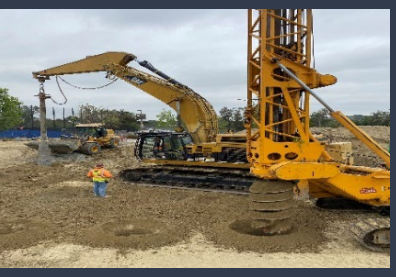
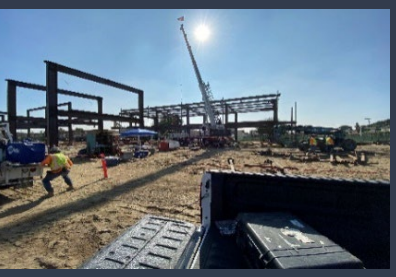
GEOTECHNICAL INVESTIGATION

RIVERSIDE CITY COLLEGE
NEW COSMETOLOGY BUILDING
4800 MAGNOLIA AVENUE
RIVERSIDE, CA 92506





RIVERSIDE COMMUNITY COLLEGE DISTRICT
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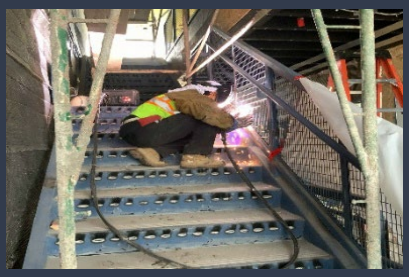
Date: November 21, 2025
MTGL Project No. 8073A06
MTGL Log No. 25-0204

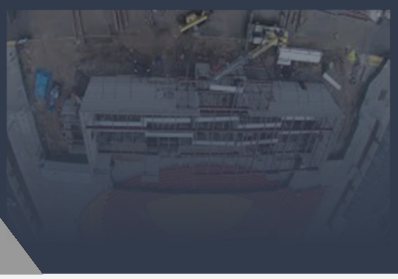
















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November 21, 2025

MTGL Project No.: 8073A06
MTGL Log No.: 25-0204
MTGL Branch: Anaheim

Mike Clark, CCM
Riverside Community College District
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Riverside, CA 92501

Subject: **GEOTECHNICAL INVESTIGATION**
Riverside City College – New Cosmetology Building
4800 Magnolia Avenue
Riverside, CA 92506

Mr. Clark,

MTGL Inc. is pleased to present this report describing the results of the geotechnical investigation for the subject project. With your authorization, we have performed this work in general accordance with our proposal dated August 15, 2025. Based on the results of our investigation, we consider the planned developments feasible from a geotechnical perspective, provided the recommendations of this report are followed.

We appreciate this opportunity to be of continued service and look forward to providing additional consulting services during the planning and construction phases of the project. Should you have any questions regarding this report, please do not hesitate to contact us.

Respectfully submitted,
MTGL, Inc.

***** DRAFT – DO NOT RELY *****

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EXECUTIVE SUMMARY

In accordance with your request and authorization, MTGL, Inc. has performed a geotechnical investigation for the proposed new Cosmetology Building on the campus of Riverside City College headquartered at 4800 Magnolia Avenue in Riverside, California. Based on information provided by the design team, the project currently consists of the construction of a new two-story building of approximately 20,000 base square feet, to be located within the existing Parking Lot G, northwest of the intersection of Saunders Street and Terracina Drive. The purpose of this investigation is to develop geotechnical conclusions and recommendations to support the planning and design of the proposed improvements.

Our subsurface investigation was performed between October 22nd and October 23rd of 2025, and consisted of drilling ten exploratory soil borings and two cone penetration tests (CPT) within areas of proposed improvements. The borings were drilled to depths ranging between 1 to 51 feet below existing ground surface (BGS) using hand tools and a truck-mounted drilling rig equipped with an 8-inch diameter hollow stem auger. The two CPT soundings (CPT-1 and CPT-2) were advanced to refusal at approximately 29 and 38 feet BGS, respectively, using a truck-mounted hydraulic CPT rig. Two of the shallow borings at the project perimeter were converted to borehole percolation tests to evaluate infiltration feasibility. An MTGL engineer logged the borings and collected samples of the encountered materials for geotechnical laboratory testing. Selected samples were tested in our laboratory to evaluate their engineering properties. Data obtained using CPTs was evaluated in conjunction with the samples collected from the auger borings to develop an idealized soil profile.

As encountered, the site is underlain by a thin veneer of undocumented artificial fill (Qf) over young axial channel deposits (Qya), which in turn overlie denser old alluvial fan deposits (Qof). Undocumented fill was encountered in each of the borings and generally extended to depths of about 2 to 3 feet BGS, although locally deeper fill was observed within existing planter areas. As encountered, the fill materials typically consist of loose to medium dense, fine- to coarse-grained silty sands and clayey sands, and soft to medium stiff sandy to silty lean clays. Young axial channel deposits encountered beneath the fill generally extend to between 15 and 20 feet BGS and consisted of loose to medium dense, fine to medium grained, clayey and silty sands and soft to firm sandy silty clays and clayey to sandy silts. These materials are underlain by older alluvial fan deposits (Qof) at depth. As encountered, the older alluvial fan deposits consist of dense to very dense fine- to coarse-grained sands with interbeds of gravel and occasional hard, sandy lean clay. Groundwater was not encountered in our borings to a maximum explored depth of about 51 feet BGS, and available regional data indicate historical groundwater levels generally deeper than approximately 30 to 50 feet BGS.

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The principal geotechnical considerations for the project include the presence of undocumented fill and loose to soft young alluvial materials in the upper approximately 15 to 20 feet, which are unsuitable for direct support of conventional shallow foundations, and the potential for significant seismically induced dry-sand settlement within the upper alluvial deposits. Accordingly, we recommend that settlement-sensitive structures be supported on deep foundations or on shallow foundations underlain by improved ground (typically improved with CAPs). For improvements not supported on piles or improved ground, additional seismic settlement should be anticipated within the upper soil profile, and the design team should incorporate appropriate settlement tolerances into the project documents.

Other geotechnical considerations for the project include the presence of a concrete-lined, approximately 5-foot-deep drainage channel along the western site boundary and potentially moisture-sensitive near-surface soils. Recommendations regarding foundation setbacks from slopes and surficial remedial grading are provided in later sections of this report. Site drainage should also be configured to provide positive drainage away from improvements and to minimize saturation of near-surface soils.

Based on borehole percolation testing and the subsurface conditions, stormwater infiltration is not recommended; the site is classified as “No Infiltration” for BMP feasibility, and any BMP facilities should be lined and drained in accordance with our recommendations.

Based on the results of our investigation, we consider the planned developments feasible from a geotechnical perspective, provided the recommendations of this report are followed. The recommendations herein are based on preliminary design information and may require refinement as the design progresses.

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Appendix A – References
Appendix B – Field Investigation
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Appendix D – General Earthwork and Grading Specifications
Appendix E – Borehole Percolation Test Results
Appendix F – Liquefaction and Seismic Settlement Analysis
Appendix G – Axial Pile Analysis

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1.00 INTRODUCTION

In accordance with your request and authorization, MTGL, Inc. has completed a geotechnical investigation for the subject site. The following report presents a summary of our findings, conclusions and recommendations based on our field investigation, laboratory testing, and engineering analysis.

1.01 SITE DESCRIPTION

The project site is located within the Riverside City College (RCC) campus in the City of Riverside, California. The site lies within the existing Parking Lot G, associated with the main RCC campus at 4800 Magnolia Avenue, Riverside, California.

The site is bounded by Terracina Drive to the south, Saunders Street to the east, tennis courts and the Campus Police Station to the north, and a concrete-lined trapezoidal drainage channel (approximately 5 feet deep) to the west. Topography at the site is relatively level, sloping gently toward the south. Ground surface elevations range from approximately 802 to 806 feet above mean sea level (MSL) (Google Earth, 2025). The approximate site location is shown on Figure 1 (Site Location Map), and the local topography is presented on Figure 2 (Site Topography Map).

1.02 PLANNED CONSTRUCTION

Based on information provided by the design team, the project currently consists of the construction of a new two-story building of approximately 20,000 base square feet, to be located within the existing Parking Lot G on the Riverside City College campus. The project remains in the planning phase; therefore, our understanding of the proposed improvements is based on preliminary schematic information made available at this time. The purpose of this investigation is to develop geotechnical conclusions and recommendations to support the planning and design of the proposed project.

1.03 SCOPE OF WORK

We conducted this investigation in general conformance with the scope of work presented in our proposal No. P-25-610, dated August 15, 2025. The scope of our geotechnical services included the following:

- Reviewing readily available literature and maps to obtain background information on regional geology, site development, seismicity, and groundwater.
- Marking out boring locations on the site and contacting Underground Service Alert (USA) to locate onsite utility lines.
- Utilizing a private utility locator service to clear borings of potential underground utility conflicts.

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- Drilling, logging, and sampling of ten exploratory borings using hand tools and an 8-inch hollow stem auger drill rig to a maximum depth of 51 feet below ground surface (BGS).
- Advancing two truck mounted electronic cone penetrometer (CPT) to a maximum depth of 37½ feet BGS.
- Converting two shallow borings to borehole percolation tests.
- Performing two borehole percolation tests.
- Performing laboratory testing of select samples.
- Performing geotechnical engineering review of compiled data and performing geotechnical engineering analyses.
- Preparing this report summarizing our findings and presenting our conclusions and recommendations for the proposed construction.

1.03.1 FIELD INVESTIGATION

Prior to performing our field investigation, a site reconnaissance was performed by an MTGL engineer to observe the existing surface conditions, mark the proposed boring locations, evaluate each location with respect to obvious subsurface structures and determine access for the drilling rig. USA Digalert was subsequently notified of the marked locations for utility clearance as required by law. In addition, a private utility locator, GPRS was subcontracted to locate and mark potential utility conflicts.

Our subsurface investigation was performed on October 22nd and October 23rd of 2025, and consisted of drilling ten exploratory soil borings within areas of proposed improvements. Ten borings (B-1 through B-10) were drilled to depths ranging from about 1 to 51 feet BGS using hand tools and a truck-mounted drilling rig equipped with an 8-inch hollow stem auger. Hand auger auger refusal was encountered in B-7 at 2 feet BGS due to densely packed roots, and at approximately 1-foot BGS due to subsurface geogrid in Boring B-1. Additionally, two CPT soundings (CPT-1 and CPT-2) were advanced to a depths of approximately 29 and 37 feet BGS, respectively, using a truck-mounted electronic CPT rig. Refusal was encountered within very dense alluvium in both CPT soundings. See Appendix B for further discussion of the field exploration and logs of test borings. Figure 3 presents a Subsurface Exploration Map.

An MTGL engineer logged the borings and collected samples of the encountered materials for geotechnical laboratory testing. Representative disturbed bulk soil samples were obtained from the borings in the upper five feet. In-situ testing, including driving Modified California (CAL) and Standard Penetration Test (SPT) samplers at selected depth intervals, was performed and relatively undisturbed samples were obtained. Samplers were driven into the bottom of the boring with successive drops of a 140-pound weight falling 30 inches.

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The energy-corrected number of blows per foot required to drive the CAL and SPT samplers are shown on the boring logs in the N60 column (Appendix B). A conversion factor of 0.65 was used to normalize N60 values obtained by Modified California samplers (Burmister, 1948). SPTs were performed in general accordance with the American Society for Testing and Materials (ASTM) D1586 standard test method.

Soil samples were inspected and described in general conformance with the Unified Soil Classification System (USCS). Samples were sealed and packaged for transportation to our geotechnical laboratory. After completion of drilling, borings were backfilled with native soil cuttings and asphalt patched with Aquaphalt® where appropriate.

1.03.2 PERCOLATION TESTING

MTGL performed a preliminary infiltration feasibility study at the site on October 23, 2025. At the time of the field investigation, the project was in the planning phase and no preliminary BMP exhibits were available for review. Two shallow borings (P-1 and P-2) were drilled to depths ranging from 5 to 10½ feet BGS using hand tools and a truck mounted drill rig. The borings were converted to borehole percolation tests to evaluate infiltration feasibility. The tests were prepared and performed by an MTGL engineer in general accordance with methods adopted by the Riverside County Low Impact Development BMP Design Handbook, 2011. To prevent caving, a 4-inch diameter perforated pipe was placed in the borehole with 2 inches of gravel placed at the bottom of the hole. The test results of the percolation test are presented in Section 4.11 Preliminary Infiltration Feasibility.

1.03.3 LABORATORY TESTING

Laboratory testing was performed on select samples to verify the field classification of the recovered samples and evaluate the geotechnical properties of the subsurface materials. Laboratory tests were performed in general conformance with applicable ASTM or State of California Department of Transportation (Caltrans) standard methods. The results of our laboratory tests are presented in Appendix C.

1.03.4 GEOTECHNICAL ANALYSIS AND REPORT PREPARATION

This report was prepared to present our geotechnical conclusions and recommendations based on the findings of the background review, field exploration, laboratory testing, and geotechnical analysis of the compiled data.

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2.00 FINDINGS

The following sections present our findings of our office research, field investigation, and laboratory testing.

2.01 REVIEW OF PREVIOUS GEOTECHNICAL STUDIES

To aid in the preparation of this report, we have reviewed the geotechnical reports previously performed in close proximity to the project site. A summary of each is provided below.

PREVIOUS GEOTECHNICAL REPORTS REVIEWED

Author, Year	Title
Moore Twining Associates, 2023	Geotechnical Engineering Investigation for Proposed Solar Canopies, RCC Parking Lots C & E, Project No. H10303.01, dated April 14, 2023
CHJ, Inc., 2007	Geotechnical Investigation, Proposed Cutter Pool Facilities, Riverside Community College District, Project No. 07298-3, dated May 3, 2007

2.01.1 MOORE TWINING ASSOCIATES, INC. (2023)

This investigation included the drilling, logging, and sampling of twenty one (21) exploratory borings, drilled with an 8 inch hollow stem auger extending to depths of approximately 3½ to 51½ feet BGS. The borings were drilled in the existing parking Lots C and E, which are located approximately 1,000 ft west and 50 feet south of the proposed new cosmetology building site, respectively.

