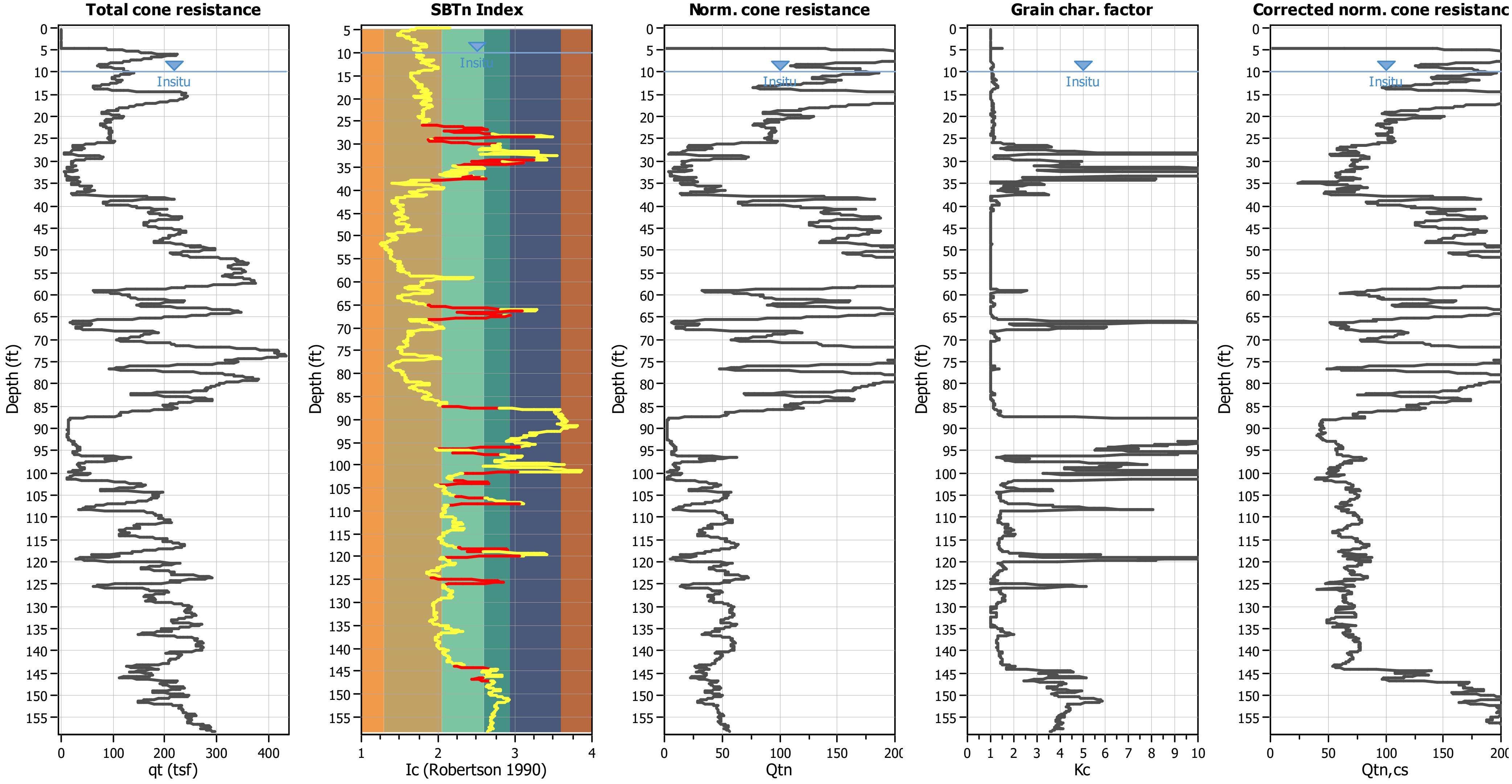
Input parameters and analysis data

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
- 🟫 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

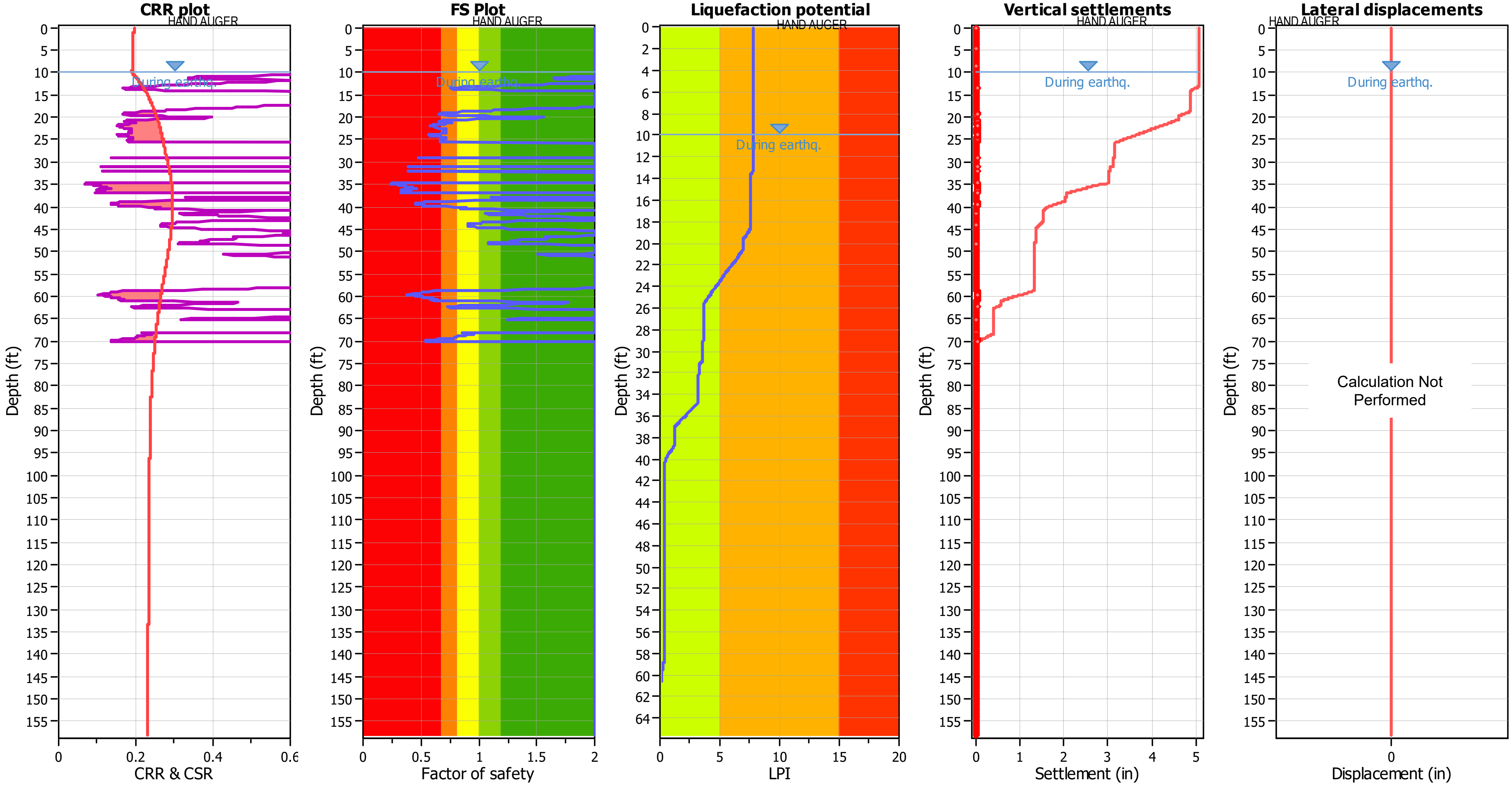
Liquefaction analysis overall plots (intermediate results)



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

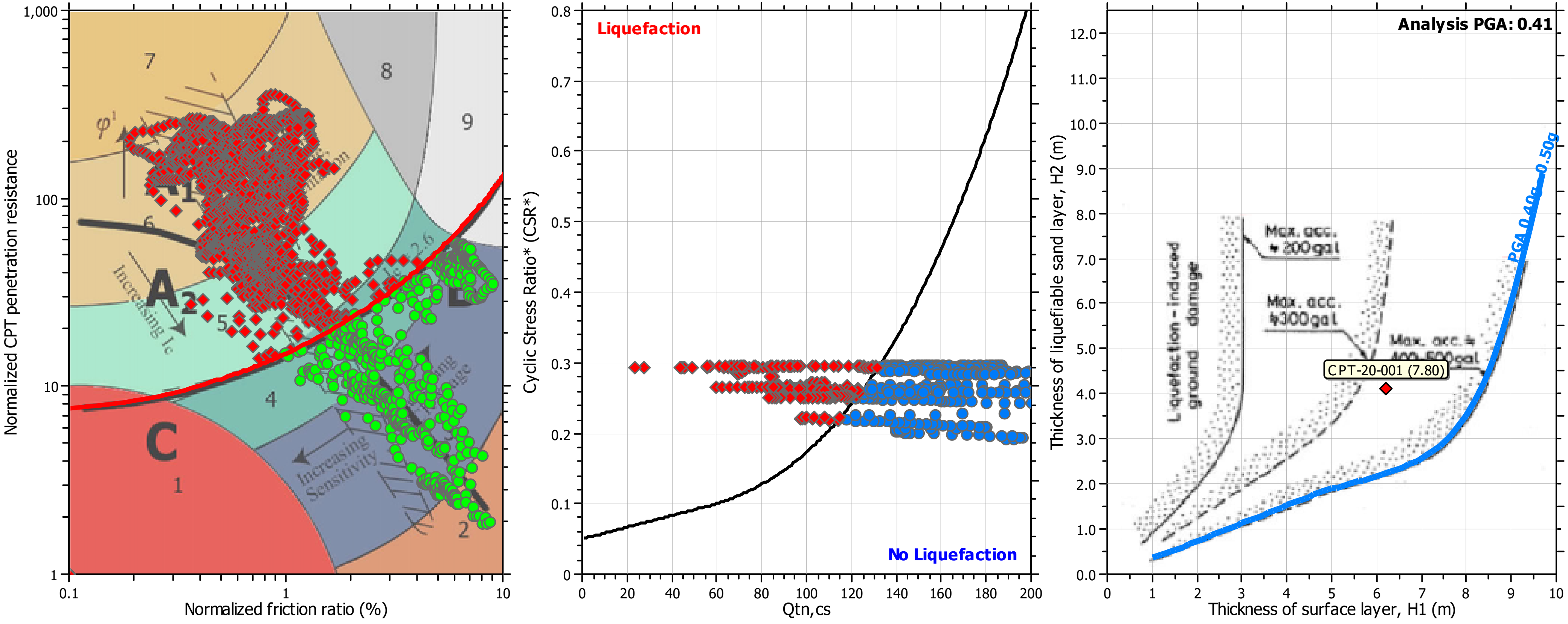
F.S. color scheme

- 🟥 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

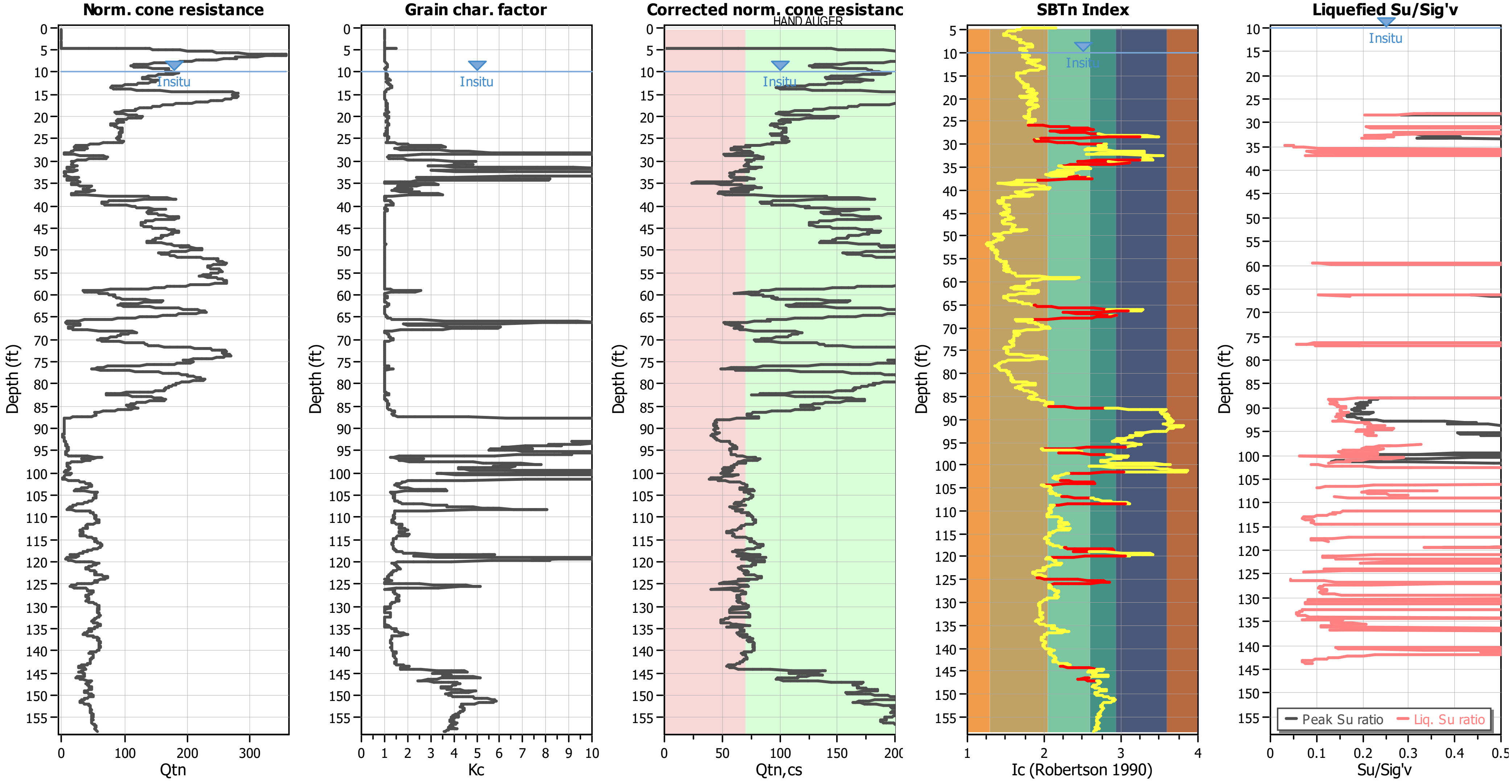
Liquefaction analysis summary plots



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

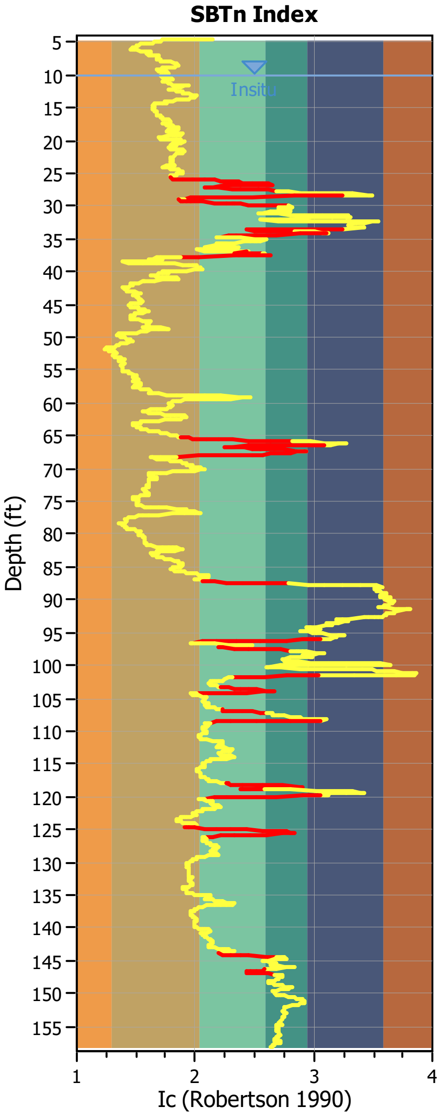
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight 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

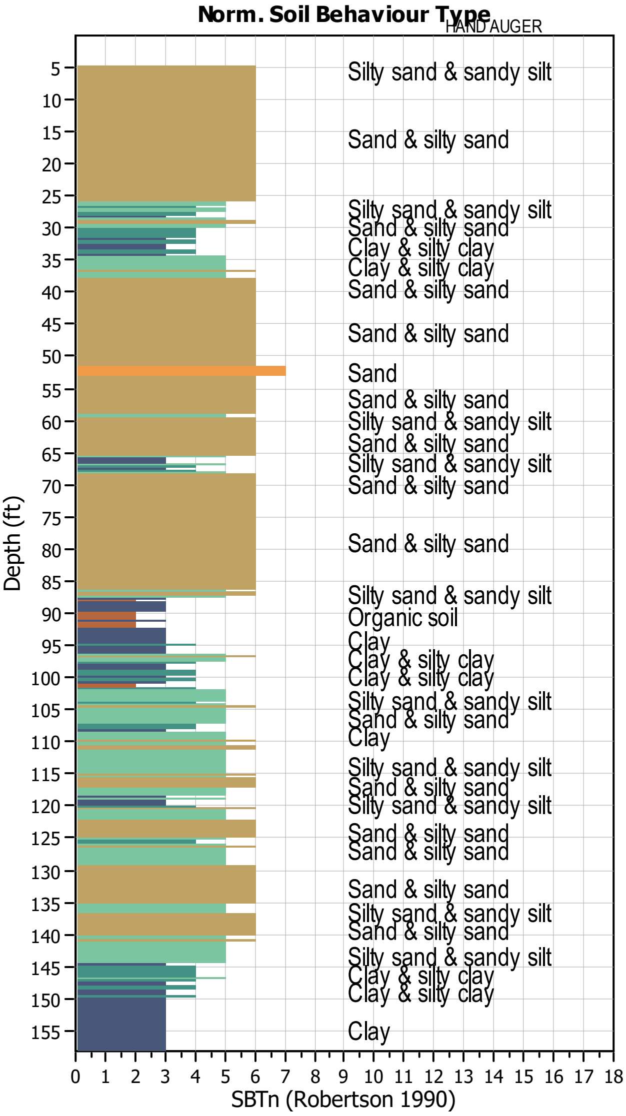
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

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).

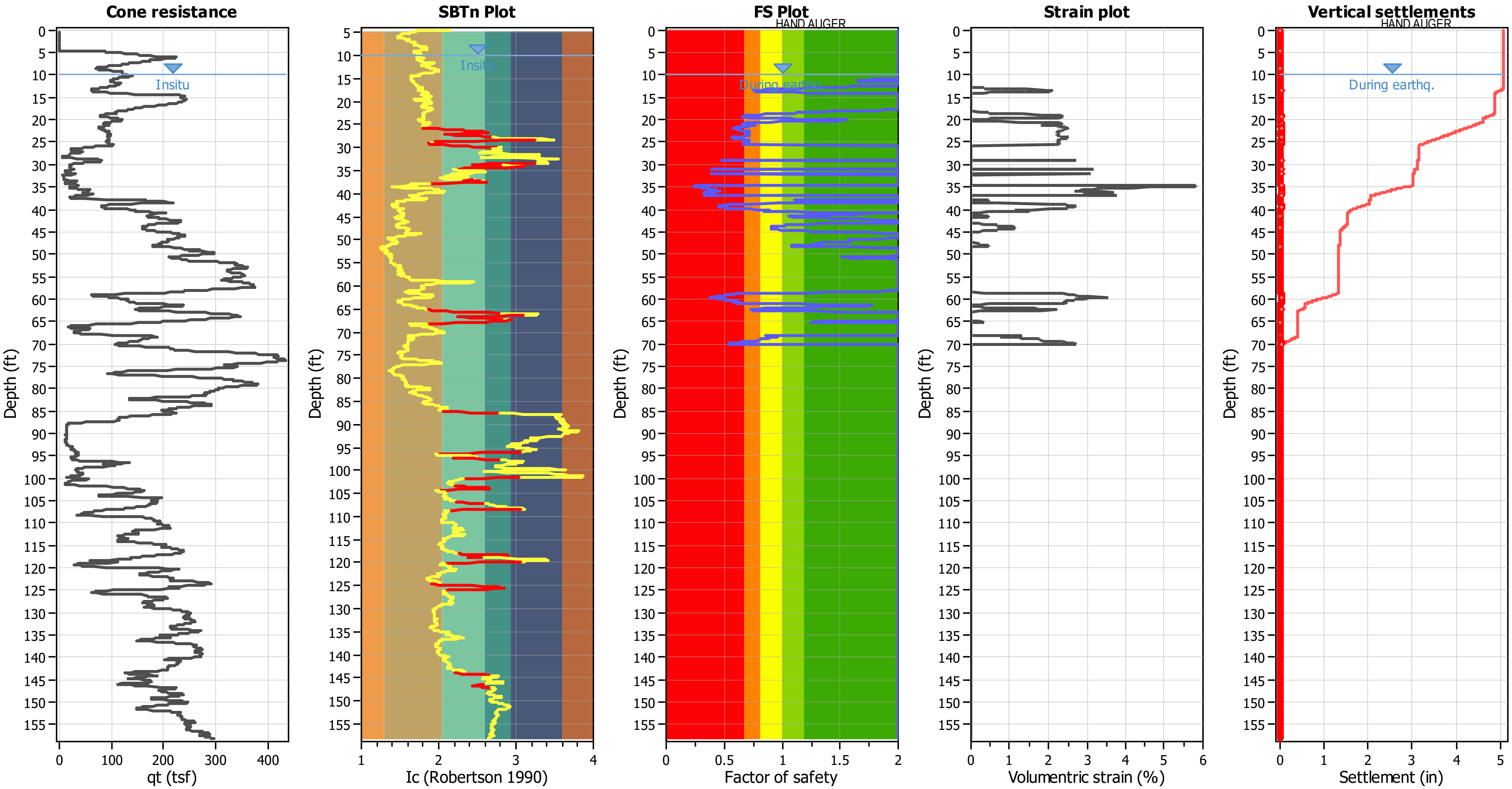
The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





Transition layer algorithm properties		**General statistics**	
I_c minimum check value:	1.70	Total points in CPT file:	2411
I_c maximum check value:	3.00	Total points excluded:	254
I_c change ratio value:	0.0250	Exclusion percentage:	10.54%
Minimum number of points in layer:	4	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

GEOLOGISMIKI
Geotechnical Software

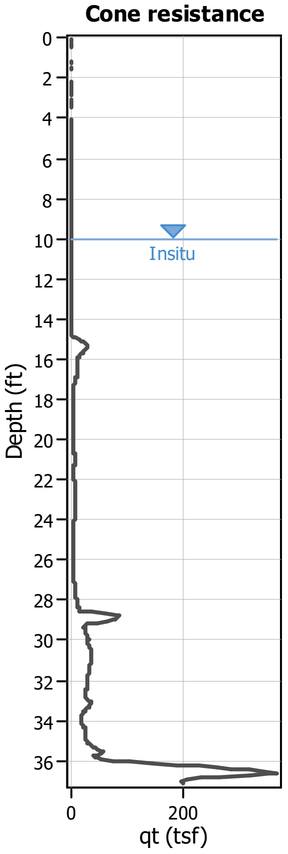
LIQUEFACTION ANALYSIS REPORT

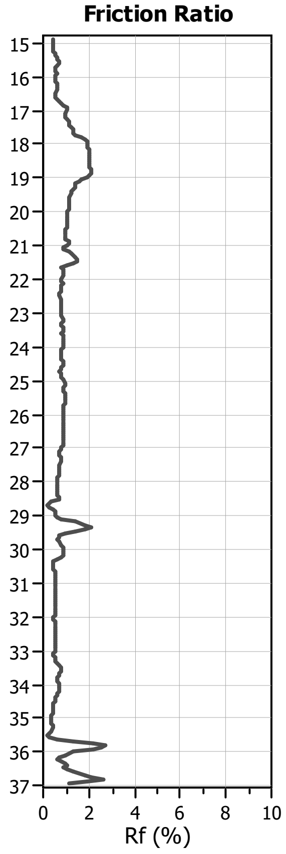
Project title : Camino Del Mar Bridge Replacement **Location : Del Mar, CA**

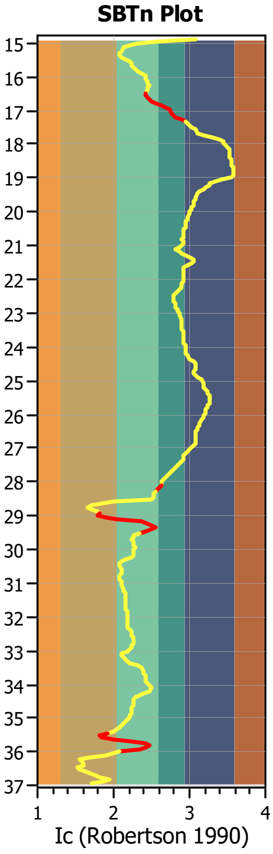
CPT file : CPT-20-002A

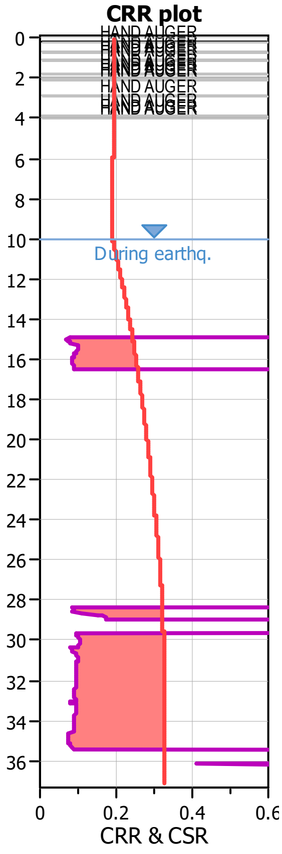
Input parameters and analysis data

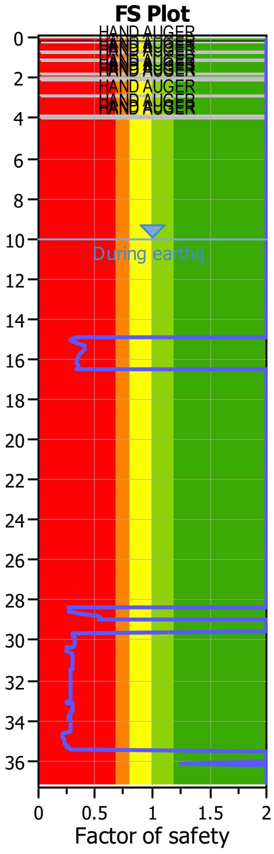
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit 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		