Their borings in Lot E encountered artificial fill ranging from 1½ to 11½ feet BGS, generally consisting of silty sands, clayey sands, sandy clays, and clayey soils with variable debris content. Beneath the fill, native alluvial soils were encountered, typically consisting of silty sands to poorly graded sands, consistent with regional Quaternary alluvial fan and axial channel deposits. Materials ranged from loose to medium dense in the upper profile, increasing to dense conditions at depth. Groundwater was not encountered in any of the borings to the maximum depth explored of 51½ feet BGS

2.01.2 CHJ, INC. (2007)

This investigation included the drilling, logging, and sampling of three hollow-stem auger borings extending to depths of approximately 51 feet BGS and four Cone Penetrometer Test (CPT) soundings advanced to depths ranging from 23½ to 32 feet BGS due to refusal. This investigation was performed at the swimming pool and aquatic center located north of the proposed cosmetology building site.

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The borings encountered alluvial materials originating from the Box Springs Mountains to the east of the site. Based on CHJ's exploratory borings, the alluvium encountered generally consisted of reddish-brown silty sands to sands with moderate amounts of clay. Bedrock was not encountered in the borings. Groundwater was encountered at approximately 38 feet below existing grade in one boring. However, it is noted that the project site is located within an area with historic high groundwater depths of approximately 30 to 50 feet BGS.

2.02 GEOLOGIC SETTING

The site is located within the Peninsular Ranges Geomorphic Province of California, which stretches from the Los Angeles basin to the tip of Baja California in Mexico. This province is characterized as a series of northwest trending mountain ranges separated by subparallel fault zones, and a coastal plain of subdued landforms. The mountain ranges are underlain primarily by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the southern California batholith, while the coastal plain is underlain by subsequently deposited marine and non-marine sedimentary formations.

More specifically, the proposed site is located within the northern portion of the relatively stable Perris Block. The Perris Block, approximately 20 miles by 50 miles in extent, is bounded by the San Jacinto Fault Zone to the northeast, the Chino Fault to the West and Elsinore Fault Zone to the southwest. The Perris Block has had a complex tectonic history, apparently undergoing relative vertical land-movements of several thousand feet in response to movement on the Elsinore and San Jacinto Fault Zones. Thin sedimentary and volcanic materials locally mantle crystalline bedrock, consisting of the Val Verde Tonalite (Kvt) and lesser amounts of Cretaceous granitic dikes (Kg). Figure 4 presents a map of regional geology within the vicinity of the site.

2.03 SUBSURFACE CONDITIONS

As encountered in our borings, the site is underlain by artificial fill (Qf), Young Axial Channel Deposits (Qya), and Old Alluvial Fan Deposits (Qof). Materials encountered in our borings are generally consistent with the mapped geologic units presented by Morton, et.al (1988, 2002). Figures 5A & 5B presents Geologic Cross Sections A-A' and B-B', respectively. Descriptions of the materials encountered in the borings are presented below. Additional descriptions of materials encountered are presented on the boring logs in Appendix B.

Artificial Fill (Qf) – Artificial fill was encountered in each of the borings either at the ground surface or beneath the existing pavement sections (asphalt and aggregate base). The fills appeared to be mostly derived of local alluvial materials and extended to depths ranging from approximately 1 to 5 feet BGS. As encountered, the fill generally consisting of variably colored, loose to medium dense, fine to coarse grained, silty to clayey sand with variable

amounts of silt, and medium stiff, silty, and sandy lean clays. Occasional gravels and trace debris were also encountered within the fill materials.

No documentation regarding the placement of the existing fill underlying the site was available at the time of this report. Therefore, the fill is considered undocumented and unsuitable for the support of proposed structures.

Young Axial Channel Deposits (Qya) – Holocene to late Pleistocene-age, younger alluvium was encountered below the fill. As encountered, these materials consisted of fine to medium grained clayey sands, silty sands, and silty clayey sands interbedded with fine grained, sandy lean clays, silty clays, and silts. Few to trace amounts of gravel were observed in these units. These deposits were generally light brown to brown and loose to medium dense or soft to stiff. As encountered, the young axial channel deposits extend to a depth of approximately 12½ to 17½ feet BGS.

Old Alluvial Fan Deposits (Qof) – Late to middle Pleistocene-age older alluvial fan deposits were encountered beneath the Young Axial Channel Deposits in Borings B-2, B-4, and B-5. These materials extended to the entire depths explored. As encountered, the old alluvial fan deposits predominantly consisted of medium to coarse grained, light brown to reddish brown and gray, clayey sands, silty sands, and poor- to well-graded sands. These materials were generally dense to very dense with the exception of a relatively thin and locally discontinuous, medium dense, surface layer encountered in B-4 and CPT-2. Minor interbeds of hard sandy lean clay were also encountered within the unit. The presence of interbedded gravel layers was also inferred based on occasional intervals of turbulent and difficult drilling observed during our investigation.

No physical samples were recovered from the cone penetration tests (CPTs); therefore, definitive soil classifications could not be established directly from the CPT soundings. However, stratigraphic interpretations were developed based on correlations between cone tip resistance, sleeve friction, and pore pressure measurements obtained during testing. Soil behavior types were interpreted using the classification methodology proposed by Robertson (2009). A summary of CPT data and interpretations is provided in Appendix B.

The interpreted soil behavior types from the CPTs are generally consistent with the lithologic conditions encountered in the soil borings, where direct sampling was performed. The CPT and boring data were used together to evaluate subsurface conditions across the site.

A summary of boring CPT data is presented in the table below.

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SUMMARY OF SUBSURFACE SOIL CONDITIONS

Boring No.	Drilled Depth (Feet)	Latitude (Degrees)	Longitude (Degrees)	Surface Conditions	Existing Ground Elevation (Feet)*	Approximate Thickness of Fill (Feet)	Groundwater Depth Below Ground Surface (Feet)
B-1	1	33.9700	-117.3793	Asphalt	802	1	Not Encountered
B-2	20½	33.9698	-117.3793	Asphalt	803	1	Not Encountered
B-3/P-1	5	33.9697	-117.3790	Grass	806	2	Not Encountered
B-4	51	33.9696	-117.3793	Asphalt	803	2	Not Encountered
B-5	21½	33.9695	-117.3790	Asphalt	803	1	Not Encountered
B-6	5	33.9695	-117.3786	Grass	802	2	Not Encountered
B-7	2	33.9694	-117.3793	Grass	804	2	Not Encountered
B-8	5	33.9694	-117.3787	Grass	803	2	Not Encountered
B-9	11½	33.9693	-117.3791	Asphalt	804	1	Not Encountered
B-10/P-2	10½	33.9693	-117.3789	Asphalt	804	1½	Not Encountered
CPT-1	29	33.9694	-117.3789	Asphalt	803	N/A	Not Encountered
CPT-2	37½	33.9695	-117.3793	Asphalt	803	N/A	Not Encountered

*Approximated using Google Earth

2.04 SURFACE AND GROUNDWATER CONDITIONS

Groundwater was not encountered in the borings. No areas of ponding or standing water were present at the time of our study. Further, no springs or areas of natural seepage were found. Additionally, no visible seepage was observed within the borehole after approximately one hour after drilling.

Based on our review of readily available data, groundwater in the immediate site vicinity is estimated to be at depth in excess of 50 feet below existing grade (Carson and Matti, 1985).

We do not anticipate groundwater to be encountered during the proposed construction. However, changes in rainfall, irrigation and local drainage may produce seepage or perched groundwater conditions below the site. These conditions are generally difficult to predict and typically mitigated if and when they occur.

2.05 HISTORICAL GROUNDWATER

Historical groundwater data directly beneath the project site is limited; however, groundwater data sourced from three monitoring wells near the project vicinity was obtained and reviewed (DWR, 2025). The following presents a summary of the historical groundwater information reviewed by our office.

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GROUNDWATER LEVELS IN NEARBY WELLS

Local Well ID	Fox Metro	Clearwater	Nichols Park
CA State Well No.	02S05W23F001S	02S05W25F001S	02S05W32B001S
Latitude (degrees)	33.984	33.969	33.9589
Longitude (degrees)	-117.375	-117.359	-117.4262
Distance (Miles)	1.02	1.16	2.80
Relative Location from Site	NE	East	SW
Historic High Elevation (ft)	768.36 (4/25/2012)	775.78 (4/25/2012)	734.74 (5/19/1983)
Depth from Ground Surface (ft)	74.64	140.78	47.62
Collecting Agency	WMWD	WMWD	USGS

In addition to monitoring-well data, groundwater observations from nearby geotechnical investigations were also reviewed. As noted in Section 2.01, groundwater was encountered at approximately 38 feet BGS in one boring drilled approximately 850 feet northeast of the site. No groundwater was encountered during a recent investigation performed by Moore Twining Associates (2023) approximately 50 feet south of the project site, to a maximum explored depth of 51½ feet BGS.

Based on the available data, groundwater is expected to occur at depths that will not influence most near-surface construction activities. However, groundwater levels may vary over time due to seasonal rainfall, irrigation, broken utilities, subsurface geologic structures, or changes in surface drainage. Because groundwater rise or seepage is inherently variable and difficult to predict, such conditions are typically addressed if and when encountered during construction.

Groundwater may be encountered during deep excavations or foundation elements extending below approximately 25 feet BGS. Accordingly, recommendations for managing potential groundwater conditions are provided in the following sections of this report.

2.06 GEOLOGIC HAZARDS

Geologic hazards are summarized and discussed with respect to the site and proposed development below.

2.06.1 STRONG GROUND MOTION AND MAPPED SEISMIC DESIGN PARAMETERS

A primary geologic hazard affecting the project is strong ground shaking due to movement on nearby active faults (USGS, 2024). To support seismic site classification in accordance with ASCE 7-22, shear-wave velocity (Vs) measurements were obtained at approximately 5-foot intervals in CPT-1 and CPT-2. As noted in Section 1.03.1, early refusal was encountered in dense older alluvial fan deposits in both CPTs; therefore, shear-wave velocity sounding depths were limited and could not extend to the conventional 30-meter (100-foot) interval required for direct calculation of Vs_{30}.

However, information from nearby borings such as B-4 indicates that relatively dense/stiff older alluvium continues to depths of at least 51 feet below the existing ground surface. Because these materials are dense and shear-wave velocity typically increases with confining stress, the deepest measured Vs value from each CPT was conservatively extrapolated to a depth of 100 feet to develop a representative depth-averaged Vs_{30} profile.

Using this approach, the computed Vs_{30} values ranged from approximately 1,489 ft/s to 1,562 ft/s. Considering the inherent uncertainty associated with extrapolating shear-wave velocities in the underlying deposits, it is our opinion that the site can be classified as Site Class CD in accordance with ASCE 7-22. The mapped site coefficients and maximum considered earthquake (MCER) spectral response acceleration parameters corresponding to this site class and project location are presented below:

2025 CALIFORNIA BUILDING CODE (ASCE 7-22) – MAPPED SITE COEFFICIENTS

Site Coordinates	
Latitude	**Longitude**
33.96956°	-117.37920°

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	CD
Mapped Spectral Response Acceleration at Short Period, S_s	1.58 g
Mapped Spectral Response Acceleration at 1-Second Period, S_1	0.60 g
Mapped Design Spectral Acceleration at Short Period, S_{DS}	1.22 g
Design Spectral Acceleration at 1-Second Period, S_{D1}	0.78 g
Probabilistic (2%-in-50-years), Peak Ground Acceleration, PGA_{uh}	0.83 g
Deterministic 84th-percentile, Peak Ground Acceleration, PGA_{84th}	0.64 g
Site Modified Peak Ground Acceleration, PGA_m	0.64 g

2.06.2 HISTORIC EARTHQUAKES

The site is located in a seismically active area, as is the case throughout the majority of southern California. At this time, it is not possible to state with certainty when or where future large-magnitude earthquakes will occur, or what the magnitude or intensity of these events will be. Historic earthquakes that have occurred within approximately 60 miles (100 kilometers) of the site are summarized in the table below.

The nearest of these historic earthquakes was the magnitude 7.3 Landers earthquake of June 28, 1992. Its epicenter was located approximately 56.3 miles to the northeast of the site. The earthquake resulted from the rupture of five major and many minor right lateral faults near the southern end of the eastern California shear zone. Our research of regional geologic and seismic data did not reveal any known instances of ground failure associated with regional seismic activity within the project site as a result of the earthquake.

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HISTORIC SEISMICITY

Site Coordinates			
Latitude		**Longitude**	
33.96956°		-117.37920°	
Date	**Event/Location**	**Magnitude (M_W)**	**Epicentral Distance From Site (Miles)**
April 21, 1918	San Jacinto	6.7	27.4
March 11, 1993	Long Beach	6.4	42.7
July 8, 1986	North Palm Springs	6.0	44.3
October 1, 1987	Whittier Narrows	5.9	40.7
June 28, 1991	Sierra Madre	5.8	40.8
June 28, 1992	Landers	7.3	56.3
June 28, 1992	Big Bear	6.3	35.5

2.06.3 ACTIVE FAULTING AND FAULT-RUPTURE HAZARD

The closest known active fault is the San Jacinto fault zone (San Bernardino Section) located approximately 6.8 miles to the east of the project site, and the Elsinore fault zone (Chino section) located approximately 15 miles to the southwest of the project site (USGS, 2025). Other regional faults capable of generating seismic hazards include the San Andreas to the east, and the Chino, Whittier, and the Newport Inglewood faults to the west.

The site is not located in an Alquist-Priolo Earthquake Fault Zone. Based on our review of the referenced fault databases and geologic maps, no active faults are known to underlie or project toward the site; therefore, the potential for fault rupture occurrence at the site is considered low. Figure 6 presents a regional fault activity map within the vicinity of the site.

2.06.4 LIQUEFACTION AND DYNAMIC DRY SETTLEMENT

Liquefaction is a phenomenon in which saturated soil temporarily loses shear strength and behaves like a liquid due to intense shaking, typically from an earthquake. Reduction of shear strength occurs when the pore water pressures within a soil matrix increase, causing the soil particles to lose contact with each other. The potential for liquefaction is dependent upon several factors including soil grain size, relative density, confining pressure, degree of saturation, strength of ground motions, and duration of ground shaking. Effects of severe liquefaction can include excessive settlements, bearing capacity failures, lateral spreading, and other mechanisms of failure.

The subject project site is not located within an area mapped by the California Geological Survey (CGS) for liquefaction hazards; however, the Riverside General Plan (2025) identifies the site as having moderate liquefaction susceptibility. While groundwater was not

encountered within depths up to 51 feet in our exploratory borings, occasional perched or seasonally elevated groundwater conditions may occur. Additionally, subsurface data from our field investigations indicate the presence of relatively loose alluvial soils that may be susceptible to liquefaction or seismic settlement under strong seismic loading. Therefore, a preliminary evaluation of liquefaction potential and dry-sands settlement was performed.

Our liquefaction and dry-sand seismic settlement analyses were conducted in general accordance with the recommended procedures of DMG Special Publication 117A (CGS, 2008). CPT soundings CPT-1 and CPT-2 were analyzed using CLiq® v.2.3.1.15 (GeoLogismiki), and results were compared with our boring data to form our opinion regarding the liquefaction potential of the encountered soils. The CPT-based method of Boulanger and Idriss (2014) was used to estimate susceptibility to liquefaction.

Our analysis considered a site adjusted Peak Ground Acceleration (PGAm) of 0.64 g, based on mapped seismic values presented in Section 2.06.1, and a modal moment magnitude of 8.1 based on USGS deaggregation. The modal magnitude was determined using the USGS Earthquake Hazard Tool (2023), using a 2475-year return period (2% in 50 years) deaggregation. Per CGS's Note 48, liquefaction induced settlement was calculated for layers with a factor of safety (FS) less than 1.3. A preliminary design groundwater depth of 25 feet BGS was considered to account for potential elevated groundwater conditions. Evaluation of dry sands seismic settlement was also included in the analyses. Dry sand seismic settlements were estimated using methods proposed by Pradel (1998) and a factor of 2.0 was used to account for the multidirectional nature of earthquakes.

Based on the results of our preliminary liquefaction analysis, most soil layers below the site are not considered likely to liquefy under the design-level seismic loading. However, significant amounts of dry sands settlement were calculated in our analyses. The total seismically induced vertical settlements for free field conditions were estimated to be on the order of 4½ to 6½ inches, with differential settlements assumed to be approximately one half of the total settlement across 30 feet. The calculated amount of free-field settlement for each CPT is presented in the following table.

SUMMARY OF SEISMIC SETTLEMENT

CPT No.	Calculated Seismic Settlement (Inches)
CPT-1	6.3
CPT-2	4.5

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The results of our preliminary liquefaction analyses are presented in Appendix E. Recommendations for mitigating liquefaction potential using ground improvement are presented in subsequent sections of this report.

2.06.4.1 BEARING FAILURE

When liquefaction occurs, soils can temporarily lose shear strength and bearing capacity, potentially resulting in foundation failure or uplift of lightweight buried structures. Based on our analyses, the potential for liquefaction to adversely affect the site is relatively low, and the likelihood of liquefaction-related bearing capacity failure is correspondingly low.

However, our evaluation indicates that significant dry-sand seismic settlement may occur within the loose, unsaturated alluvial soils underlying the site. Although these soils are not expected to liquefy, cyclic densification during strong ground shaking can produce several inches of total and differential settlement. Such movements may affect slabs-on-grade, pavements, utilities, and shallow foundations if not properly addressed.

Recommendations for mitigating seismic settlement, such as ground improvement or the use of deep foundations, are presented in subsequent sections of this report.

2.06.4.2 LATERAL SPREADING (LATERAL DISPLACEMENT)

Lateral spreading is a condition in which a relatively stiff block of soil, underlain by liquefied soils, moves laterally toward a free face or slope. Such movements typically occur on gently sloping ground adjacent to features like river channels or incised drainageways and can result in fissures, scarps, and significant horizontal displacement. These movements have the potential to disrupt foundations, utilities, pavements, and other structures that cross or are supported by the affected ground.

A concrete-lined drainage channel is located along the northwest side of the site, with an approximate depth of 5 feet and side slopes on the order of 1:1 (horizontal to vertical). Although this channel represents a potential free face, the relatively deep design groundwater level and the relatively impervious concrete lining substantially reduce the likelihood that liquefiable conditions would develop beneath or adjacent to the channel. As a result, the potential for lateral spreading to affect the site is considered low.

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It should be noted that localized soil saturation could occur during stormwater conveyance if the channel lining becomes cracked or compromised. To minimize this potential, regular inspection and maintenance of the drainage channel should be performed to ensure the integrity of the concrete lining is maintained.

2.06.4.3 LIFELINE HAZARDS

Seismically induced settlement of structures may also pose problems for streets and lifelines. Specifically, critical utilities may rupture, creating safety hazards and interrupting essential services. Therefore, consideration should be given to providing isolated and/or flexible connections for gas, water, and other critical utility lines as a preventive measure.

2.06.5 TSUNAMIS, SEICHES, DAM INUNDATION

The site is not mapped as being located within an area mapped by the California Geological Survey as subject to inundation by tsunami (State of California, 2021). Therefore, the potential for a tsunami to affect the site is considered low.

Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The site is not located adjacent to any confined bodies of water; therefore, the potential for a seiche to affect the site is considered low.

The site is not mapped as being located within a flood hazard area as identified by FEMA flood hazard maps and the city of Riverside General Plan Flood Hazard Areas Map PS-4 (2025). According to the Flood Insurance Rate Map (FIRM Map 06065C0710H, 2024), the site is mapped as being located within an area designated as Zone X (FEMA, 2024). Zone X is defined as an area of minimal flood hazard.

However, the drainage channel alignment adjacent to the site is mapped as an area within a potential inundation zone for seismically induced dam/reservoir failure from the Sycamore Canyon Dam and the Box Springs Dam (Riverside General Plan, 2025). These dams are monitored by the Riverside County Flood Control and Water Conservation District and State of California.

2.06.6 LANDSLIDES AND SLOPE STABILITY

Based on our review of available literature, no landslides have been mapped within the project boundaries. Additionally, no evidence of landslides was observed during our site reconnaissance and subsurface investigation. Given the relatively flat topography, and lack

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of surficial landslide evidence at the site, the potential for landslides or slope instabilities to occur at the site is considered low.

2.06.7 SUBSIDENCE

The site is not mapped as being located in an area of known subsidence associated with fluid withdrawal (groundwater or petroleum) (USGS, 2024); therefore, the potential for subsidence due to the extraction of fluids is considered negligible.

2.06.8 HYDRO-CONSOLIDATION

Hydro-consolidation can occur in recently deposited sediments (less than 10,000 years old) that were deposited in a semi-arid environment. Examples of such sediments are aeolian sands, alluvial fan deposits, and mudflow sediments deposited during flash floods. The pore spaces between the particle grains can re-adjust when inundated by groundwater causing the material to consolidate. Loose and uncompacted fill materials can also be susceptible to collapse upon wetting. The underlying younger axial channel materials were tested for one-dimensional wetting-induced collapse. The laboratory test results indicated minimal collapse. The relatively dense older alluvial materials underlying the site are not considered susceptible to hydro-consolidation.

2.06.9 EXPANSION AND SHRINKAGE

The on-site near surface materials were tested for expansion index in response to wetting. The results from our laboratory testing indicate an expansion index of 29 which classify the materials as having a low expansion potential. Recommendations for remediating expansive soils are provided in Section 4.01 of this report.

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3.00 CONCLUSIONS

3.01 GENERAL CONCLUSIONS

Based on our geotechnical review of the planned construction, it is our opinion that the site is suitable for the proposed improvements provided that the conclusions and recommendations presented in this report are incorporated into the project design, included in the construction documents, and implemented during grading and foundation construction. These recommendations are based on preliminary design information and may require refinement once final plans are developed; MTGL should review the completed design to confirm consistency with the geotechnical recommendations.

The principal geotechnical considerations for the project include the presence of undocumented fill and loose to soft young alluvial materials in the upper approximately 15 to 20 feet, which are unsuitable for direct support of conventional shallow foundations, and the potential for significant seismically induced dry-sand settlement within the upper alluvial deposits. Accordingly, we recommend that settlement-sensitive structures be supported on deep foundations or on shallow foundations underlain by improved ground (typically improved with CAPs). For improvements not supported on piles or improved ground, additional seismic settlement should be anticipated within the upper soil profile, and the design team should incorporate appropriate settlement tolerances into the project documents.

Additional geotechnical considerations include the concrete-lined, approximately 5-foot-deep drainage channel along the western site boundary and the presence of potentially moisture-sensitive near-surface soils. Recommendations regarding foundation setbacks from slopes and surficial remedial grading are provided in later sections of this report. Site drainage should be configured to provide positive drainage away from structures and to minimize saturation of near-surface soils.

Conventional shallow footings located adjacent to the drainage channel should be deepened such that a minimum horizontal distance of H/3 (where H is the total channel or slope height) or 7 feet, whichever is greater, is maintained between the lower outside footing edge and the face of the slope. Footings should also be deepened as necessary to prevent surcharge loading on the channel walls. Foundations should extend to a depth where the footing influence zone—defined as the area within a 1:1 (horizontal:vertical) line projected downward from the outside base of the footing—does not intercept or impose loads on the channel walls or slope. Given the close proximity of the proposed structure to the channel, the final foundation design in this area should be reviewed by MTGL prior to construction.

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Based on borehole percolation testing and the observed subsurface conditions, stormwater infiltration is not recommended. The site is classified as “No Infiltration” for BMP feasibility, and any BMP facilities should be lined and drained in accordance with our recommendations.

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4.00 RECOMMENDATIONS

The following recommendations are provided to address the geotechnical aspects of this project and are considered minimum. They may be superseded by more conservative requirements of the architect, structural engineer, building code, or governing agencies. In addition to the recommendations in this section, additional general earthwork and grading specifications are included in Appendix D.

4.01 EARTHWORK

4.01.1 SITE PREPARATION AND CLEARING

Site preparation should begin with the removal of surface vegetation, trash, debris, and existing structures or improvements. Abandoned underground improvements such as utility pipes and tanks should be removed from the site and capped or rerouted at the project perimeter. Resulting excavations should be backfilled and compacted in accordance with the recommendations provided in this report.

Removal of underground tanks is subject to state law as regulated by the County, City and/or Fire Department. If storage tanks containing hazardous or unknown substances are encountered, the proper authorities must be notified prior to any attempts at removing such objects. If water wells are encountered during construction, they should be exposed and capped in accordance with the requirements of the regulating agencies.

4.01.2 EXCAVATION CHARACTERISTICS

Based on the materials encountered during our investigation, it is anticipated that excavations can be achieved with conventional heavy duty earthwork equipment in good working order. Contract documents should specify that the contractor mobilize equipment capable of excavating and compacting strongly cemented materials.

4.01.3 REMOVALS AND OVER EXCAVATIONS

Recommendations for removals and over-excavation are provided for building areas supported on shallow foundations with ground improvement, deep foundations, minor structures supported on conventional foundations, and non-structural improvements. Structural plans and foundation elevations were not available at the time of our investigation. Once formal plans become available, MTGL should review them from a geotechnical standpoint and revise these recommendations, if necessary. For improvements not supported on piles or improved ground, additional settlement should be anticipated, as noted in Section 2.06.4, due to the potential for seismic settlement within the upper soil

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profile. The design team should incorporate appropriate settlement tolerances into the project documents.

4.01.3.1 CONVENTIONAL FOUNDATIONS ON GROUND IMPROVEMENTS

Compacted Aggregate Piers (CAPs) may be used to support the proposed structures and to reduce potential settlement and increase allowable bearing capacity. Where CAPs are utilized, all remedial grading and over-excavation shall be completed prior to installation of the CAPs. Existing undocumented fill should be removed in its entirety beneath the proposed buildings and other settlement-sensitive improvements to expose native materials.

Over-excavation should extend to a minimum depth of 2 feet below existing grade. The horizontal limits of over-excavation should extend a minimum of 5 feet beyond the perimeter of proposed structures or settlement-sensitive improvements, or up to existing improvements, whichever is less. Additional CAP-specific recommendations are presented in Section 4.01.4.

4.01.3.2 STRUCTURES SUPPORTED ON DEEP FOUNDATIONS

Existing soils below the proposed building should be over excavated to a minimum depth of 2 feet below the bottom of the slab, pile caps, or grade beams and replaced with compacted fill. The required horizontal limits of the over excavated area shall be defined as the area extending from the edge of the perimeter of the structure for a distance of 5 feet, or up to existing improvements.

Shoring and/or underpinning of any existing buildings or improvements within close proximity to the grading area may be required prior to performing any over-excavations or removals.

4.01.3.3 MINOR STRUCTURES SUPPORTED ON CONVENTIONAL FOUNDATIONS

Existing undocumented fills and underlying loose native soils should be excavated in beneath the minor structures such as trash enclosures and retaining walls. Over excavations should extend to a depth of 2 feet below existing ground surface or 2-feet below the bottom of footings, whichever is greater. Over excavations should extend laterally a minimum of 5 feet outside the planned structures or up to existing improvements, whichever is less. Additionally, the proposed structures should not be underlain by cut/fill transitions.

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4.01.3.4 NON-STRUCTURAL AREAS

Non-structural areas such as sidewalks, paved areas not subjected to vehicular loads, and other miscellaneous flatwork areas will require a minimum depth of 2 feet of removal and re-compaction below the lowest adjacent grade or bottom of bearing elevation. Excavation for hardscape areas should extend a minimum distance of 2 feet outside the hardscape limits.

The exposed soils beneath over-excavation and in cut areas not otherwise requiring over-excavation should be scarified to a minimum depth of 12 inches, moisture conditioned and compacted to a minimum of 90% relative compaction.

The above recommendations are based on the assumption that soils encountered during field exploration are representative of soils throughout the site. Removal and over-excavation depths must be verified, and adjusted if necessary, at the time of grading.

4.01.4 GROUND IMPROVEMENT - COMPACTED AGGREGATE PIERS

Compacted Aggregate Piers (CAPs) may be considered as a ground-improvement option to reduce the potential for static and seismic settlement and to increase the allowable bearing capacity for shallow foundations. The design and installation of CAP systems are typically proprietary to specialty design-build contractors; however, the construction process typically utilizes pre-augured or displacement methods to introduce compacted aggregate to the underlying soil strata. The augured or displaced soil cavities are backfilled with aggregate that is compacted using high-energy densification techniques, to create a column of well compacted material.