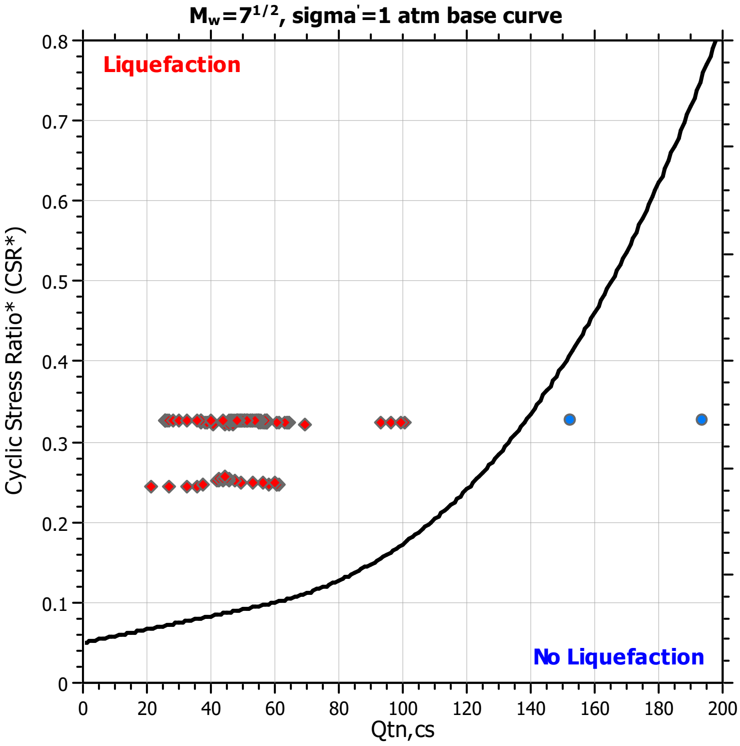


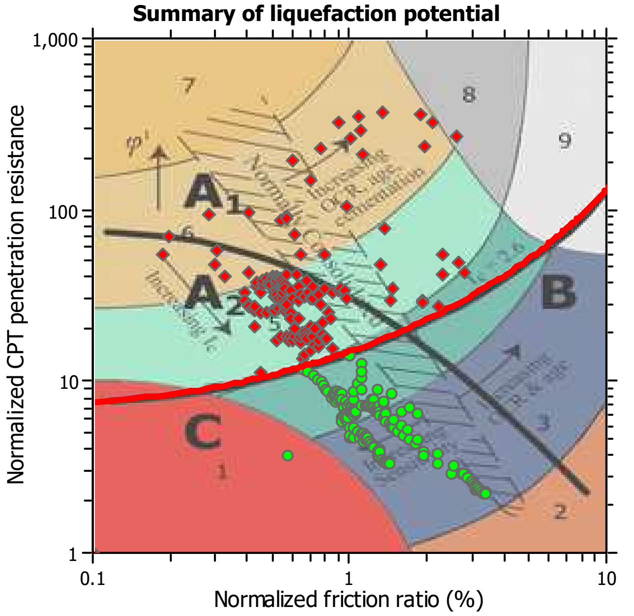






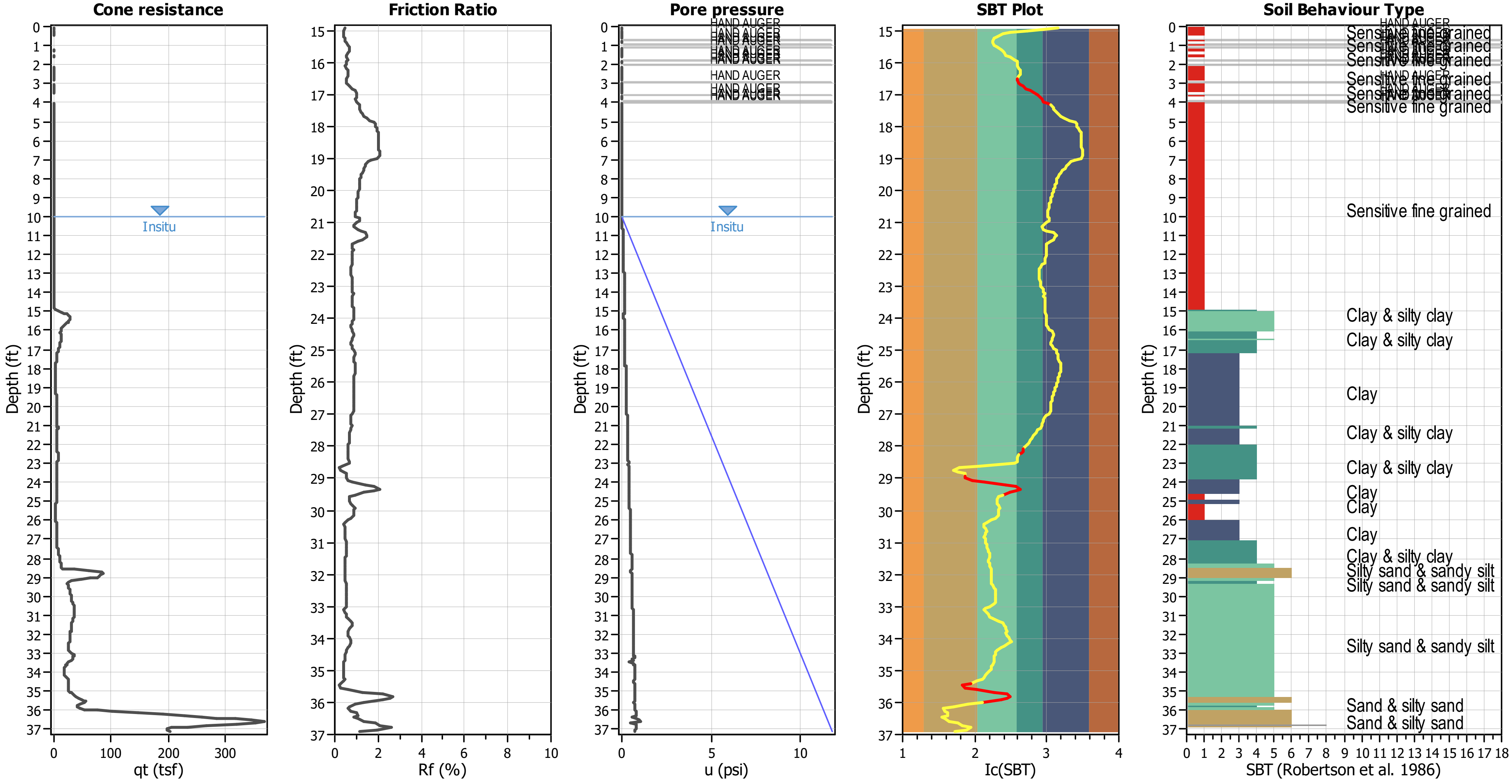






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



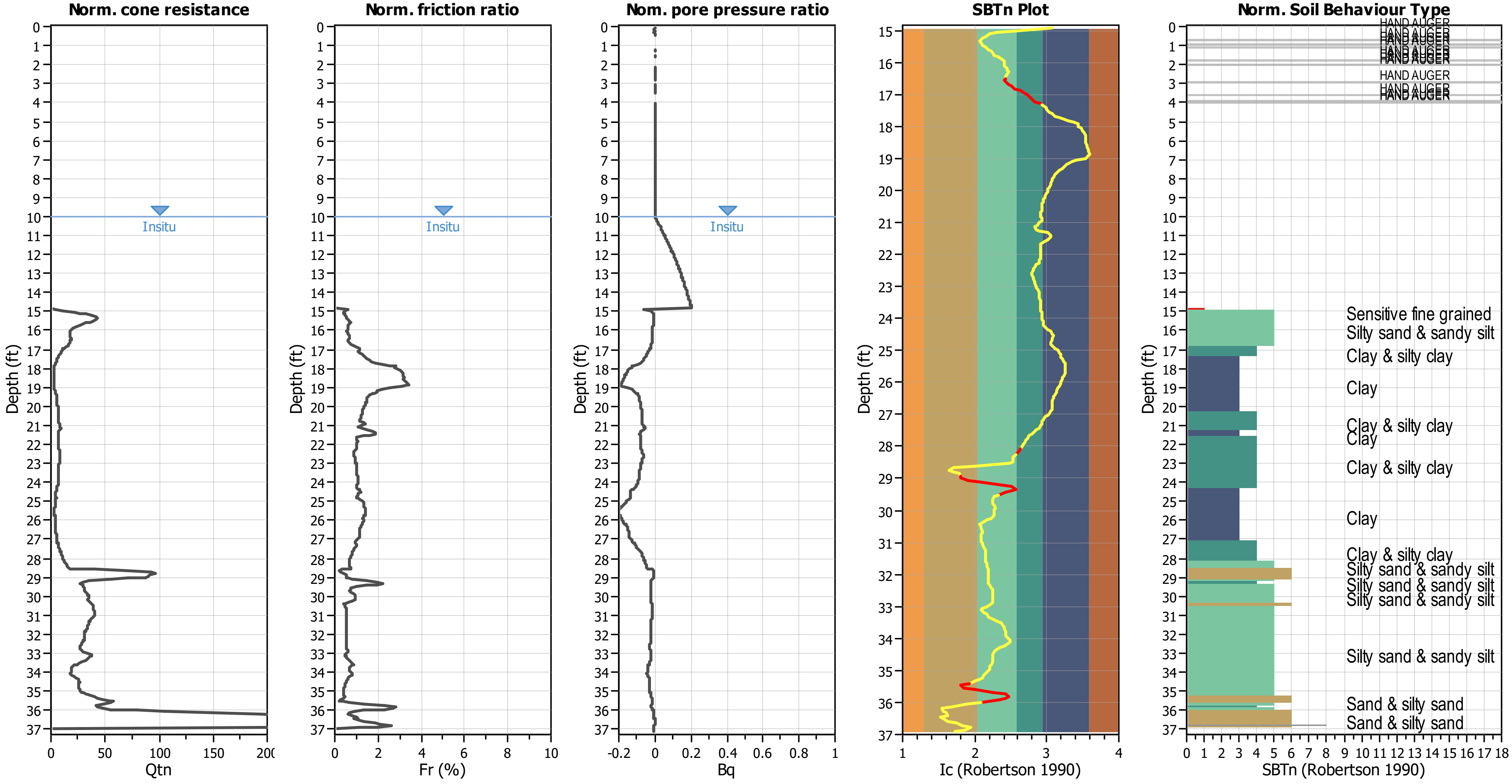
Input parameters and analysis data

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)



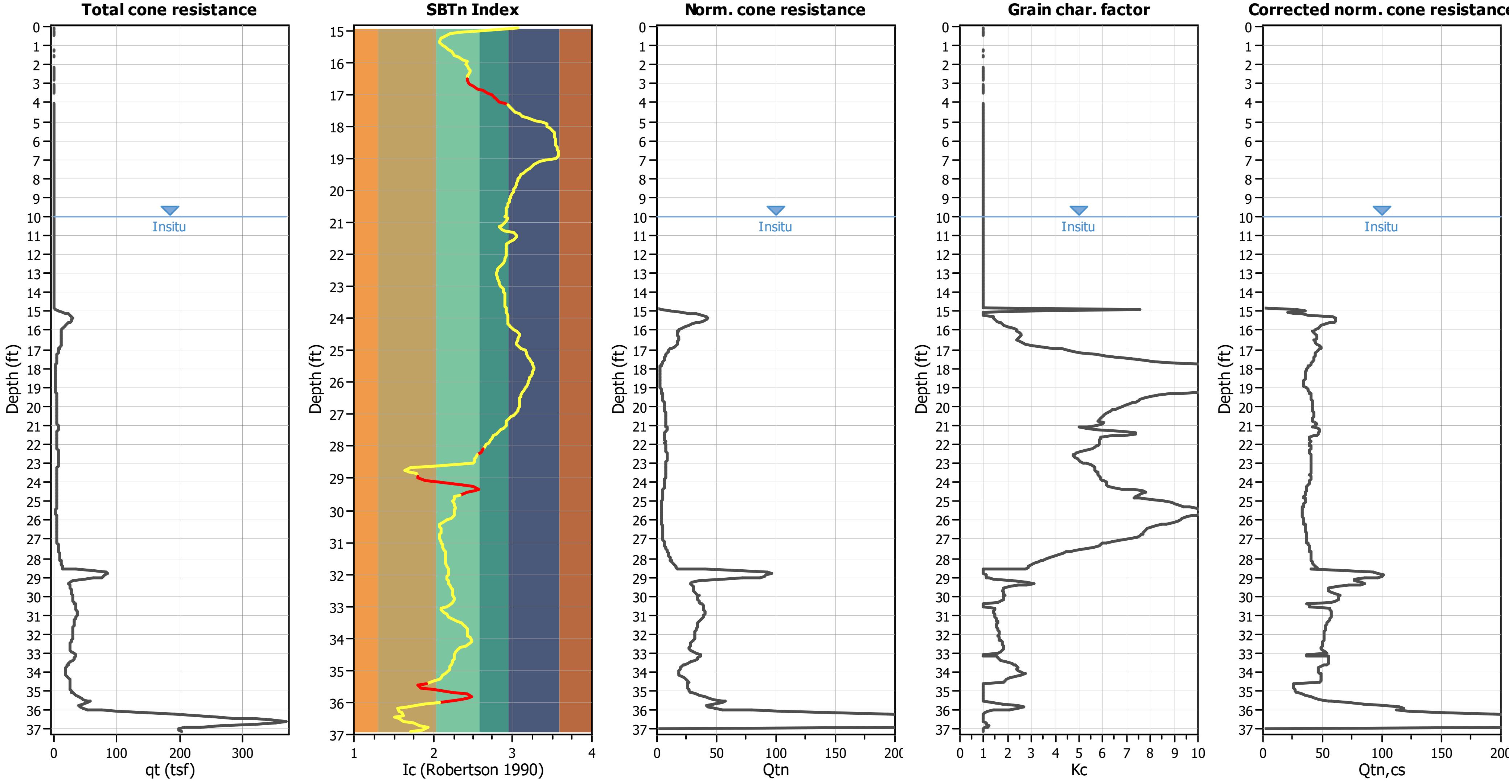
Input parameters and analysis data

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
- 🟫 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

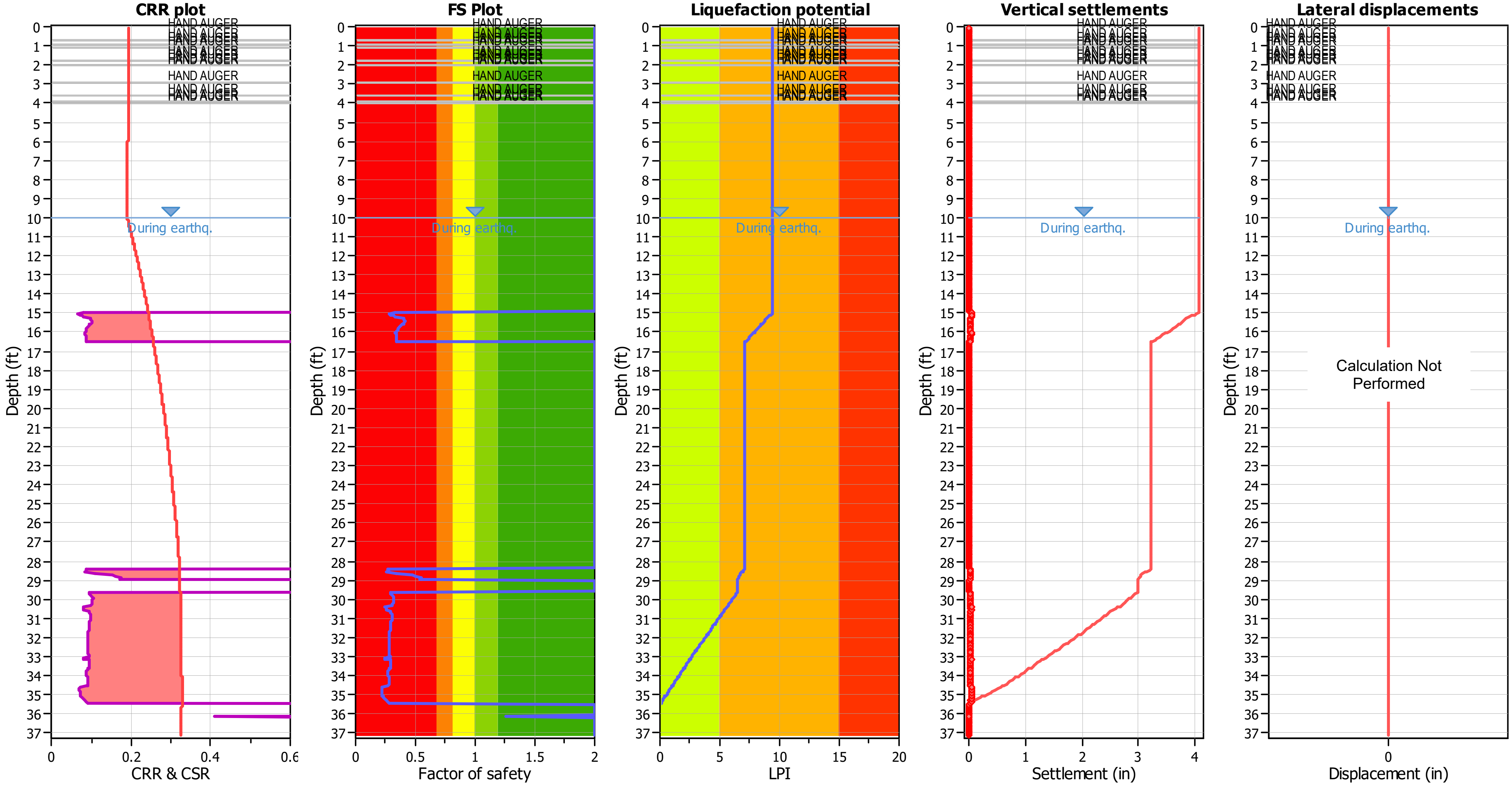
Liquefaction analysis overall plots (intermediate results)



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

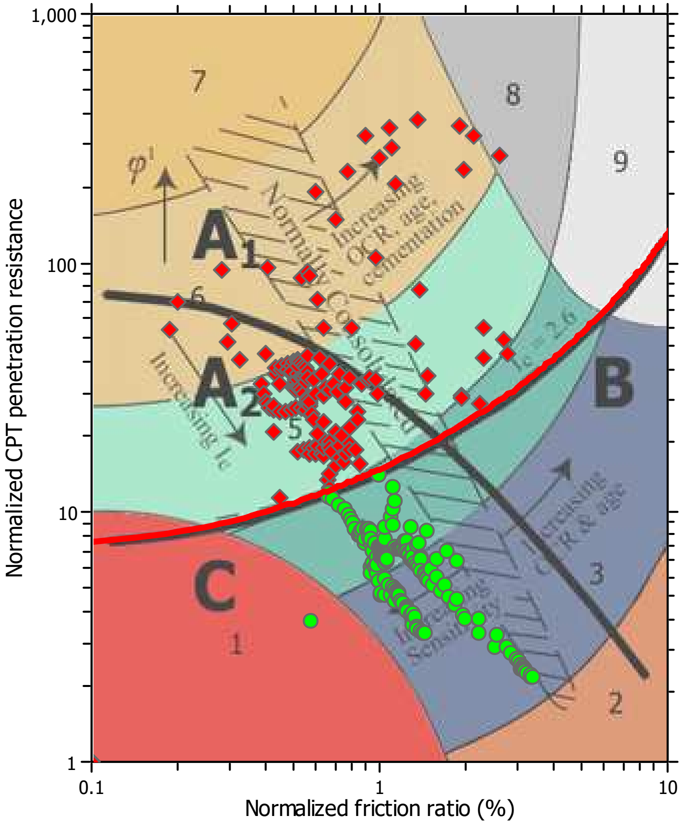
F.S. color scheme

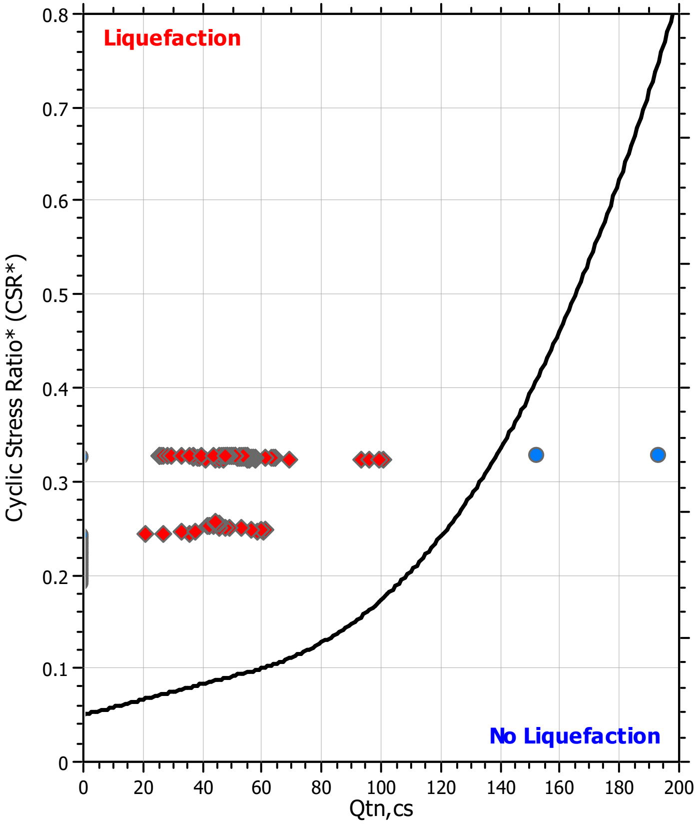
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

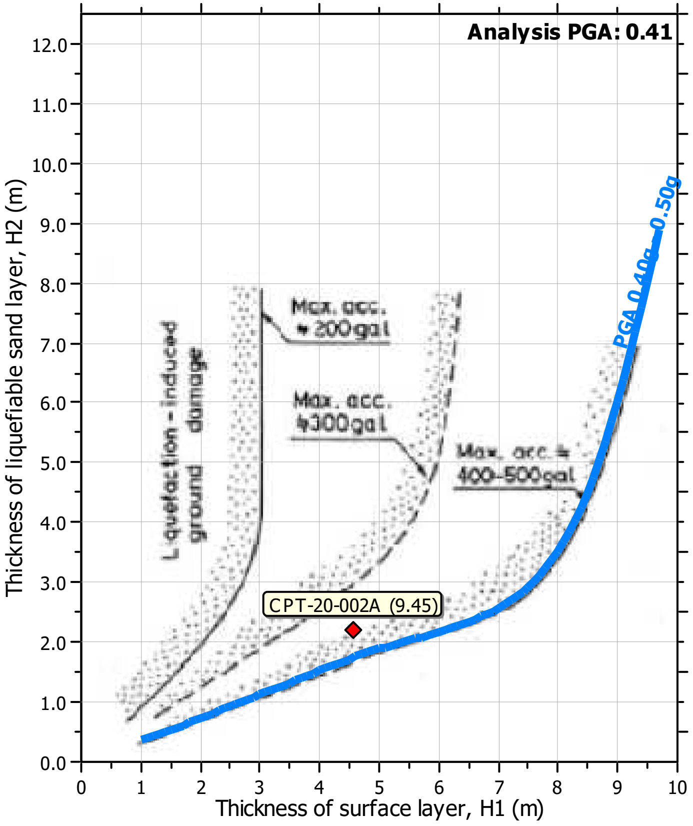
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



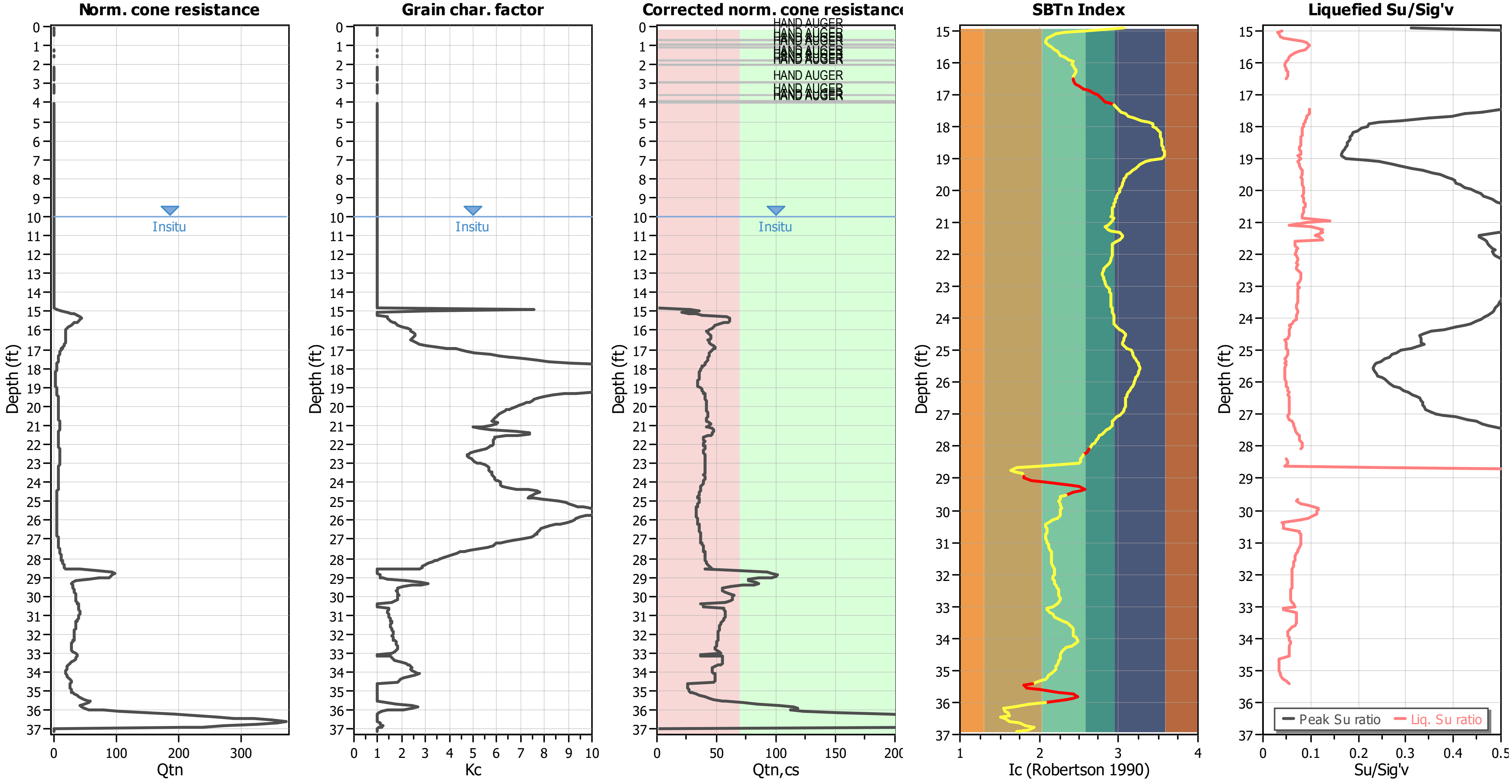




Input parameters and analysis data

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

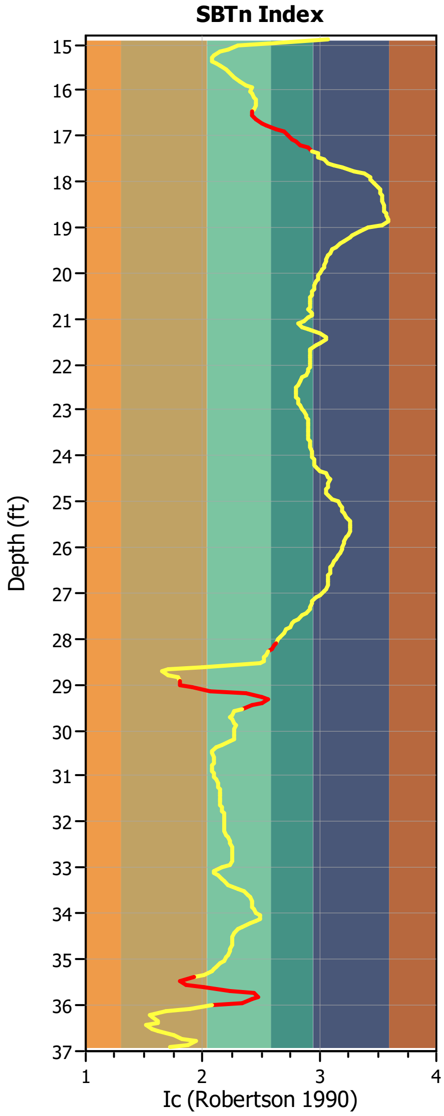
Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: Yes
Limit depth: 70.00 ft

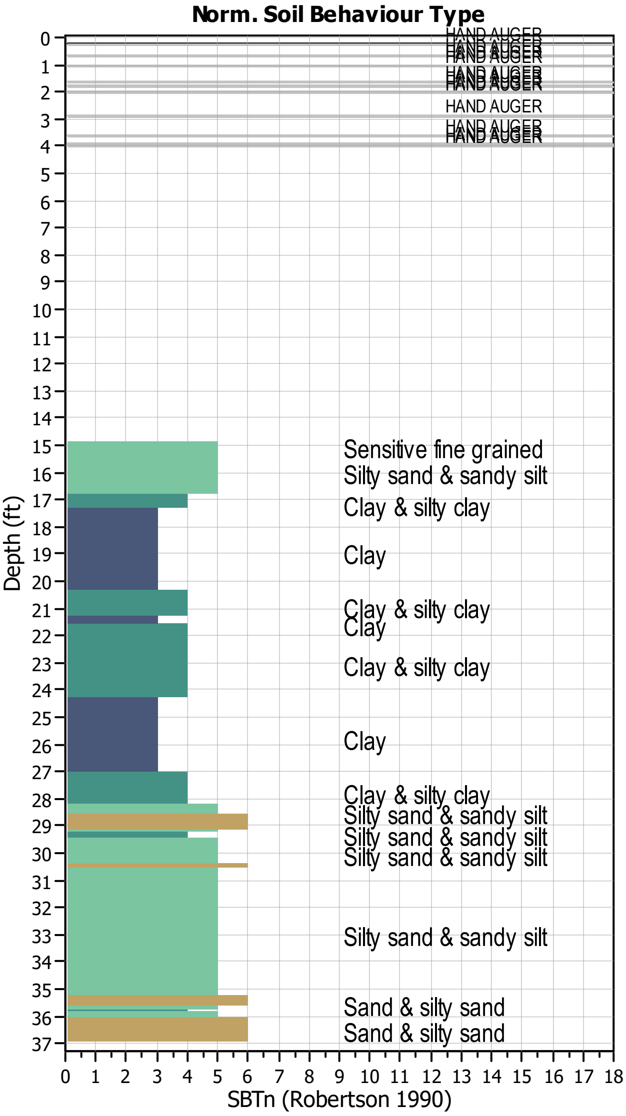
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

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.