The ground-improvement design should be performed by a qualified specialty contractor experienced in the design and installation of CAP systems. The CAPs should extend to dense or very dense old alluvial fan deposit (Qof). On a preliminary basis, we recommend that CAPs extend to depths of approximately 20 feet below existing site grade and be spaced to provide a minimum area replacement ratio of 15 percent. The final layout and extent of CAP improvement should be coordinated with the design team to address the locations of settlement-sensitive structures and improvements.

Properly designed and installed CAPs may be capable of increasing allowable bearing pressures to up to 10,000 psf, depending on the contractor's design. At a minimum, the CAP system should satisfy the following performance criteria:

- Post-construction settlement of foundations supported on CAPs shall be less than 1 inch total settlement and ½ inch of differential settlement over 40 feet.

- When load tested to twice the design load, total settlement should be ≤ 1 inch. A minimum of two (2) test piers should be performed; additional testing may be required depending on the extent of improvement.

Because the subgrade modulus of improved ground varies with the proprietary design and installation methodology, preliminary modulus values should be provided by the design-build ground-improvement contractor. In-situ field testing must be performed during construction to validate the initial design values.

4.01.5 FILL MATERIALS

Removed and/or over-excavated soils, except for roots, debris, and rocks greater than 6 inches, may be used as compacted fill. Concrete slabs should be underlain by at least 2 feet of material with an expansion index of 50 or less as determined by ASTM D4829. Based on our field investigation, we expect that most of the near surface onsite materials will meet the expansion index criteria.

Prior to placing fill, exposed surfaces at the bottom of the excavations should be scarified to a depth of 12 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. Fill should be placed in horizontal lifts at a thickness appropriate for the equipment spreading, mixing, and compacting the material, but generally should not exceed 8 inches in loose thickness. Fill should be moisture conditioned to near optimum moisture content and compacted to at least 90% relative compaction. Fill should be benched into sloping ground inclined steeper than 5:1 (horizontal to vertical). The maximum dry density and optimum moisture content for evaluating relative compaction should be determined in accordance with ASTM D1557.

Utility trench backfill beneath structures, pavements and hardscape should be compacted to at least 90% relative compaction. The upper 12 inches of subgrade beneath pavements should be compacted to at least 95%.

4.01.6 EXPANSIVE SOILS

Laboratory test results indicate the on-site soils have an expansion index of 29, classified as a low expansion potential. To reduce the potential for expansive heave, soils with an expansion index greater than 50 should be removed in the upper 2 feet of the planned structure or any hardscape areas. Granular material having an expansion index of 50 or less should be used as replacement fill.

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4.01.7 IMPORTED SOILS

Imported soil should consist of predominately granular soil, free of organic matter and rocks greater than 4 inches. Imported soil should have an expansion index of 20 or less and should be inspected and, if appropriate, tested prior to transport to the site.

4.01.8 OVERSIZED MATERIALS

Excavations may generate oversized material. Oversized material is defined as rocks or cemented clasts greater than 6 inches in largest dimension. Oversized material should be broken down to no greater than 6 inches in largest dimension for use in engineered fill, used as landscape material, or disposed of off-site.

4.01.9 TEMPORARY EXCAVATIONS

Temporary slopes and excavations should be made in conformance with applicable OSHA standards and requirements. Based on the results of our investigation, the subsurface materials can preliminarily be categorized as Type C soil. Accordingly, temporary excavations in fill and alluvium should be laid back no steeper than 1½:1 (horizontal:vertical), up to a maximum depth of 20 feet. The faces of temporary slopes should be inspected daily by the Contractor's Competent Person before personnel are allowed to enter the excavation. Zones of potential instability, sloughing, or raveling should be brought to the attention of the engineer and corrective action implemented before personnel begin working in the excavation. Excavated soils should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. MTGL should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of slopes to prevent runoff water from entering the excavation and eroding the slope faces.

Slopes steeper than those described above will require shoring. Additionally, temporary excavations that extend below a plane inclined at 1½:1 (horizontal:vertical) downward from the outside bottom edge of existing structures or improvements will require shoring. Soldier piles and lagging, internally braced shoring or trench boxes could be used. If trench boxes are used, the soil immediately adjacent to the trench box is not directly supported. Ground surface deformations immediately adjacent to the pit or trench could be greater where trench boxes are used compared to other methods of shoring.

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4.01.10 TEMPORARY SHORING

For design of cantilevered shoring, an active soil pressure equal to a fluid weighing 40 pcf can be used for level retained ground or 65 pcf for 2:1 (horizontal:vertical) sloping ground. The surcharge loads on shoring from traffic and construction equipment adjacent to the excavation can be modeled by assuming an additional 2 feet of soil behind the shoring.

For design of soldier piles, an allowable passive pressure of 350 psf per foot of embedment over 2.5 times the pile diameter or the spacing of the piles, whichever is less, up to a maximum of 4,500 psf can be used for soil above the groundwater level. An allowable passive pressure of 175 psf per foot of embedment over 2.5 times the pile diameter or the spacing of the piles, whichever is less, up to a maximum of 2,250 psf can be used for soil below the groundwater level. Hydrostatic pressure should be applied below the groundwater level.

Soldier piles should be spaced at least three pile diameters, center to center. Continuous lagging will be required throughout. The soldier piles should be designed for the full-anticipated lateral pressure; however, the pressure on the lagging will be less due to arching in the soils. For design of lagging, the earth pressure but can be limited to a maximum value of 400 psf.

Installation of soldier piles below groundwater (or dewatered soil) will require special construction techniques and equipment, such as temporary casing and/or drilling slurry to cope with groundwater and potential heavy caving. Other installation methods may be available. If soldier piles are anticipated to extend to a depth below approximately 25 feet (approximate groundwater depth), contract documents should specify that the contractor mobilize equipment capable of installing piles below groundwater (or dewatered soil) in order to reduce the potential for claims of delays or extra work to occur.

Piles should be filled with concrete immediately after drilling. The concrete should be pumped to the bottom of the drilled holes using the tremie method. If casing is used, the casing should be removed as the concrete is placed, keeping the level of the concrete at least 5 feet above the bottom of the casing at all times.

4.01.11 SLOPES

All permanent slopes should be constructed no steeper than 2:1 (horizontal: vertical). Faces of fill slopes should be compacted either by rolling with a sheepsfoot roller or other suitable equipment or by overfilling and cutting back to design grade. Fills should be benched into sloping ground when inclined steeper than 5:1 (horizontal: vertical). An engineering

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geologist should observe all cut slopes during grading to ascertain that no unforeseen adverse geologic conditions are encountered that require revised recommendations.

Where buildings are planned near the top of a slope steeper than 3:1 (horizontal to vertical), special foundations or stabilization measures are recommended. Footings located adjacent to or within existing slopes should be deepened to a depth such that a minimum horizontal distance of H/3 (where H is the height of the slope) or 7 feet, whichever is greater, exists between the lower outside footing edge and the face of the slope. Due to the close proximity of the proposed structure to the existing drainage channel slope walls, the foundation design should be reviewed by MTGL.

All slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the top of slope. Additionally, slopes should be planted with vegetation that will reduce the potential for erosion.

4.01.12 TEMPORARY DEWATERING

Groundwater seepage may occur locally due to local irrigation or following heavy rain. Temporary dewatering can be accomplished by sloping the excavation bottom to a sump and pumping from the sump. A layer of gravel about 6 inches thick placed in the bottom of the excavation will facilitate groundwater flow and can be used as a working platform.

A specialty dewatering contractor should be contacted for additional project specific dewatering recommendations.

4.02 FOUNDATIONS

Based on the subsurface conditions encountered at the site and the potential for significant seismic settlement within the upper soil profile, the proposed building and other settlement sensitive structures be supported on deep foundations or shallow foundations underlain by ground improved with Compacted Aggregate Piers (CAPs).

Our recommendations are only minimum criteria based on geotechnical factors and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or by the structural engineer. The foundation system should be designed by the project's structural engineer, incorporating the geotechnical parameters described herein and the requirements of applicable building codes.

The foundation recommendations provided herein are considered generally consistent with methods typically used in southern California, however, other alternatives may be available. Based

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on the results of our geotechnical investigation, recommendations for various foundation systems are presented in the following sections.

4.02.1 CONVENTIONAL SHALLOW FOUNDATIONS WITH GROUND IMPROVEMENT

The planned structures may be supported on conventional shallow spread and/or continuous footings bearing entirely on ground improved with Compacted Aggregate Piers (CAPs). A preliminary allowable bearing pressure of 4,000 psf may be used for design; however, the final allowable bearing pressure should be confirmed by the specialty ground-improvement contractor based on the selected CAP system and layout.

The recommended minimum footing width and embedment depth below the lowest adjacent grade are as follows:

CONVENTIONAL SHALLOW FOUNDATIONS

Foundation Type	Minimum Width	Minimum Depth
Continuous (Interior)	12 inches	18 inches
Continuous (Perimeter)	12 inches	24 inches
Spread Footings	24 inches	24 inches

Lateral loads will be resisted by friction between the bottoms of footings and passive pressure on the faces of footings and other structural elements below grade. An allowable coefficient of friction of 0.35 can be used. Passive pressure can be computed using a lateral pressure of 350 psf per foot of depth below the ground surface for level ground conditions. When combining passive pressure and friction for lateral resistance, the passive pressure component should be reduced by one-third.

The passive pressure can be increased by one-third when considering the total of all loads, including wind or seismic forces. The upper 1-foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

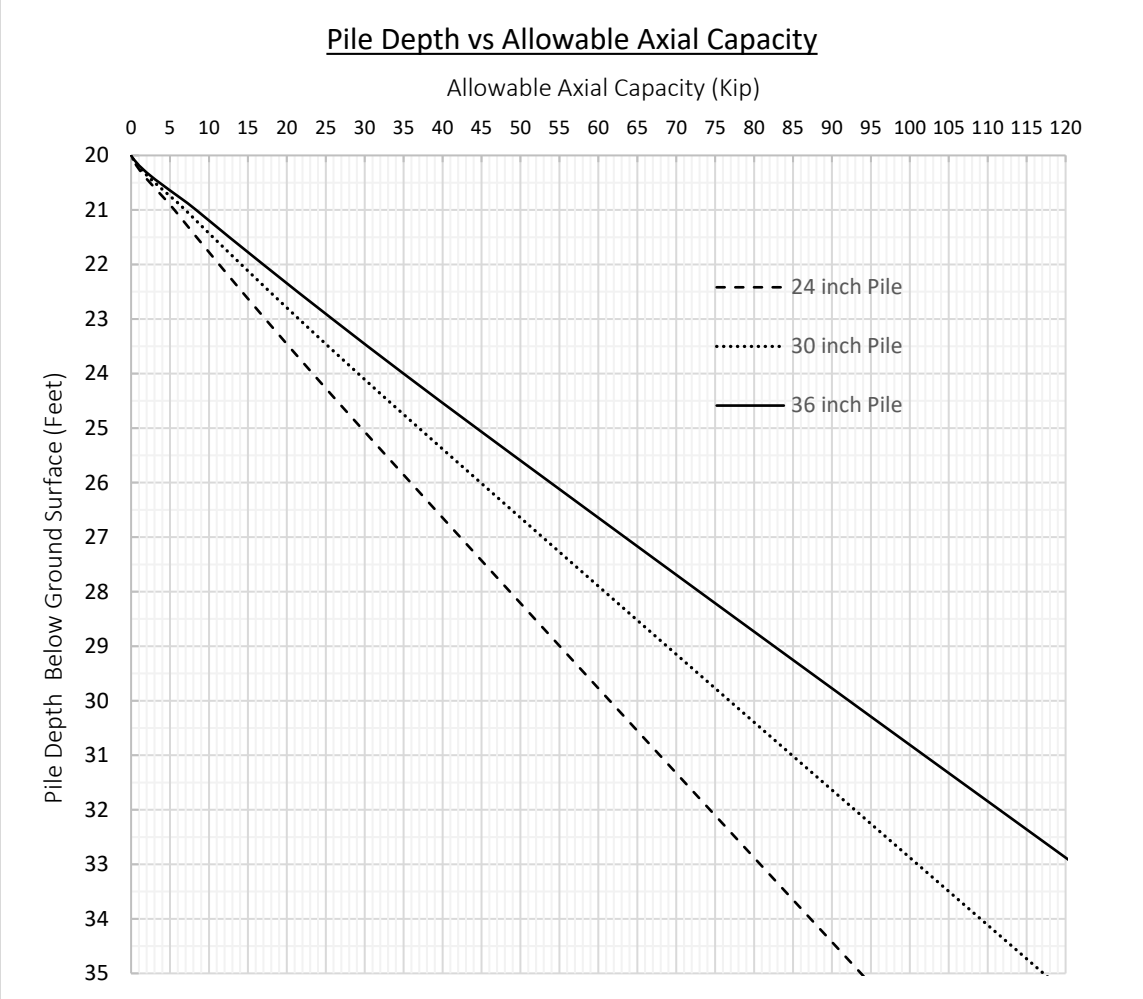
4.02.2 CAST IN DRILLED HOLE (CIDH) DEEP FOUNDATIONS

As an alternative to ground improvement, drilled cast-in-place concrete (CIDH) piles may be used to support the proposed structures. Considering the potential for seismic settlement of the upper fill (Qf) and young axial channel deposits (Qya), positive shaft resistance within the upper 20 feet should be neglected for axial capacity considerations. Pile support should instead be derived from skin friction within the underlying dense to very dense older alluvial materials (Qof). We recommend that piles extend a minimum depth of 10 feet into

competent older alluvium, which corresponds to a pile tip elevation of approximately 30 feet below existing ground surface.

Preliminary axial capacities for 24-inch, 30-inch, and 36-inch diameter piles were evaluated using the computer program SHAFT. The recommended allowable axial capacities presented in the chart below are based on geotechnical resistance only and incorporate a factor of safety of 2.5.

AXIAL CAPACITIES OF PILES (ALLOWABLE)



The capacities above represent allowable downward geotechnical capacities. The structural engineer should confirm project-specific safety factors and adjust them as necessary based on the anticipated load combinations and/or design methodology. Uplift resistance may be taken as the sum of the pile weight plus two-thirds of the allowable downward frictional capacity. The pile capacities are based on the estimated strength of the soils; the strength of

the pile section itself should be checked to verify the structural capacity of the pile. Piles should be spaced at least three (3) pile diameters, center to center.