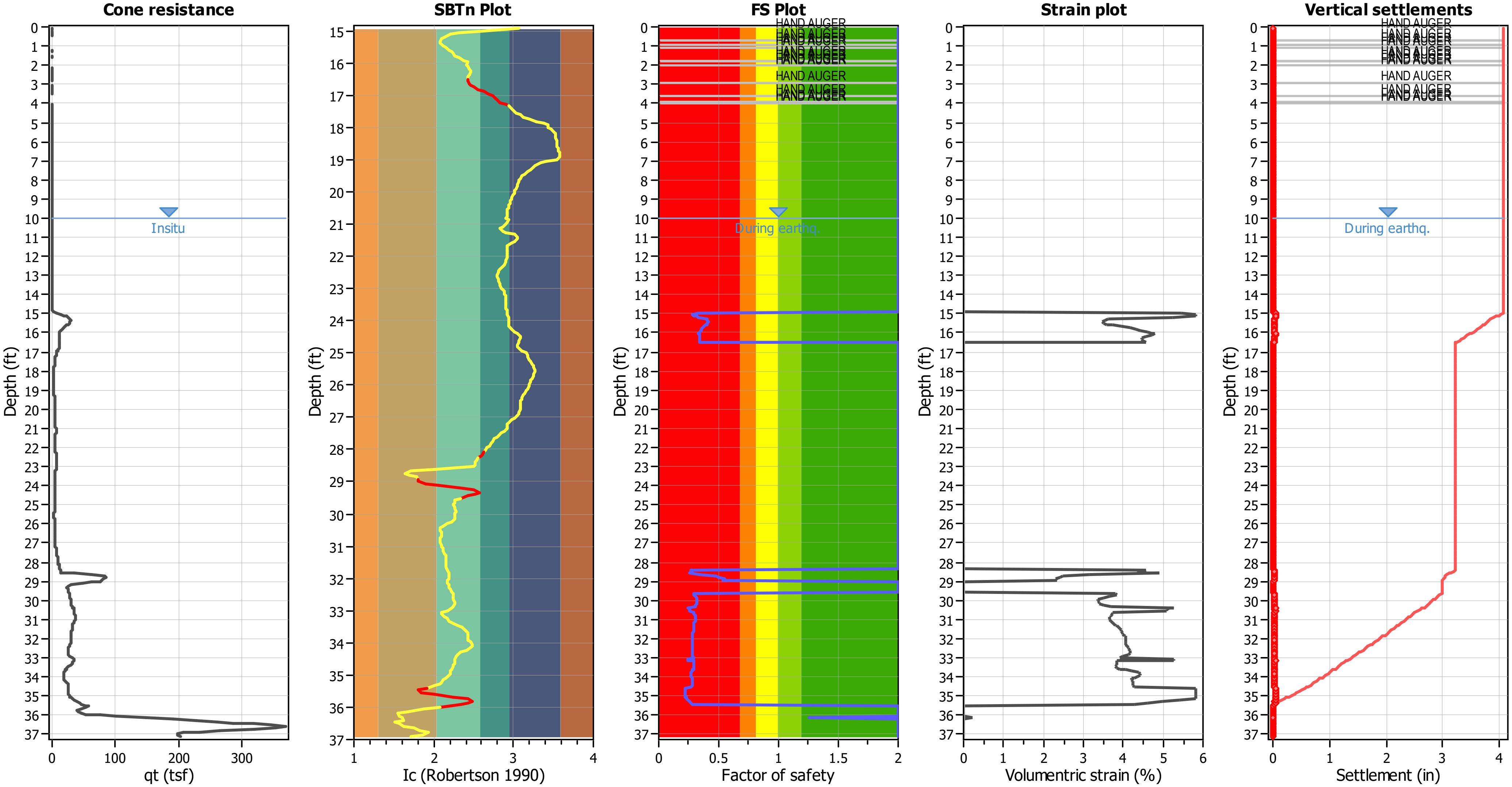
Transition layer algorithm properties

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:	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

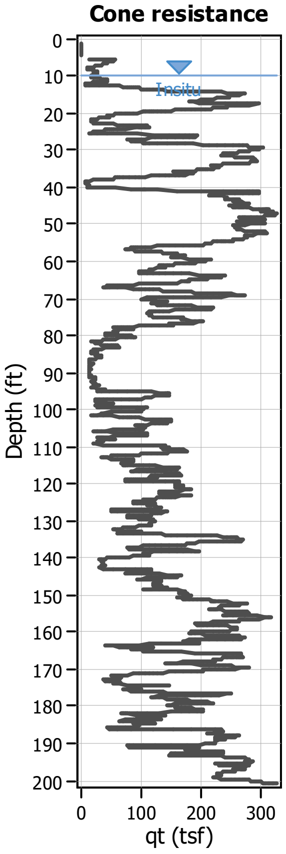
LIQUEFACTION ANALYSIS REPORT

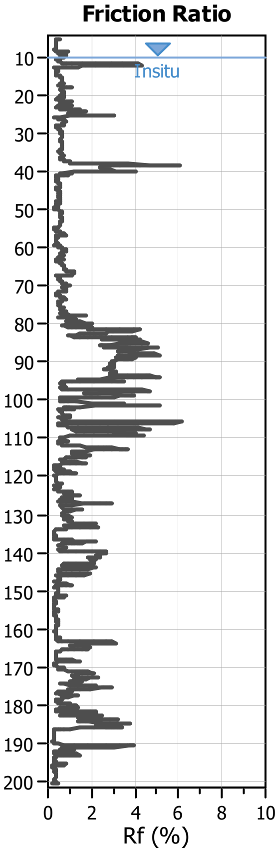
Project title : Camino Del Mar Bridge Replacement **Location : Del Mar, CA**

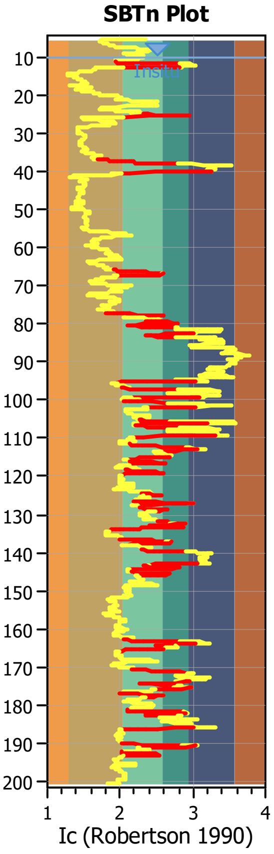
CPT file : CPT-20-003

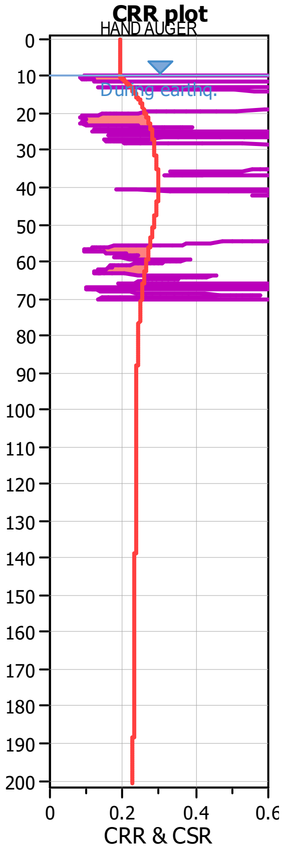
Input parameters and analysis data

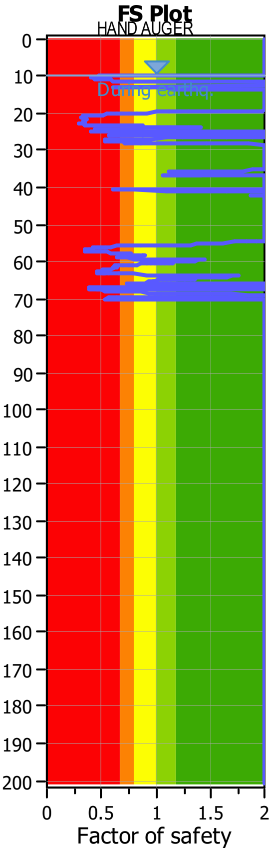
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	6.63	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	70.00 ft
Peak ground acceleration:	0.41	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

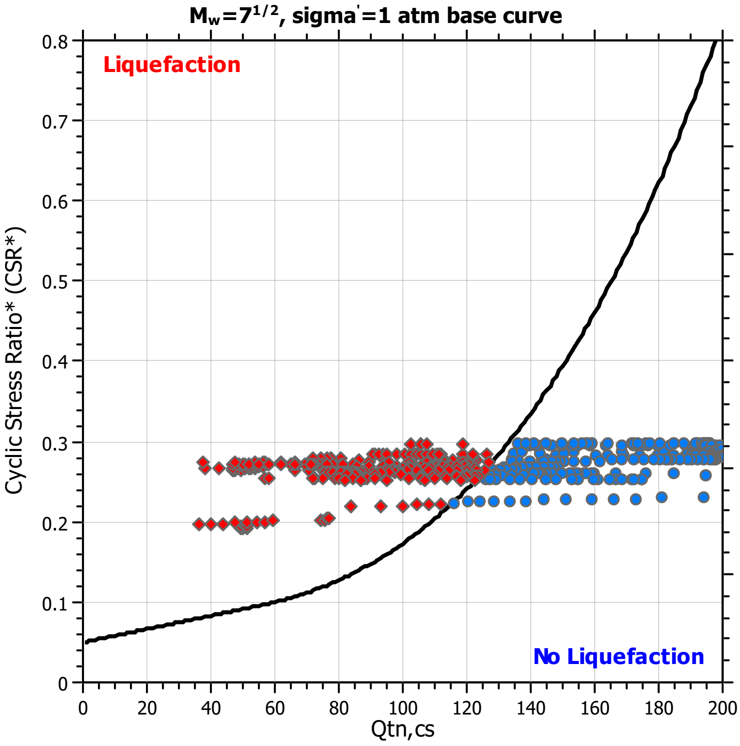

Cone resistance

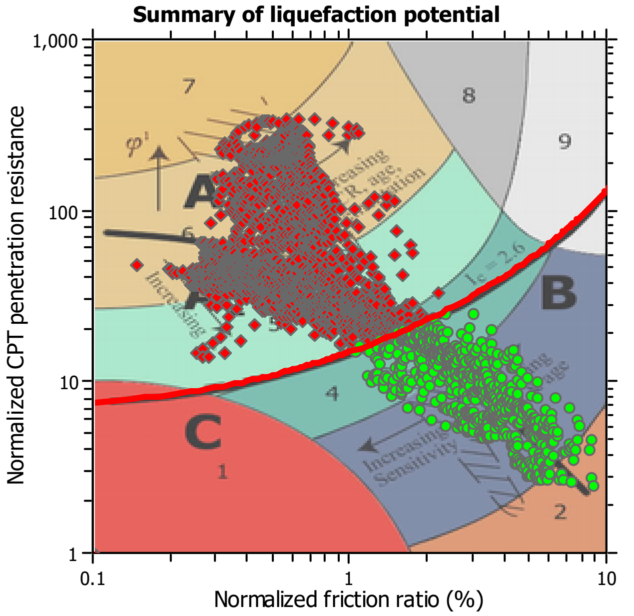

Friction Ratio


SBTn Plot


CRR plot

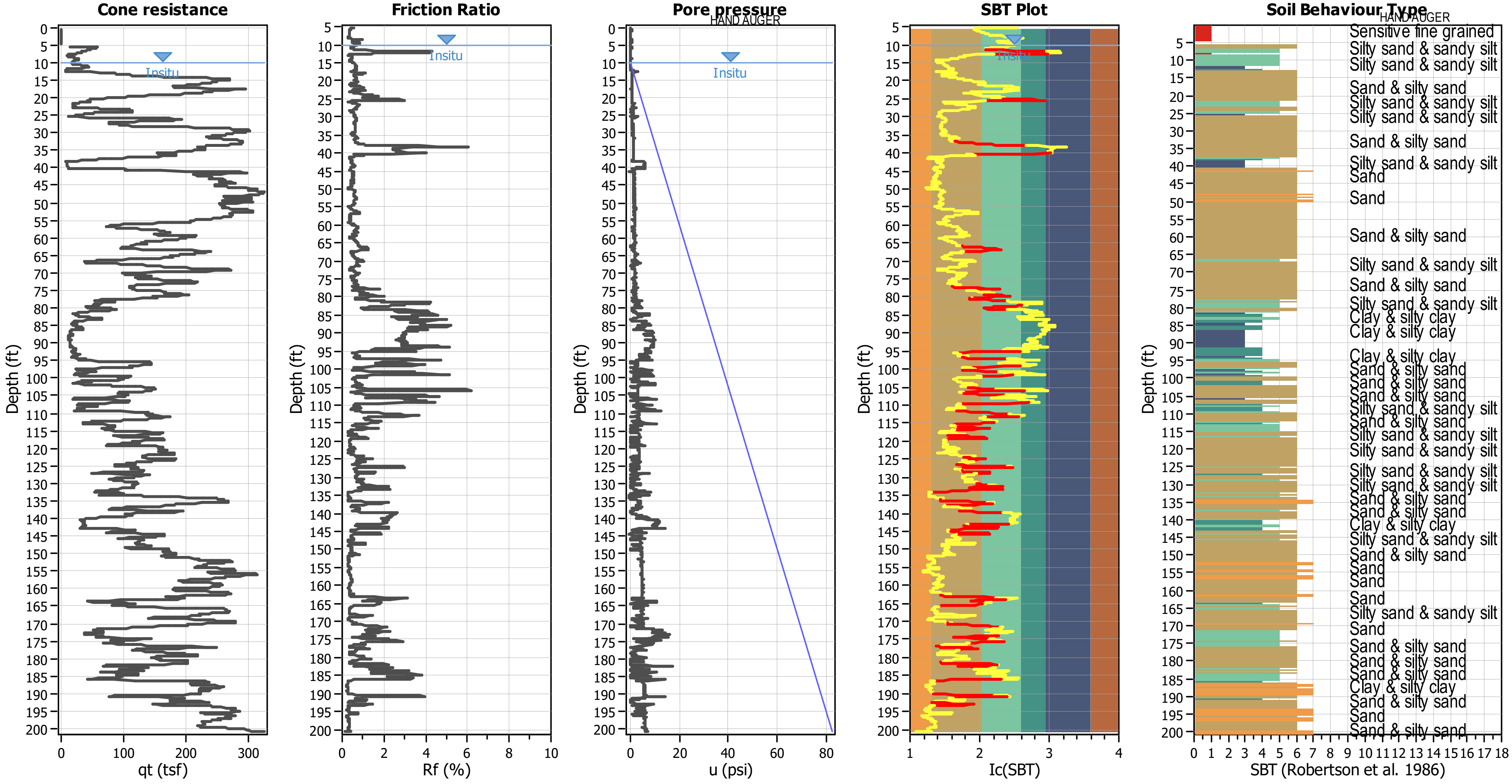

FS Plot


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


Summary of liquefaction potential

Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



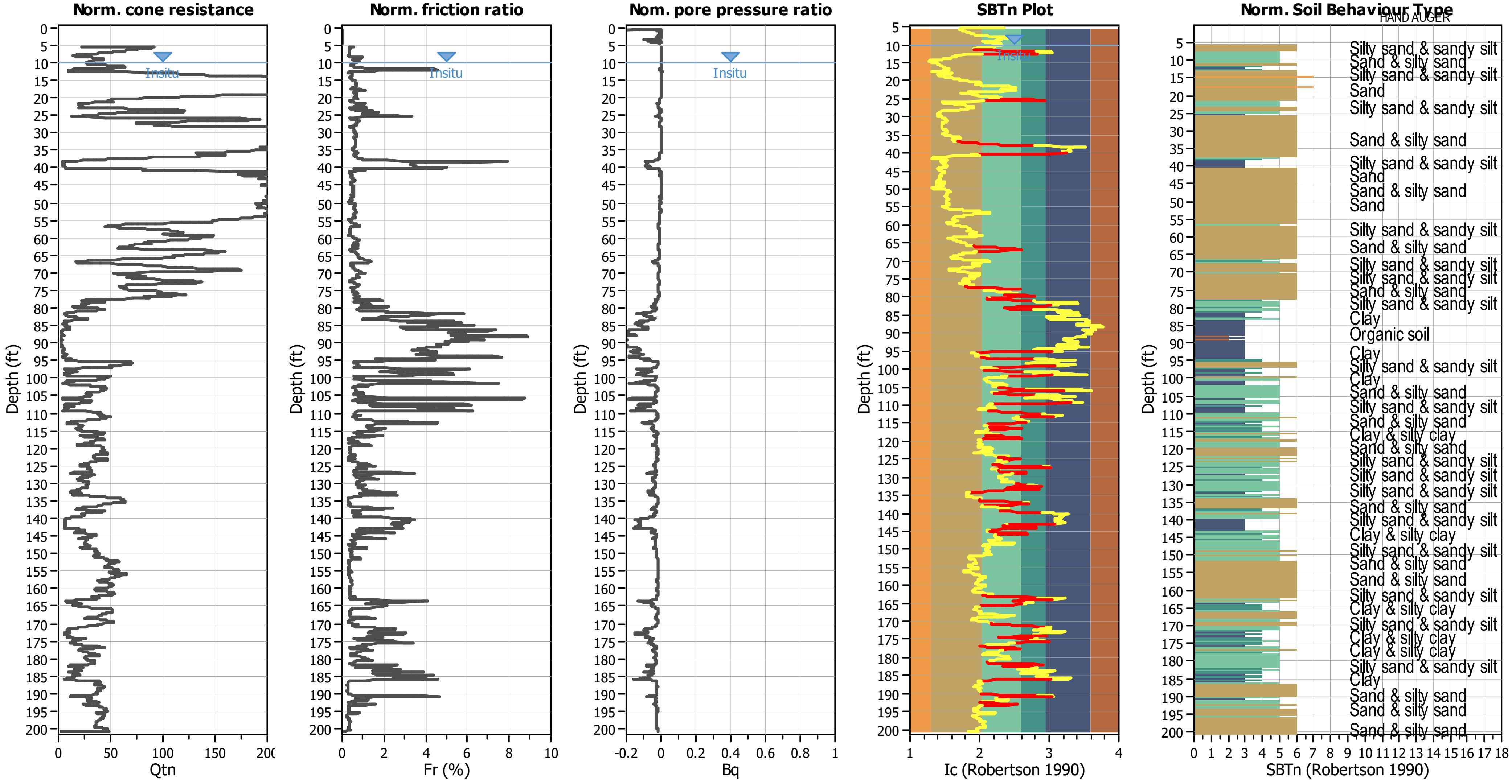
Input parameters and analysis data

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)



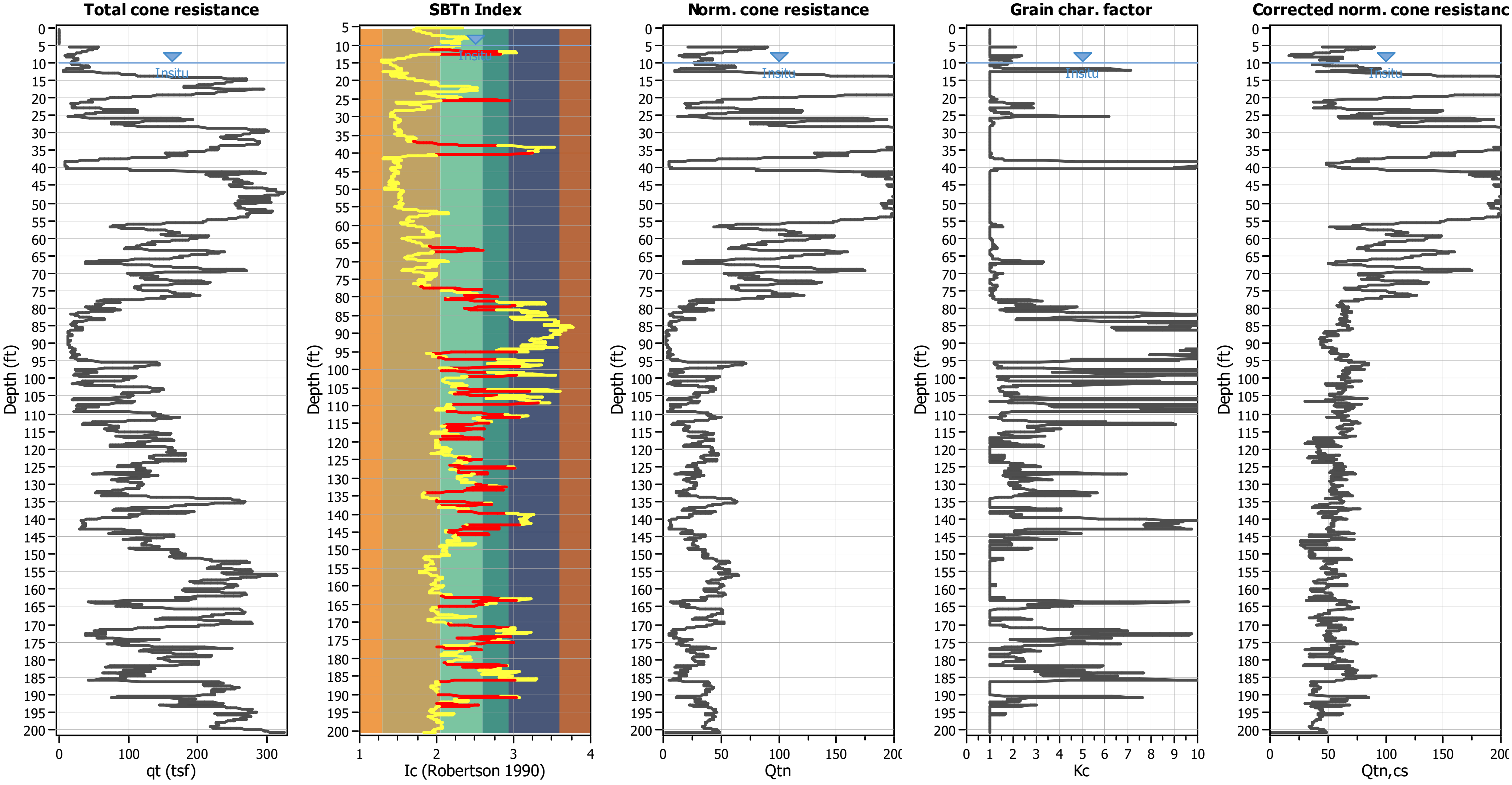
Input parameters and analysis data

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

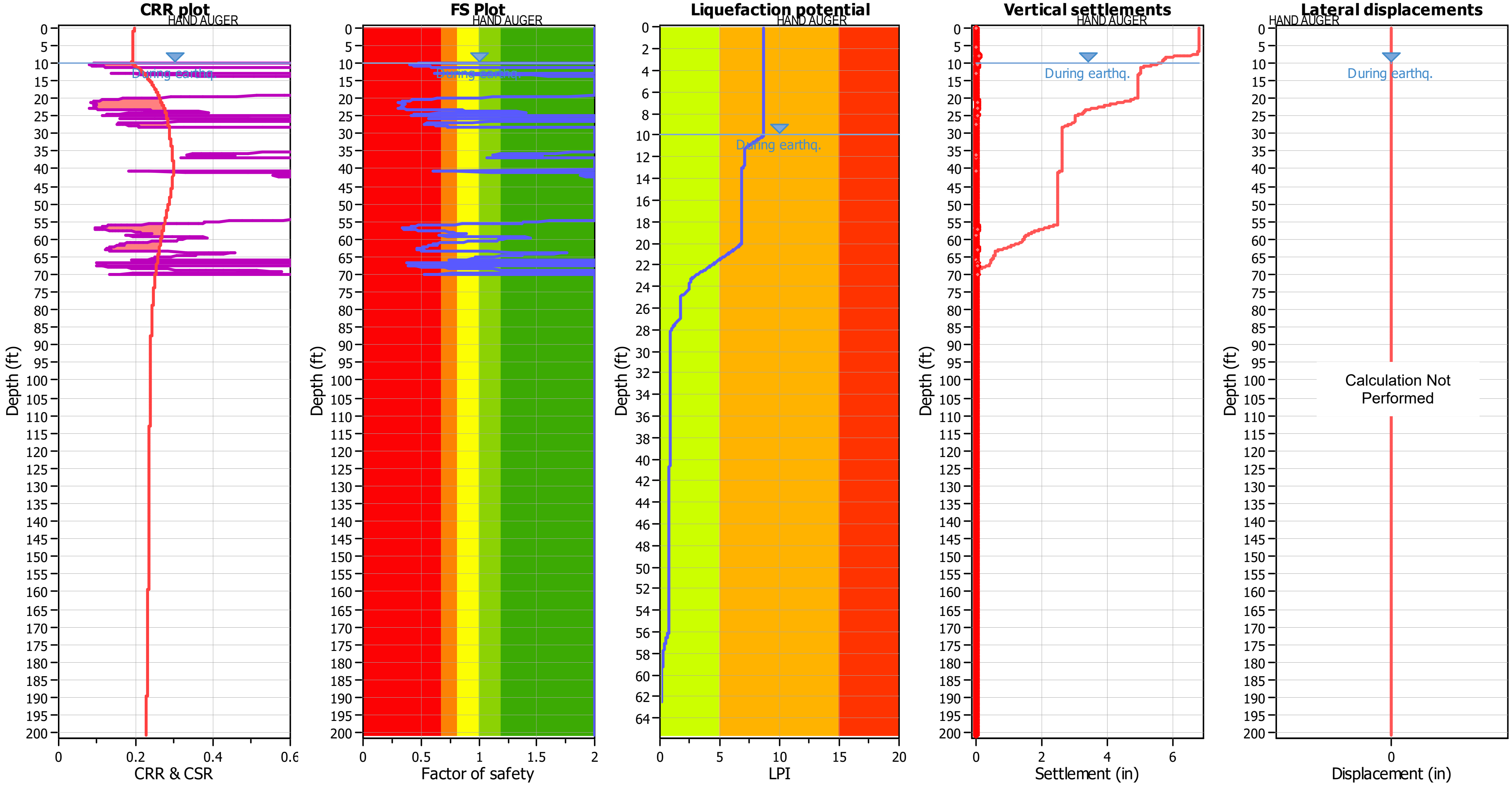
Liquefaction analysis overall plots (intermediate results)