Due to the potential for significant seismic settlement within the upper 20 feet, negative skin friction (downdrag) should be applied over this depth interval. The following table presents the estimated downdrag forces per linear foot of pile length for 24-inch, 30-inch, and 36-inch diameter piles.

Pile Diameter (inches)	Downdrag (Negative skin friction), kip/foot
24	0.86
30	1.07
36	1.29

The structural design loads should include these downdrag forces in combination with the structural service loads critically at depths of 20 feet and greater.

If desired, downdrag forces may be reduced by installing a friction-reducing casing or slip sleeve (e.g., smooth steel casing, HDPE sleeve, or other proprietary debonding system) along the upper 20 feet of the pile. These systems generally reduce soil-pile adhesion, allowing the surrounding soils to settle relative to the pile without mobilizing significant negative skin friction along the pile interface. If used, the casing should be provided by a specialty manufacturer and installed by an experienced specialty contractor.

Due to the relatively loose soil conditions within the upper 15 to 20 feet, temporary caving of the pile shafts should be anticipated. Special care should be taken during drilling to stabilize the sides of the CIDH piles, including the use of temporary casing or slurry methods, as required by field conditions.

Foundation plans or building layouts and types are not available at this time. However, based on the anticipated load ranges and the dense nature of bearing soils, we estimate total post-construction pile settlements will be less than approximately 1 inch. Differential settlement between adjacent piles will be dependent on the final design and foundation layout, although we anticipate that differential settlement will be less than approximately ½ inch between piles. Once foundation plans are available, MTGL should review the plans and revise these recommendations, as necessary.

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Lateral Load Resistance (To Be Finalized)

Lateral load recommendations will be developed once the project lateral design loads, pile head fixity conditions, and foundation layouts are provided by the structural engineer. After receiving this information, MTGL can perform a lateral pile analysis (e.g., using L-PILE) and provide preliminary lateral capacities, load-deflection behavior, and bending moment profiles for the selected pile diameters.

When laterally loaded drilled shafts are used in closely spaced groups, a given shaft will deflect further under a given system of loads than if loaded when the neighboring shafts are not present and bending stresses will increase beyond those that occur when neighboring shafts are not present. It is therefore important to consider group effects due to lateral loading when shaft spacing is less than approximately six diameters in any direction. Recommended group effect factors are as follows.

GROUP EFFECT FACTORS FOR LATERALLY LOADED PILES

Pile Spacing (Center to Center)	3D	4D	5D	≥6D
Lead Row	0.7	0.85	1.0	1.0
2nd Row	0.5	0.65	0.85	1.0
3rd and Higher Row	0.35	0.5	0.7	1.0

Piles should be designed with sufficient steel to adequately resist lateral loads and moments that develop in the pile. A preconstruction test pile and pile load test program may be required by the structural engineer to verify the geotechnical and structural capacities, based on the contractor's construction means and methods.

For lateral resistance of pile caps, an allowable passive pressure of 350 psf per foot of depth may be used. The passive pressure can be increased by 1/3 when considering the total of loads, including wind or seismic forces. The upper 1-foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs. For sliding resistance, a friction coefficient of 0.35 may be used at the concrete and soil interface.

The actual required depths must be field verified by the geotechnical engineer. The adequacy of the CIDH piles will depend heavily on construction methods and procedures. Large zones of disturbance around the CIDH piles can lead to lower skin friction due to excessive stress relief around the length of the piles. The piles should be constructed by qualified contractors experienced in this type of construction and monitored on a full-time basis by the geotechnical consultant.

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Contract documents should specify that the contractor mobilize equipment capable of penetrating hard, cemented material to reduce the potential that claims for delays or extra work will arise.

Since a portion of the piles may be constructed below the groundwater table and in granular soils, caving of the drilled holes should be anticipated. A slurry and/or temporary casing may be required to stabilize the sides of the CIDH pile shafts. The CIDH piles should be placed using a down-hole tremie to limit the concrete from striking the sides of the drilled shafts. Once concrete pumping has begun, a minimum head of 5 feet of concrete above the bottom of the tremie should be maintained throughout the concrete placement. If a steel casing is used, the casing must be removed slowly with the minimum concrete head maintained to prevent soils caving and necking of the pile. The concrete mix must be capable of disseminating around the reinforcing bars and in contact with the soils without arching during extraction of the casing.

4.02.3 SETTLEMENT CONSIDERATIONS

Foundations should be designed for the anticipated settlements. Static settlement of an individual foundation member will vary depending on the plan dimensions of the foundation and the actual load supported.

We estimate maximum static settlement of foundations designed and constructed in accordance with the recommendations presented to be on the order of 1 inch. Differential settlement between similarly loaded and adjacent footings are expected to be less than ½ inch across 40 feet, provided footings are founded on similar materials. Static settlement of all foundations is expected to occur rapidly and should be essentially complete shortly after initial application of the loads.

4.03 INTERIOR SLABS ON GRADE

The project's structural engineer should design concrete slabs-on-grades for buildings. However, it is recommended that interior slabs be at least 5 inches thick and reinforced with at least No. 4 bars at 18 inches on center each way. The concrete should have a 28-day compressive strength of at least 3,000 psi, a water to cement ratio of 0.50 or less, and a slump of 4 inches or less.

Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. The project's architect should select the final joint patterns. A 1-inch maximum size aggregate mix is recommended for concrete slabs. The corrosion potential of on-site soils with respect to reinforced concrete will need to be taken into account in

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concrete mix design. Coarse and fine aggregate in concrete should conform to the "Greenbook" Standard Specifications for Public Works Construction.

Moisture protection should be installed beneath slabs where moisture sensitive floor coverings will be used. The project's architect should review the tolerable moisture transmission rate of the proposed floor covering and specify an appropriate moisture protection system. Typically, a plastic vapor barrier (15-mil Stegowrap or equivalent) is used and should comply with ASTM E1745. The vapor barrier installation should comply with ASTM E1643 and ACI 302.1R. The floor covering manufacturer should be consulted to determine the volume of moisture vapor allowable and treatment needed to reduce moisture vapor emissions to acceptable limits for the particular type of floor covering installed.

4.04 HARDSCAPE

Hardscape and other exterior concrete slabs-on-grade not subjected to vehicular loads should be underlain by at least 2 feet of material with an expansion index of 50 or less. Exterior slabs should be at least 4 inches thick and reinforced with at least No. 3 bars at 18 inches on center each way. Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. The project's architect should select the final joint patterns. A 1 inch maximum size aggregate mix is recommended for concrete for exterior slabs. The corrosion potential of on-site soils with respect to reinforced concrete will need to be taken into account in concrete mix design. Coarse and fine aggregate in concrete should conform to the "Greenbook" Standard Specifications for Public Works Construction.

4.05 PREWETTING RECOMMENDATIONS

Prior to placing concrete slabs and flatwork, the underlying soils should be brought to within a minimum of 2% and a maximum of 4% above its optimum moisture content for a depth of 12 inches prior to the placement of concrete. The geotechnical consultant should perform in-situ moisture tests to verify that the appropriate moisture content has been achieved a maximum of 24 hours prior to the placement of concrete or moisture barriers.

Once the slab subgrade soil has been pre-wetted and compacted, the soil should not be allowed to dry prior to concrete placement. If the subgrade soil is dry, the moisture content of the soil should be restored prior to placement of concrete and re-tested.

Proper moisture conditioning and compaction of subgrade soils should be performed prior to placement of concrete. Even with proper site preparation, some soil moisture changes of the subgrade soils supporting the concrete flatwork due to edge effects (shrink/swell) may occur. Drying and/or wetting of subgrade soils adjacent to landscaped areas or open fields may increase the

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potential of shrink/swell effects beneath concrete flatwork areas. To help reduce edge effects, lateral cutoffs, such as inverted curbs are recommended. Control joints should be used to reduce the potential for flatwork panel cracks as a result of minor soil shrink/swell.

The recommendations are intended to reduce the potential for cracking of slabs; however, even with the incorporation of the recommendations presented herein, slabs may still exhibit some cracking. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics.

4.06 CORROSIVITY

Soluble sulfate tests indicate that concrete at the subject site will have a negligible to moderate exposure to water soluble sulfate in the soil. We recommend that the concrete be designed to resist a moderate exposure category (Class S1). Our recommendations for concrete exposed to sulfate-containing soils are presented below.

RECOMMENDATIONS FOR CONCRETE EXPOSED TO SULFATE CONTAINING SOILS

Sulfate Exposure Severity	Class	Water soluble sulfate (SO_4) in soil (% by wgt)	Sulfate (SO_4) in water (ppm)	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)	Cement Type	Calcium Chloride Admixture
Negligible	S0	0.00 - 0.10	0-150	---	2,500	---	No Restriction
Moderate	S1	0.10 - 0.20	150-1,500	0.50	4,000	II/V	No Restriction
Severe	S2	0.20 - 2.00	1,500-10,000	0.45	4,500	V	Not Permitted
Very Severe	S3	Over 2.00	Over 10,000	0.45	4,500	V Plus Pozzolan	Not Permitted

Corrosivity testing consisting of soils reactivity (pH) and resistivity (ohms-cm) were also tested on select soil samples. The test results indicate that the soils have a soil reactivity of 7.6 and a resistivity of 5,400 ohms-cm. A neutral or non-corrosive soil has a reactivity value ranging from 5.5 to 8.4. Generally, soils that could be considered corrosive to metal have resistivities less than 3,000 ohms-cm. Those soils with resistivity values of less than 1000 ohms-cm can be considered extremely corrosive.

Based on our test results, it is our opinion that the underlying soils at the site have a low corrosion potential. Protection of buried pipes utilizing coatings on all underground pipes; clean backfills and a cathodic protection system can be effective in controlling corrosion. A qualified corrosion engineer should be consulted to further assess the corrosive properties of the soil and provide mitigation measures appropriate to the improvements, if necessary.

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4.07 RETAINING STRUCTURES

Embedded structural walls should be designed for lateral earth pressures exerted on the walls. The magnitude of these earth pressures will depend on the amount of deformation that the wall can yield under the load. If the wall can yield sufficiently to mobilize the full shear strength of the soils, it may be designed for the active condition. If the wall cannot yield under the applied load, then the shear strength of the soil cannot be mobilized, and the earth pressures will be higher. These walls such as basement walls and swimming pools should be designed for the at rest condition. If a structure moves towards the retained soils, the resulting resistance developed by the soil will be the passive resistance.

For design purposes, the recommended equivalent fluid pressure for each case for walls constructed above the static groundwater table and backfilled with granular, well-draining, non-expansive soils is provided below. Retaining wall backfill should be compacted to at least 90% relative compaction based on the maximum density defined by ASTM D1557.
Retaining structures should be designed to resist the following lateral earth pressures.

- Passive Earth Pressure – Same as for shallow foundations.
- At rest lateral earth pressure - 60 pcf.
- Active Earth Pressures (Equivalent Fluid Weights):

Slope of Retained Material	Equivalent Fluid Weight (pcf)
Level	40
2:1 (H:V)	65

It is recommended that retaining wall footings be embedded at least 24 inches below the lowest adjacent finish grade. In addition, the wall footings should be designed and reinforced as required for structural considerations. The wall areas should be over excavated to a minimum depth of 2 feet below the bottom of the proposed footings. The required horizontal limits of the over excavation area shall be a minimum distance of 2 feet.

Lateral resistance parameters provided above are ultimate values. Therefore, a suitable factor of safety should be applied to these values for design purposes. The appropriate factor of safety will depend on the design condition and should be determined by the project's structural engineer. These parameters do not include loading from adjacent structures. If any super-imposed loads are anticipated, this office should be notified so that appropriate recommendations for earth pressures may be provided.

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Retaining structures should be designed with effective drainage to prevent the accumulation of subsurface water behind the walls. Backdrains should be installed behind retaining walls exceeding 3 feet in height. Backdrains may consist of a 2-foot wide zone of ¾ inch crushed rock. The backdrain should be separated from the adjacent soils using a non-woven filter fabric, such as Mirafi 140N or equivalent. Weep holes should be provided, or a perforated pipe should be installed at the base of the back-drain and sloped to discharge to a suitable storm drain facility. As an alternative, a geo-composite drainage system such as Mira-drain 6000 or equivalent placed behind the wall and connected to a suitable storm drain facility can be used. The project's architect should provide waterproofing specifications and details. A typical detail for retaining wall backdrains is presented as Figure 8. Backdrains should outlet to suitable drainage devices.

4.08 SEISMIC EARTH PRESSURES

If required, the seismic earth pressure can be taken as equivalent to the pressure of a fluid weighing 23 pcf. This value is for level backfill and does not include a factor of safety. Appropriate factors of safety should be incorporated into the design. This pressure is in addition to the un-factored, static active earth pressure. The passive pressure and bearing capacity can be increased by ⅓ in determining the seismic stability of the wall.

4.09 PAVEMENT STRUCTURAL SECTIONS

Recommended pavement structural sections were developed using the procedures outlined in the Design Procedures for Flexible Pavements in the California Department of Transportation (Caltrans) Highway Design Manual (HDM). This method is based on the principle that the pavement section must be sufficiently thick to distribute the design traffic loading (Traffic Index, TI) such that the resulting stresses imposed on the subgrade do not exceed its support capacity, represented by the R-Value.

Given the predominately clayey subgrade soils encountered at the site, a design R-Value of 10 was selected for pavement thickness calculations. The recommended pavement structural sections for the assumed TI levels are presented below

ASPHALT PAVEMENT STRUCTURAL SECTIONS

Pavement Area	Traffic Index	Asphalt Thickness (inches)	Base Thickness (inches)
Parking Areas	5.0	3	10
Driveways	6.0	4	12
Fire Access Lanes	7.0	4	14

The upper 12 inches of subgrade should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction. Soft or yielding areas should be removed and replaced with compacted fill or aggregate base. Aggregate base and asphalt concrete should conform to the Caltrans Standard Specifications or the "Greenbook" and should be compacted to at least 95% relative compaction. Aggregate base should have an R-value of not less than 78. Materials and methods of construction should conform to good engineering practices.

4.10 UTILITY TRENCHES

Cal/OSHA construction safety orders should be observed during all underground work. Utility trench backfill within street right of way, utility easements, under or adjacent to sidewalks, driveways, or building pads should be observed and tested by the geotechnical consultant to verify proper compaction. Utility trenches which parallel a footing and extend below a 1:1 plane projected from the outside edge of the footing should be backfilled with structural fill soils and compacted to at least 90% of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches. Trenches crossing perpendicular to foundations should be excavated and backfilled prior to the construction of the foundations. The excavations should be backfilled in the presence of the geotechnical engineer and tested to verify adequate compaction beneath the proposed footing.

4.10.1 THRUST BLOCKS

For level ground conditions, a passive earth pressure of 350 psf per foot of depth below the lowest adjacent final grade can be used to compute allowable thrust block resistance. A value of 175 psf per foot should be used below groundwater level, if encountered.

4.10.2 PIPELINE SUPPORT

It is anticipated that most of the materials along the pipeline alignments will provide adequate support for the pipes, although loose, soft, wet, and otherwise unsuitable materials may be encountered locally. Unsuitable materials encountered near trench bottom levels should be evaluated during construction by the geotechnical engineer. Unsuitable materials should be removed from the full width of the trench. The bottoms of the excavations should be observed by the geotechnical consultant prior to placement of pipe bedding. The use of a stabilizing fabric such as Mirafi® HP 570 can be used to stabilize the bottom of the excavations, if needed.

4.10.3 MODULUS OF SOIL REACTION

A modulus of soil reaction (E') of 2,000 psi can be used to evaluate the deflection of buried flexible pipelines. This value assumes that granular bedding material is placed adjacent to the pipe and is compacted to at least 90% relative compaction.

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4.10.4 BEDDING

Pipe bedding as specified in the *Standard Specifications for Public Works Construction* ("Greenbook") may be used. Bedding material should consist of clean sand with a sand equivalent (SE) not less than 30 and should extend to at least 12 inches above the top of pipe. Alternative materials meeting the intent of the Greenbook bedding requirements are also acceptable. Samples of materials proposed for use as bedding should be submitted to the engineer for review and approval prior to import. The on-site soils are not expected to meet the Greenbook bedding specifications.

The pipe bedding material should be placed over the full width of the trench, and after the pipe is set, the bedding should be brought up uniformly on both sides to minimize unbalanced loading. No voids or uncompacted zones should remain beneath the pipe haunches. Ponding or jetting of bedding materials should not be permitted.

If gap-graded or open-graded bedding materials (such as drain rock or crushed rock) are used, they should be fully encased in an appropriate geotextile fabric to prevent migration of fines into the bedding zone and to maintain long-term support of the pipe.

4.10.5 BACKFILL

Excavated material free of organic debris and rocks greater than 6 inches in any dimension are generally expected to be suitable for use as backfill. Imported material should not contain rocks greater than 4 inches in any dimension or organic debris. Imported material should have an expansion index of 20 or less. MTGL should observe and, if appropriate, test proposed imported materials before they are delivered to the site. Backfill should be placed in lifts 8 inches or less in loose thickness, moisture conditioned to optimum moisture content or slightly above, and compacted to at least 90% relative compaction. The upper 12 inches of soil beneath pavement subgrade should be compacted to at least 95% relative compaction. Backfill should not be placed on overly wet, soft, or pumping subgrade until such conditions are corrected.

4.11 INFILTRATION FEASIBILITY

MTGL conducted an infiltration feasibility study at the site on May 25–26, 2024. At the time of field exploration, no preliminary BMP layout or stormwater design exhibits were available for review, and the project remained in the planning phase. Our fieldwork consisted of converting two shallow geotechnical borings into percolation test holes. The test borings were prepared and tested by an MTGL engineer in general accordance with the Riverside County Low Impact Development BMP Design Handbook (County of Riverside, 2011). The percolation data and calculations are presented in Appendix E. A summary of the test results is provided in the following table.

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INFILTRATION RATE TEST RESULTS

Test Location	Test Depth (Feet)	Material Type at Test Depth (USCS Classification)	Adjusted Infiltration Rate* (Inches/Hour)
P-1	5	**Young Axial Channel Deposits (Qya)**: Clayey Sand (SC)	0.58
P-2	10	**Young Axial Channel Deposits (Qya)**: Clayey Sand (SC)	0.07

***The adjusted infiltration rate is calculated by applying a factor of safety of 2 to the tested infiltration rate.**

The test results indicate variable and generally poor infiltration characteristics, consistent with the presence of loose, unconsolidated near-surface deposits underlain by finer-grained soils with limited permeability. The tested layers are not laterally uniform, and infiltration at these depths may exceed the permeability of the underlying low-permeability materials, creating a potential for perched groundwater conditions at or near foundation bearing elevations. These conditions could result in localized saturation, softening, and loss of soil strength within the proposed building footprint.

Based on these conditions, stormwater infiltration is not recommended at this site. The feasibility screening category for infiltration BMPs is therefore classified as "No Infiltration." BMP facilities should be lined with an impermeable geomembrane to limit infiltration into the surrounding soils. A subdrain should be placed at the bottom of lined BMPs to collect and convey seepage, preventing the buildup of hydrostatic pressures or localized ponding. Foundations should be set back at least 10 feet from BMP facilities or constructed to extend below the bottom elevation of the BMPs to reduce the potential for water-related distress to adjacent structures and improvements.

4.12 CONSTRUCTION CONSIDERATIONS

4.12.1 MOISTURE-SENSITIVE SOILS AND WEATHER-RELATED CONCERNS

The upper soils encountered at this site may be sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and its support capabilities. In addition, soils that become excessively wet may be slow to dry and therefore significantly delay the progress of the grading operations. Therefore, it will be advantageous to perform earthwork and foundation construction activities during the dry season. Much of the on-site soils may be susceptible to erosion during periods of inclement weather. As a result, the project's Civil Engineer/Architect and Grading Contractor should take appropriate precautions to reduce the potential for erosion during and after construction.

4.12.2 DRAINAGE AND GROUNDWATER CONSIDERATIONS

Based on our investigation, groundwater is expected to be below the anticipated depths of grading and the installation of subdrains is not expected to be necessary. However,

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variations in the ground water table may result from fluctuation in the ground surface topography, subsurface stratification, precipitation, irrigation, and other factors such as impermeable and/or cemented formational materials overlain by fill soils. In addition, during retaining wall excavations, seepage may be encountered. Therefore, we recommend that a representative of MTGL, Inc. be present during grading operations to evaluate areas of seepage. Drainage devices for reduction of water accumulation can be recommended should these conditions occur.

Water should not be allowed to collect in the foundation excavation, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped to facilitate removal of any collected rainwater, groundwater, or surface runoff. Positive site drainage should be provided to reduce infiltration of surface water around the perimeter of the structure and beneath the floor slabs. The grades should be sloped away from the structure and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas.

4.12.3 SITE DRAINAGE

The site should be designed to provide positive drainage away from all structures in accordance with applicable building codes and local jurisdictional requirements. From a geotechnical standpoint, maintaining proper drainage is essential to prevent saturation, softening, and loss of support in the near-surface soils.

Unpaved areas should be graded to slope at least 2 percent away from structures. Paved areas should slope not less than 1 percent away from structures. Final grading should avoid creating low spots or areas where surface water may pond adjacent to foundations, slabs, retaining walls, or other settlement-sensitive improvements.

Concentrated roof runoff and surface drainage should be collected in engineered, non-erosive drainage devices (such as downspout extensions, splash blocks, area drains, swales, or piped systems) and conveyed to a safe point of discharge. Drainage should not be allowed to discharge uncontrolled onto slopes or across unprotected soils. Standing water and uncontrolled infiltration near the building footprint should be avoided, as these conditions can lead to softening of the upper soil profile, differential settlement, and distress to pavements and flatwork.

Where lined BMPs or stormwater management features are used, they should be located and detailed to prevent inadvertent infiltration into supporting soils, consistent with the

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recommendations presented in this report. The overall site drainage system should be designed by the project civil engineer, and should incorporate these geotechnical considerations to maintain the long-term performance of site improvements.

4.13 PLAN REVIEW

MTGL should review the grading and foundation plans to verify that the intent of the recommendations presented in this report has been implemented and that revised recommendations are not necessary as a result of design changes after the date of this report.

5.00 GEOTECHNICAL OBSERVATION AND TESTING

The recommendations provided in this report are based on preliminary design information and subsurface conditions as interpreted from the investigation. Our preliminary conclusions and recommendations should be reviewed and verified during site grading and revised accordingly if exposed geotechnical conditions vary from our preliminary findings and interpretations. The geotechnical consultant should perform geotechnical observation and testing during the following phases of grading and construction:

- During site grading and over-excavation.
- During foundation excavations and placement.
- Upon completion of retaining wall footing excavation prior to placing concrete.
- During installation of geotechnical related drain elements.
- During excavation and backfilling of public utility trenches; of private utility trenches, as directed.
- During processing and compaction of the subgrade for the access and parking areas and prior to construction of pavement sections.
- When any unusual or unexpected geotechnical conditions are encountered during any phase of construction.

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6.00 LIMITATIONS

The findings, conclusions, and recommendations contained in this report are based on the site conditions as they existed at the time of our investigation, and further assume that the subsurface conditions encountered during our investigation are representative of conditions throughout the site. Should subsurface conditions be encountered during construction that are different from those described in this report, this office should be notified immediately so that our recommendations may be re-evaluated.

This report was prepared for the exclusive use and benefit of the owner, architect, and engineer for evaluating the design of the facilities as it relates to geotechnical aspects. It should be made available to prospective contractors for information on factual data only, and not as a warranty of subsurface conditions included in this report.

Our investigation was performed using the standard of care and level of skill ordinarily exercised under similar circumstances by reputable soil engineers and geologists currently practicing in this or similar localities. No warranty, express or implied, is made as to the conclusions and professional advice included in this report.

This firm does not practice or consult in the field of safety engineering. We do not direct the Contractor's operations, and we are not responsible for their actions. The contractor will be solely and completely responsible for working conditions on the job site, including the safety of all persons and property during performance of the work. This responsibility will apply continuously and will not be limited to our normal hours of operation.

The findings of this report are considered valid as of the present date. However, changes in the conditions of a site can occur with the passage of time, whether they are due to natural events or to human activities on this or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge.

Accordingly, this report may become invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

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FIGURES

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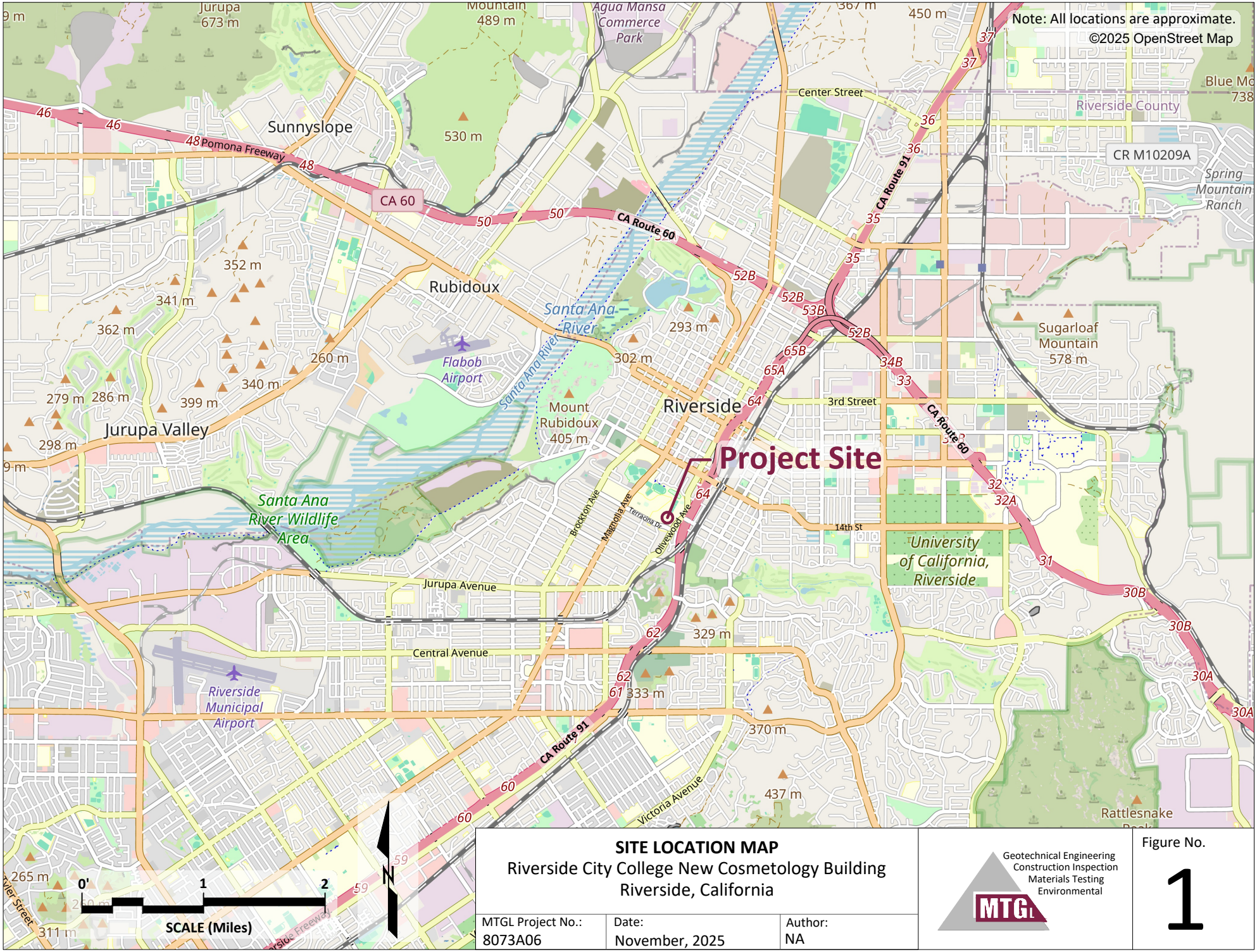


SITE LOCATION MAP
Riverside City College New Cosmetology Building
Riverside, California