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

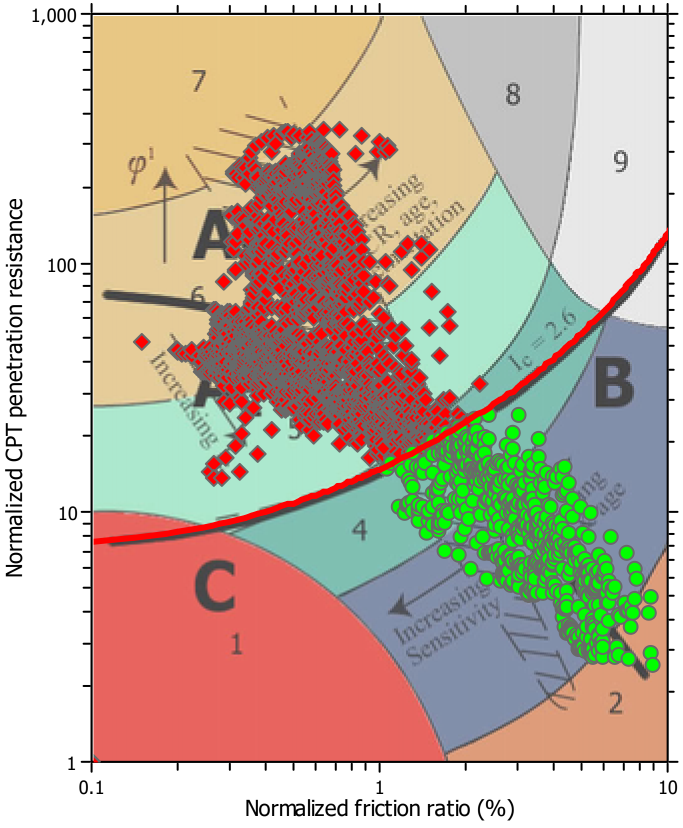
F.S. color scheme

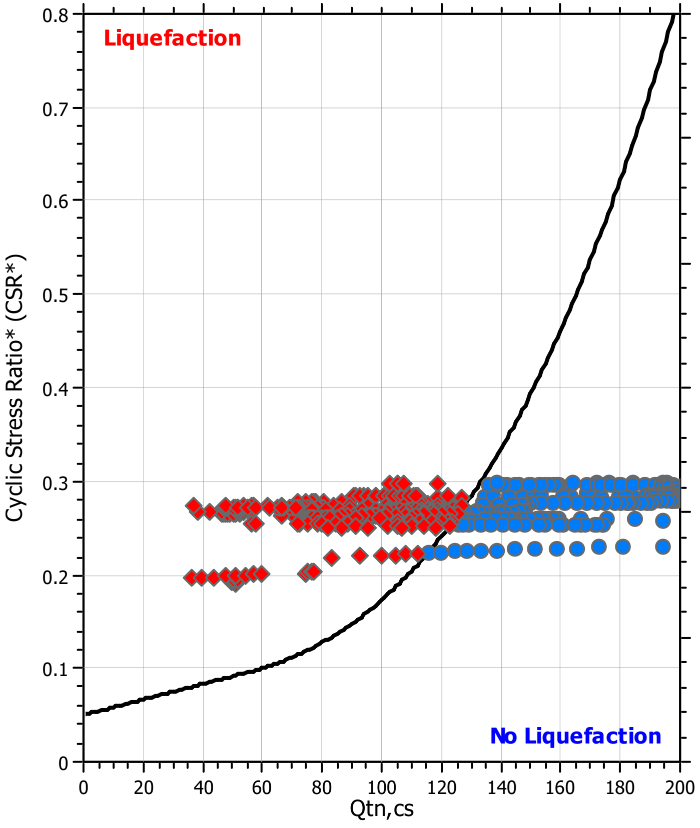
🟥 Almost certain it will liquefy
🟧 Very likely to liquefy
🟨 Liquefaction and no liq. are equally likely
🟩 Unlike to liquefy
🟩 Almost certain it will not liquefy

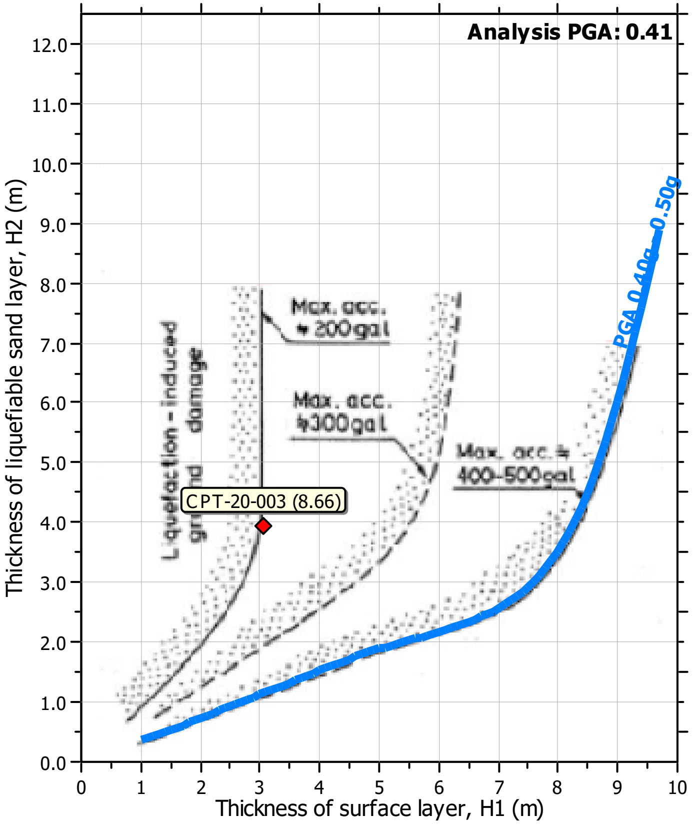
LPI color scheme

🟥 Very high risk
🟧 High risk
🟨 Low risk

Liquefaction analysis summary plots



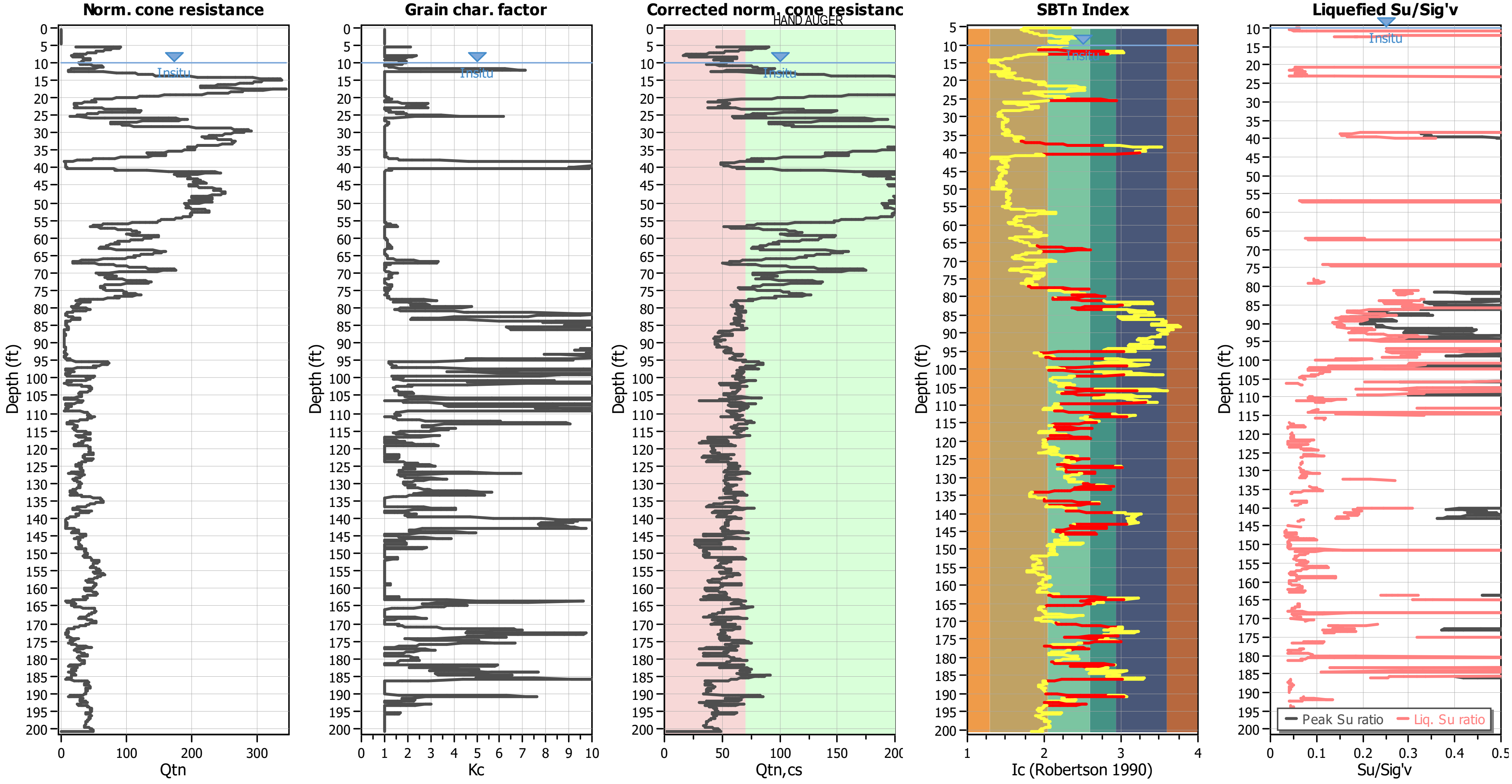




Input parameters and analysis data

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

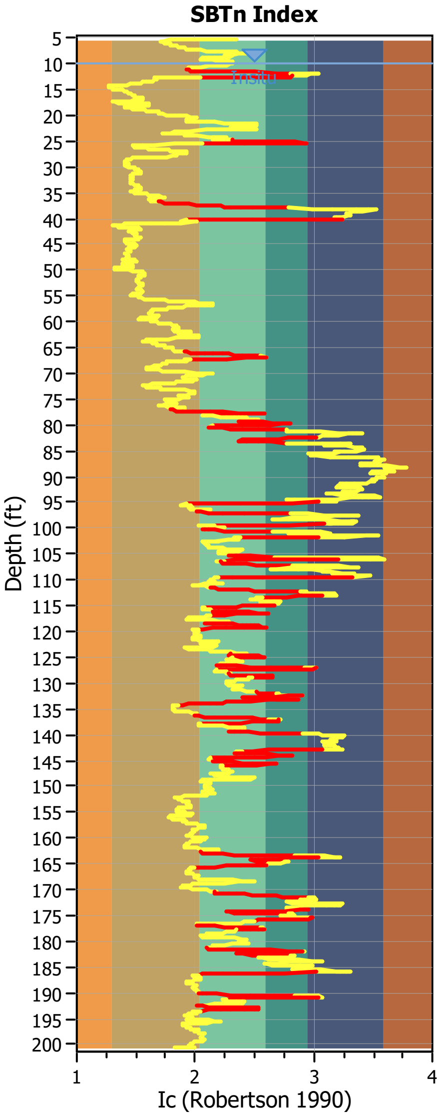
TRANSITION LAYER DETECTION ALGORITHM REPORT

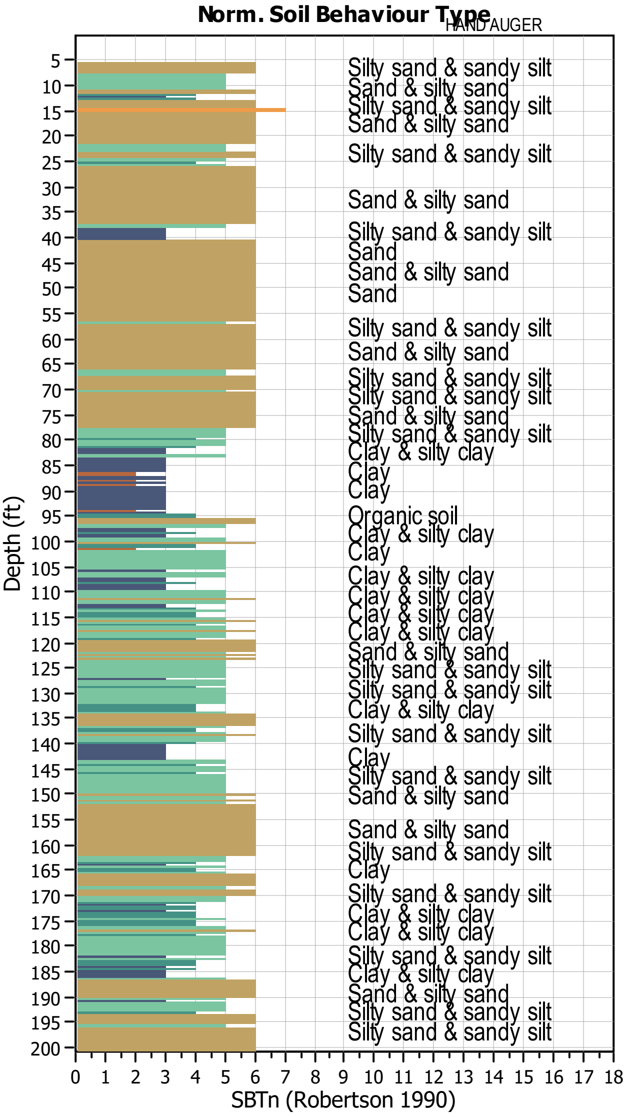
Summary Details & Plots

Short description

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.