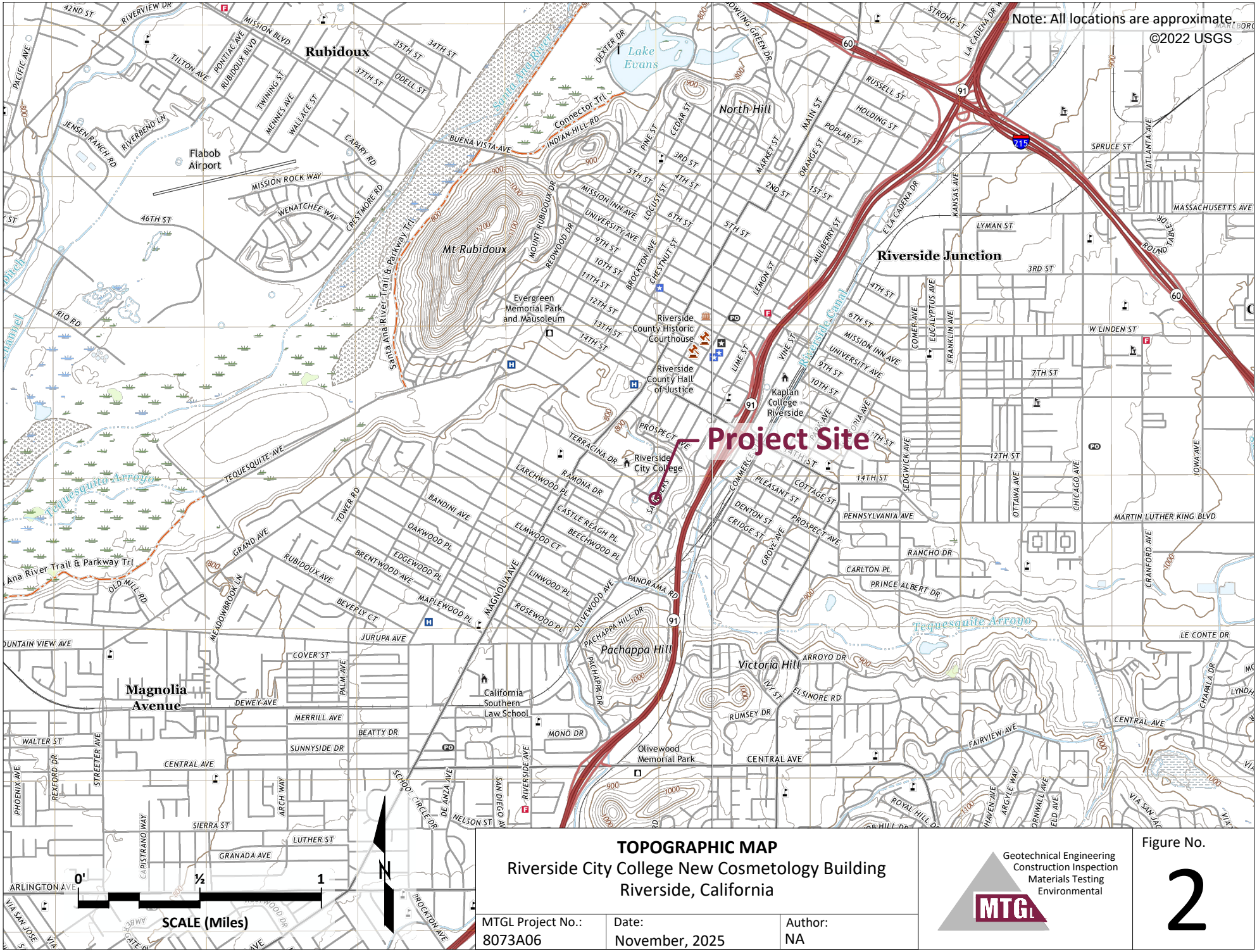
MTGL Project No.:	Date:	Author:
8073A06	November, 2025	NA

Geotechnical Engineering
Construction Inspection
Materials Testing
Environmental

MTGL

Figure No.

1

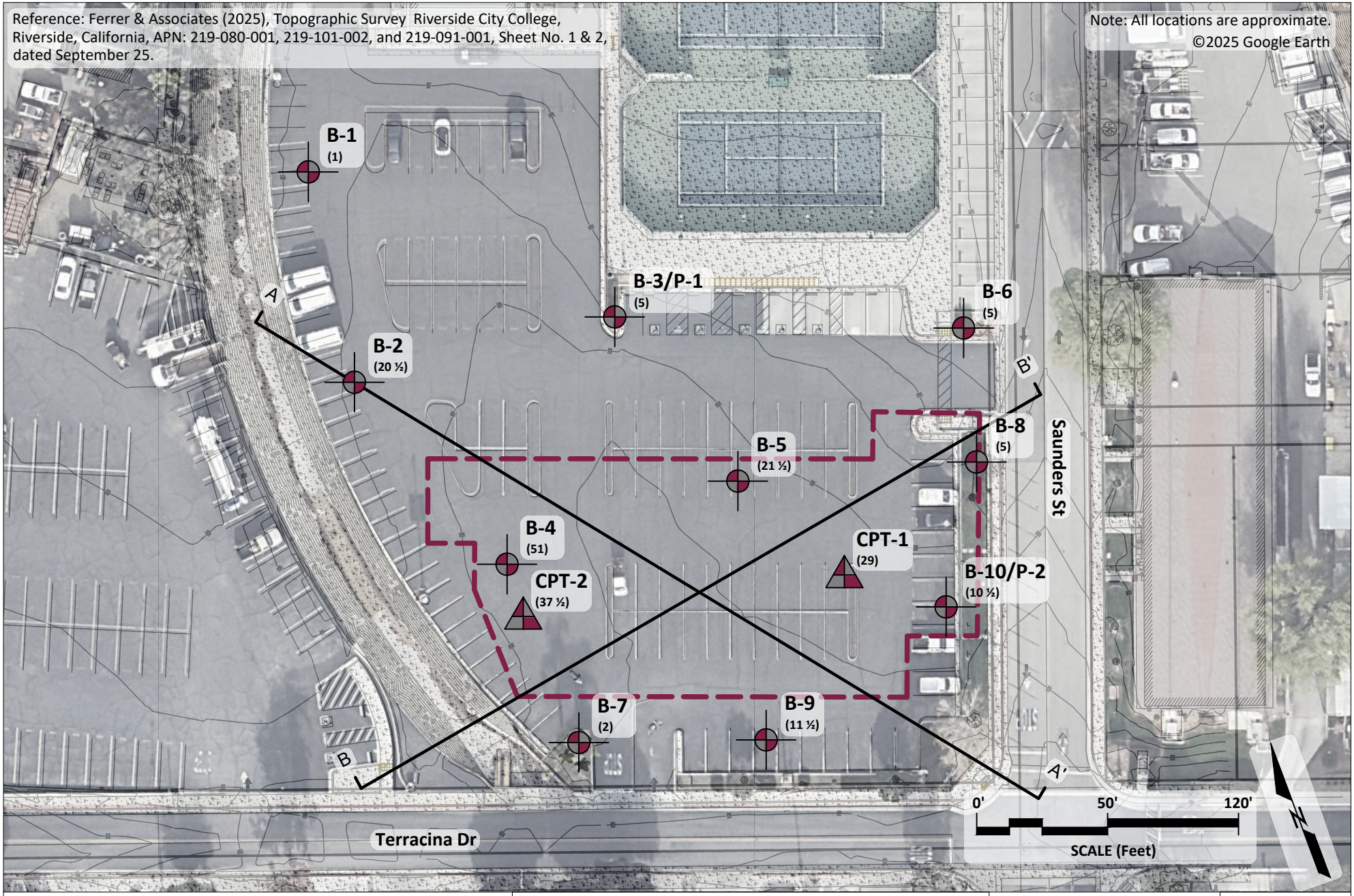


TOPOGRAPHIC MAP
Riverside City College New Cosmetology Building
Riverside, California

MTGL Project No.:	Date:	Author:
8073A06	November, 2025	NA

Geotechnical Engineering
Construction Inspection
Materials Testing
Environmental

Figure No.
2



LEGEND

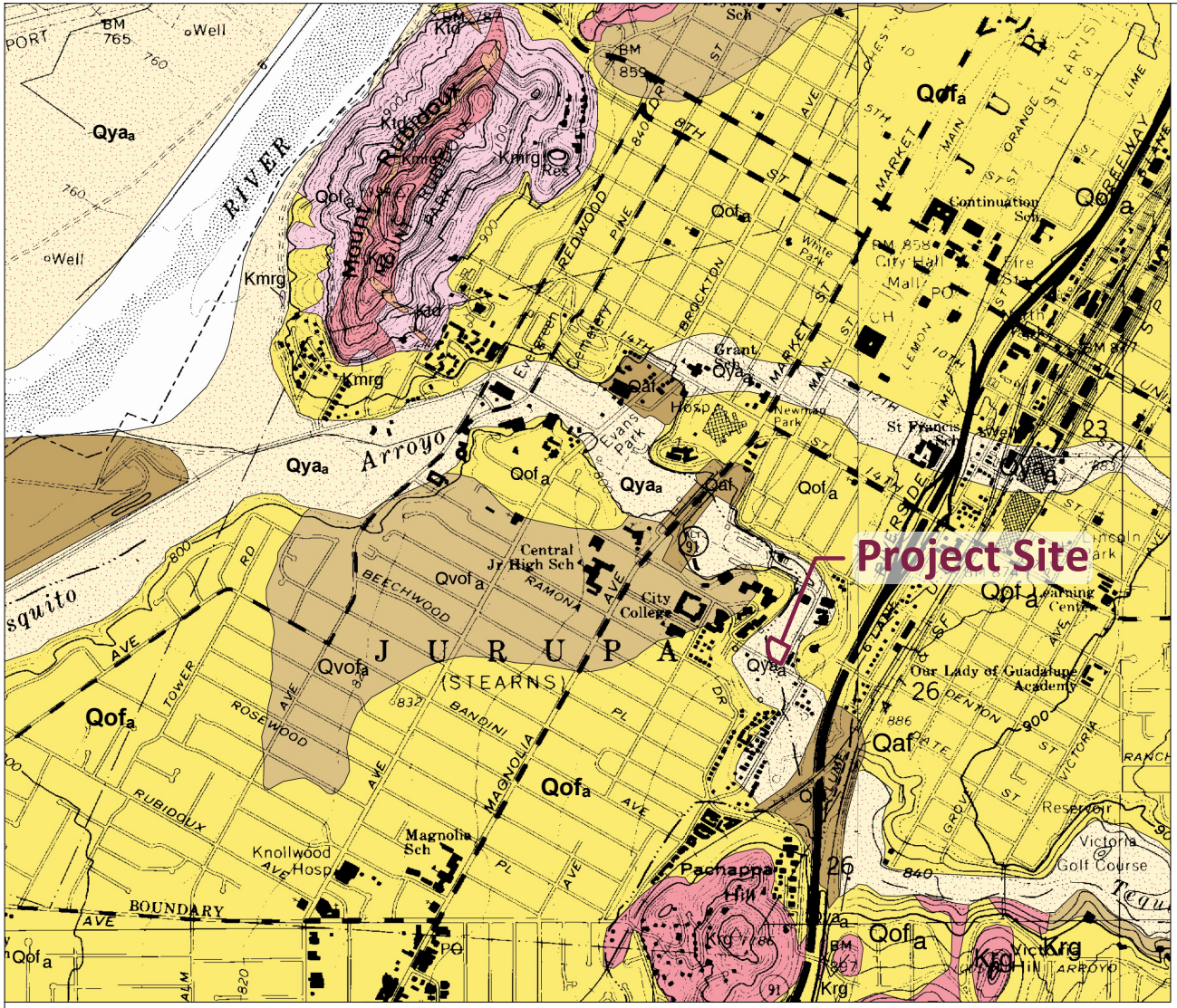
- Approximate Extent of Building
- B B' - Geologic Cross Section
- B-10 P-2 (10) - Location of HSA Boring Percolation Test (Depth in Feet)
- CPT-2 (37 ½) - Location of CPT (Depth in Feet)

SUBSURFACE EXPLORATION MAP
Riverside City College New Cosmetology Building
Riverside, California

MTGL Project No.:	Date:	Author:
8073A06	November, 2025	NA

Geotechnical Engineering
Construction Inspection
Materials Testing
Environmental

Figure No.
3



EXPLANATION:

Qaf — **Artificial fill (late Holocene)** – Deposits of fill resulting from human construction or mining activities.

Qya_a — **Young axial channel deposits (Holocene and late Pleistocene)** – Gray, unconsolidated alluvium consisting of coarse- to fine-grained sand and lesser gravel and silt.

Qof_a — **Old alluvial fan deposits (late to middle Pleistocene)** – Indurated, to slightly indurated, sandy, alluvial fan deposits. Most of unit is slightly to moderately dissected and reddish-brown. Locally includes thin, discontinuous surface layer of Holocene alluvial fan material.

$Qvof_a$ — **Very old alluvial fan deposits (early Pleistocene)** – Mostly well-dissected, well-indurated, reddish-brown sand deposits. Commonly contains duripans and locally silcretes. Deposits flanking bedrock slopes typically have well developed, dissected surfaces.

Kmrg — **Granite of Mount Rubidoux (Cretaceous)** – Massive granite characterized by coarse grain size and presence of hypersthene and fayalitic olivine.

Krg — **Granite of the Riverside area (Cretaceous)** – Medium- to coarse-grained, massive to faintly foliated, leucocratic biotite granite.

References:
Cox, B. and Morton, D.M. (2002), Geologic Map of the Riverside East 7.5' Quadrangle, Riverside County, California, scale 1:24,000.

0' 2000' 4000'
SCALE (Feet)

Note: All locations are approximate.