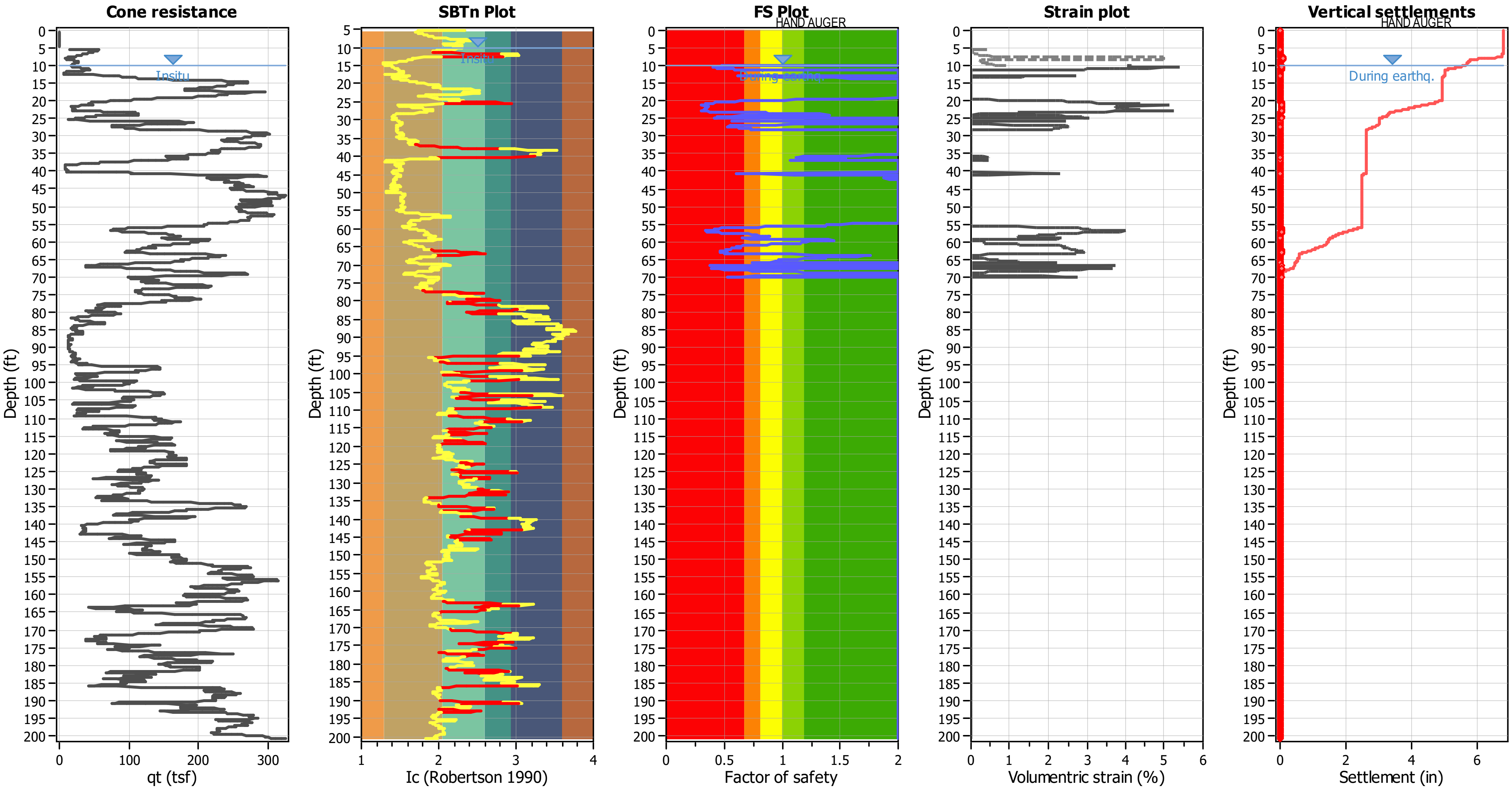
Transition layer algorithm properties

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:	3061
Total points excluded:	554
Exclusion percentage:	18.10%
Number of layers detected:	67

Estimation of post-earthquake settlements

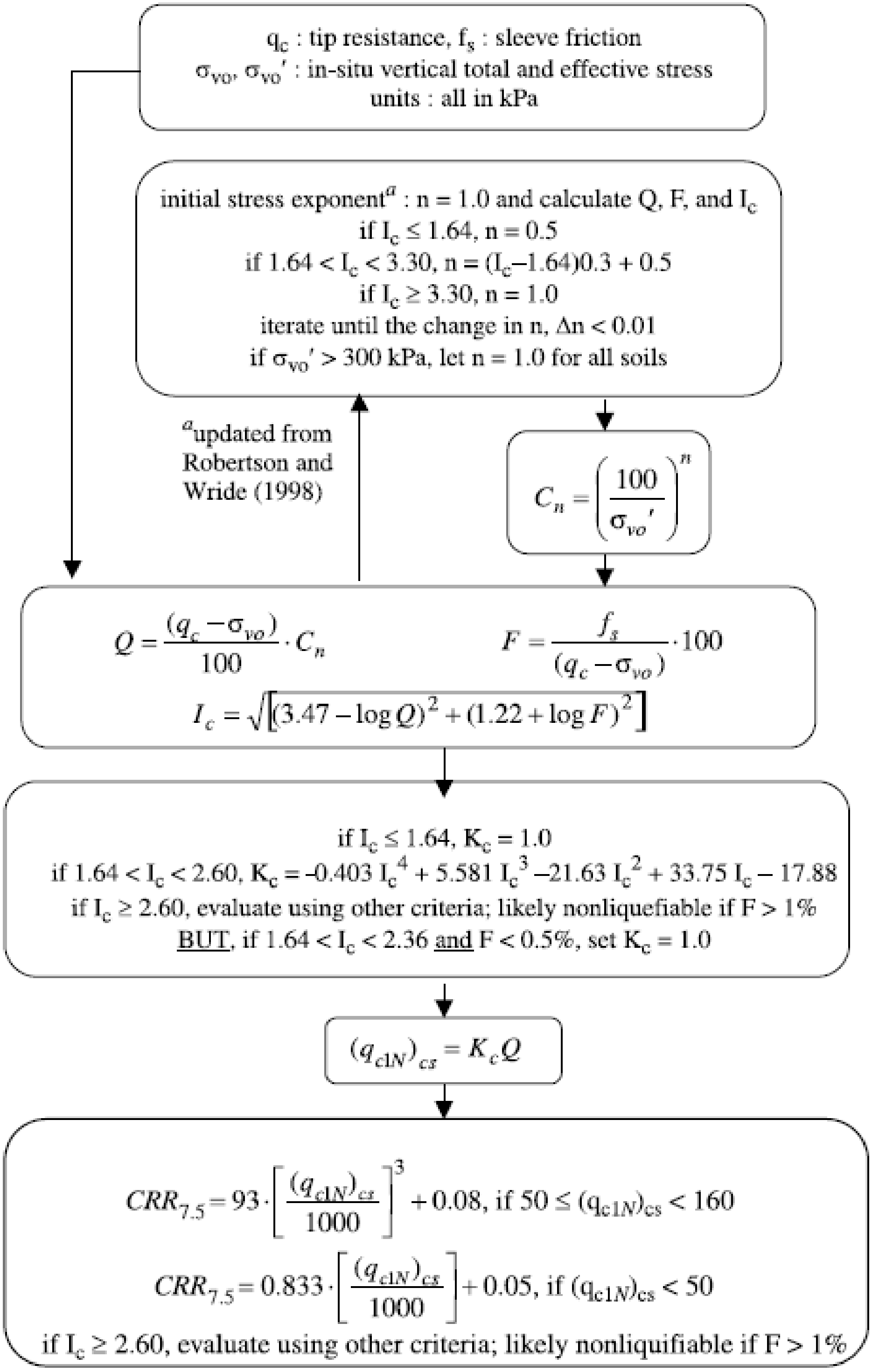


Abbreviations

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)

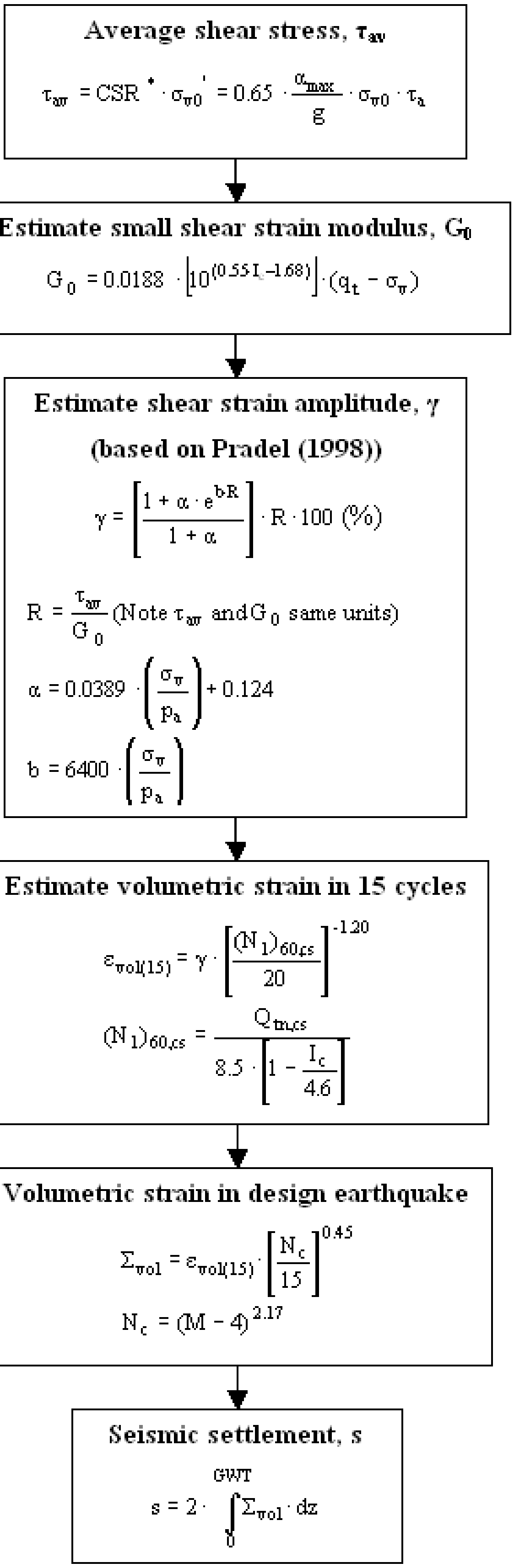
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



---

[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

$$G_0 = 0.0188 \cdot \left\lfloor 10^{(0.55 I_c - 1.68)} \right\rfloor \cdot (q_t - \sigma_v)$$

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.

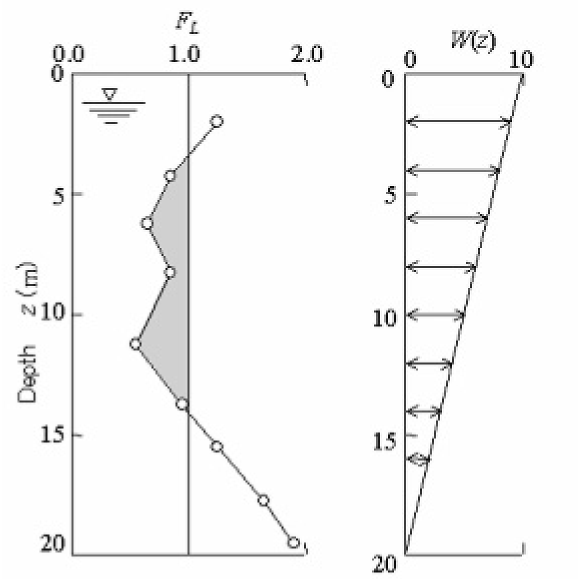
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_{0}^{20} (10 - 0{,}5_z) \times F_L \times d_z$$

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

References

- Lunne, T., Robertson, P.K., and Powell, J.J.M 1997. Cone penetration testing in geotechnical practice, E & FN Spon Routledge, 352 p, ISBN 0-7514-0393-8.

- Boulanger, R.W. and Idriss, I. M., 2007. Evaluation of Cyclic Softening in Silts and Clays. ASCE Journal of Geotechnical and Geoenvironmental Engineering June, Vol. 133, No. 6 pp 641-652

- Boulanger, R.W. and Idriss, I. M., 2014. CPT AND SPT BASED LIQUEFACTION TRIGGERING PROCEDURES. DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING COLLEGE OF ENGINEERING UNIVERSITY OF CALIFORNIA AT DAVIS

- Robertson, P.K. and Cabal, K.L., 2007, Guide to Cone Penetration Testing for Geotechnical Engineering. Available at no cost at http://www.geologismiki.gr/

- Robertson, P.K. 1990. Soil classification using the cone penetration test. Canadian Geotechnical Journal, 27 (1), 151-8.

- Robertson, P.K. and Wride, C.E., 1998. Cyclic Liquefaction and its Evaluation based on the CPT Canadian Geotechnical Journal, 1998, Vol. 35, August.

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

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

- Zhang, G., Robertson. P.K., Brachman, R., 2004, Estimating Liquefaction Induced Lateral Displacements using the SPT and CPT, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 130, No. 8, 861-871

- Pradel, D., 1998, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 124, No. 4, 364-368

- Iwasaki, T., 1986, Soil liquefaction studies in Japan: state-of-the-art, Soil Dynamics and Earthquake Engineering, Vol. 5, No. 1, 2-70

- Papathanassiou G., 2008, LPI-based approach for calibrating the severity of liquefaction-induced failures and for assessing the probability of liquefaction surface evidence, Eng. Geol. 96:94–104

- P.K. Robertson, 2009, Interpretation of Cone Penetration Tests - a unified approach., Canadian Geotechnical Journal, Vol. 46, No. 11, pp 1337-1355

- P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering - from case history to practice, IS-Tokyo, June 2009

- 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

- 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

- I. M. Idriss and R. W. Boulanger, 2008. Soil liquefaction during earthquakes, Earthquake Engineering Research Institute MNO-12

- Jonathan D. Bray & Jorge Macedo, Department of Civil & Environmental Engineering, Univ. of California, Berkeley, CA, USA, Simplified procedure for estimating liquefaction-induced building settlement, *Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering, Seoul 201*

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

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org