N

REGIONAL GEOLOGY MAP
Riverside City College New Cosmetology Building
Riverside, California

MTGL Project No.:	Date:	Author:
8073A06	November, 2025	NA

Geotechnical Engineering
Construction Inspection
Materials Testing
Environmental
MTGL

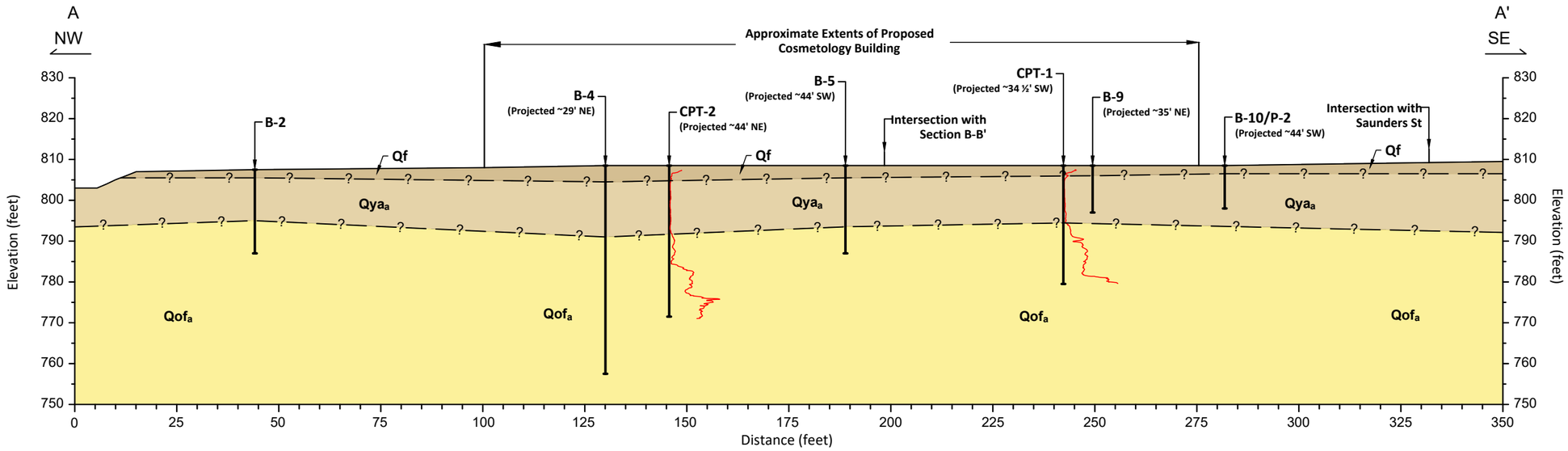
Figure No.
4

Plan Reference:
Ferrer & Associates (2025), Topographic Survey Riverside City College- 4800 Magnolia Ave Riverside California, F&A Project No. 219-080-001, 219-101-002, and 219-091-001, Sheet No. 1 & 2, dated September 24, 2025.

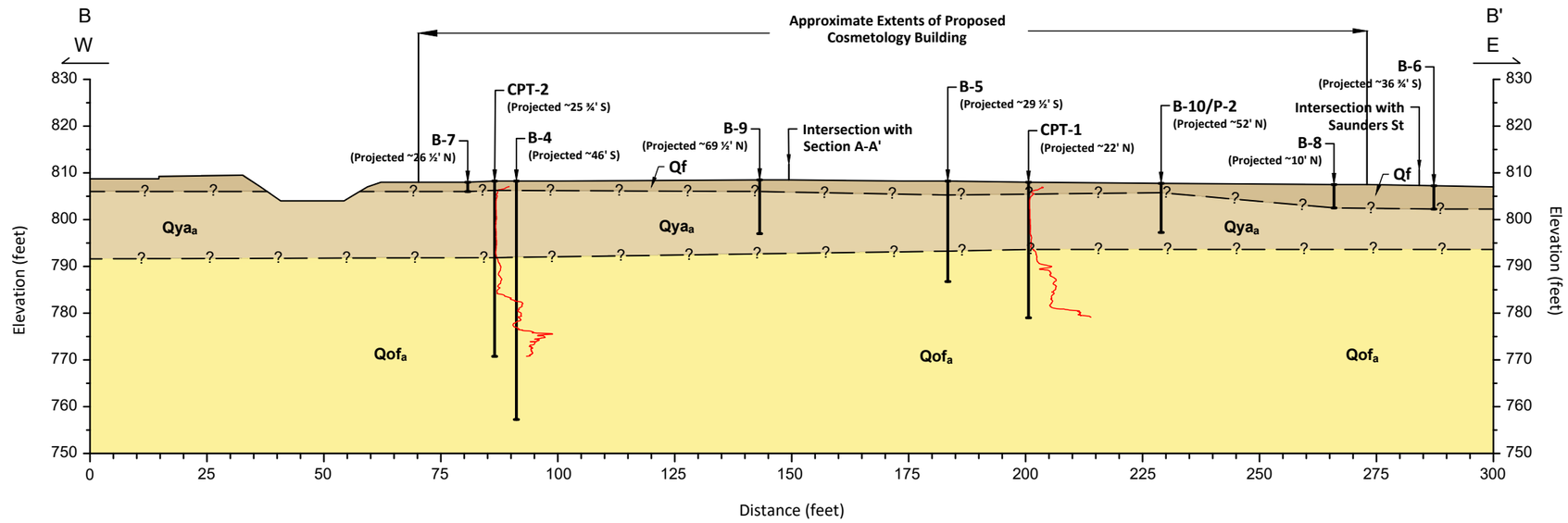
West Group Designs (2025), Riverside CCD - Enlarged 1st Floor Plan, undated.

Note: All locations and elevations are approximate.

SECTION A-A'



SECTION B-B'



SCALE (feet)
1:1 Horizontal to Vertical

LEGEND

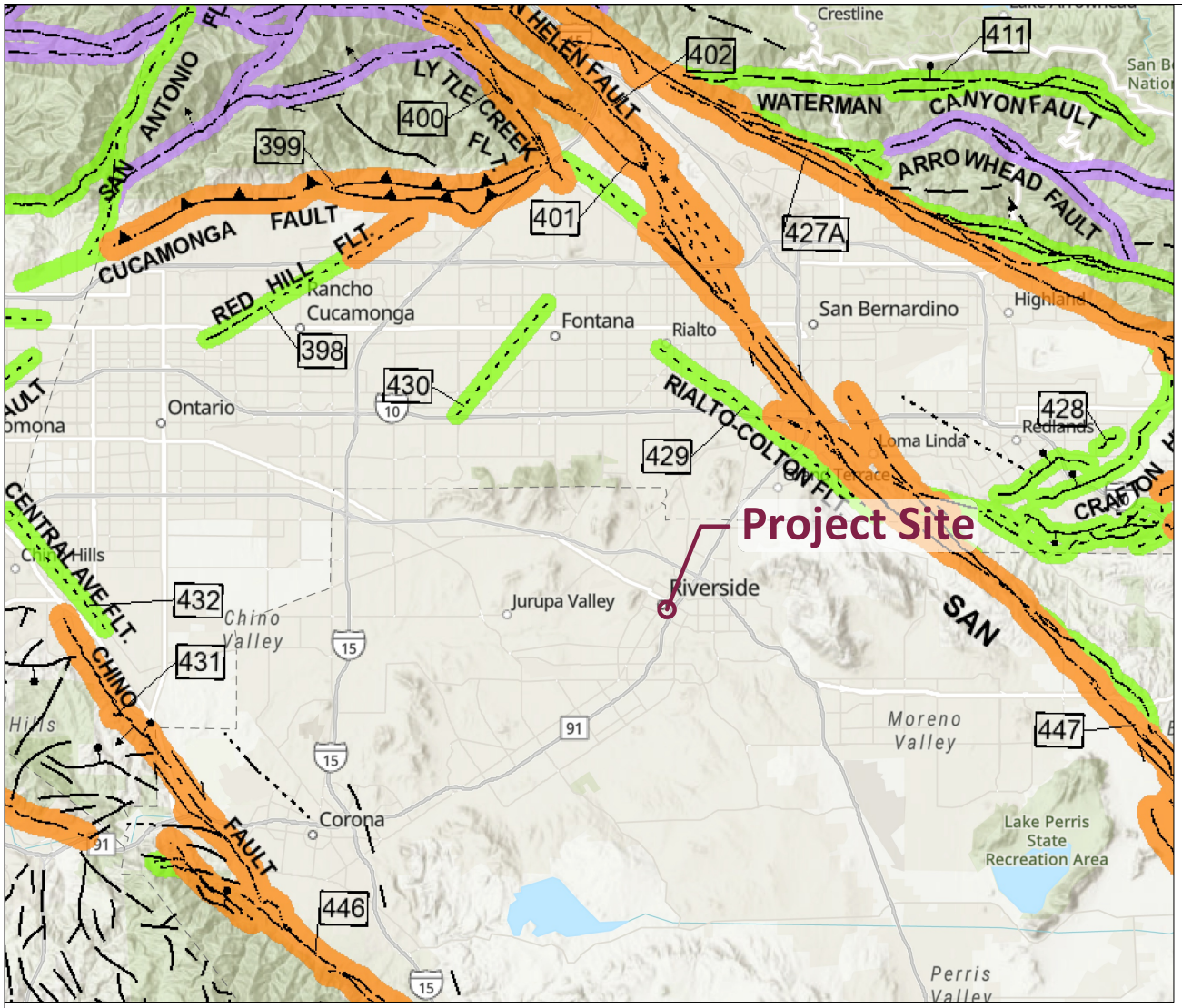
- **B-10/P-2 (XX)** — Location of Boring/Percolation Test (Distance Projected)
- **CPT-2 (XX)** — Location of CPT (Distance Projected)
- —?— Geologic Contact, Queried Where Uncertain or Inferred
- **Qf** — Fill
- **Qya$_a$** — Young Axial Channel Deposits
- **Qof$_a$** — Old Alluvial Fan Deposits

GEOLOGIC CROSS SECTIONS A-A' & B-B'
Riverside City College Cosmetology Building
Riverside, California

MTGL Project No.:	Date:	Author:
8073A06	November, 2025	NA/TC

Geotechnical Engineering
Construction Inspection
Materials Testing
Environmental

Figure No. 5



EXPLANATION:

Holocene fault displacement (during past 11,700 years) without historic record.

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

REFERENCE FAULT NUMBER:

429	Rialto-Colton Fault (San Bernardino Fault Zone), Late Quaternary
430	Inferred Fault Near Fontana, Possibly Late Quaternary; numerous closely aligned small earthquakes
446	Chino Fault (Elsinore Fault Zone), Holocene; Late Quaternary
447	Claremont Fault (San Jacinto Fault Zone), Holocene

Reference:
California Geological Survey, 2010, Fault Activity Map, C.W. Jennings, W.A. Bryant, accessed on November 21, 2025 at https://gis.conservation.ca.gov/server/admin/CGS/FaultActivityMapCA/MapServer

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

0 5 ½ 11

SCALE (Miles)

Note: All locations are approximate.

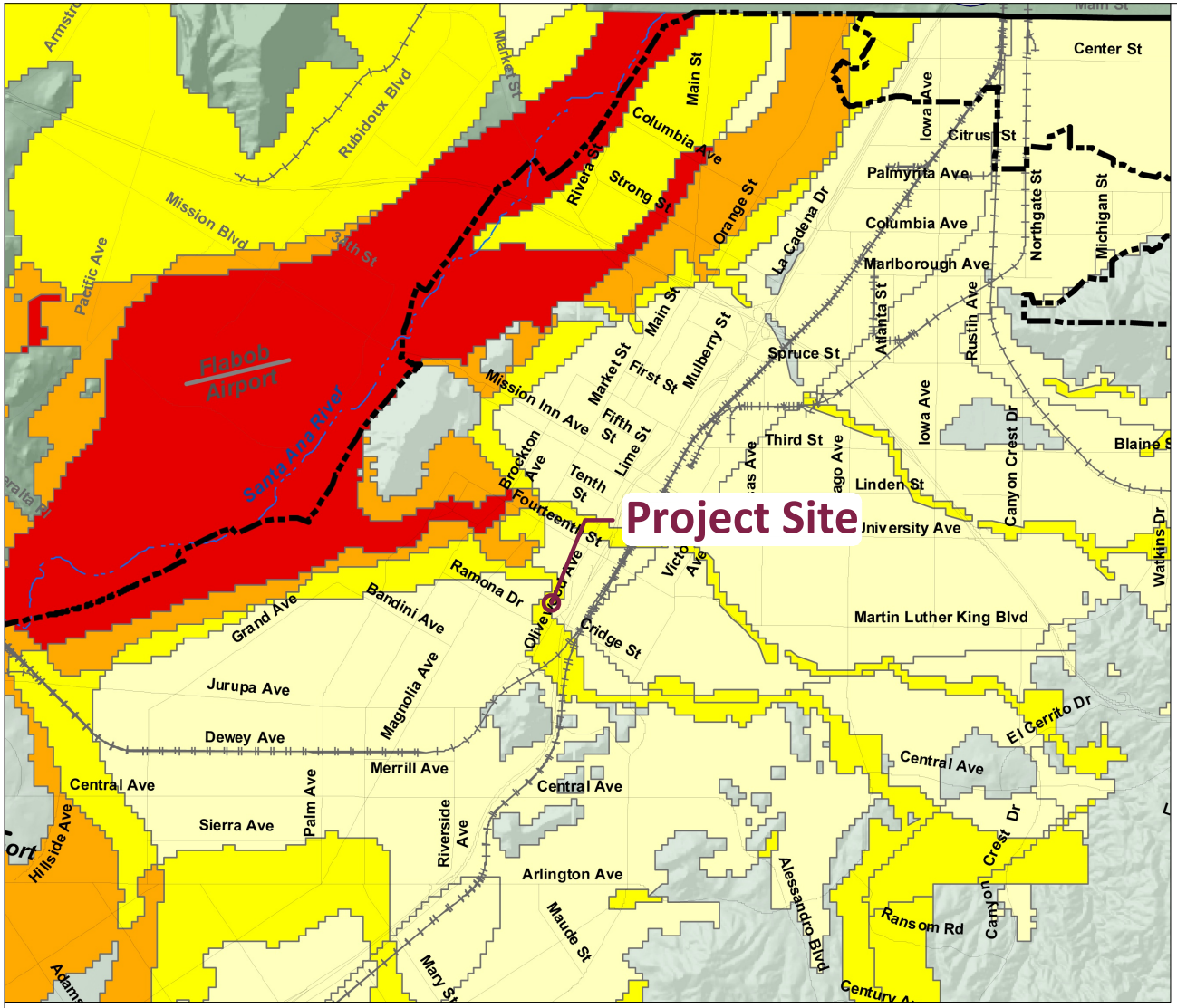
N

REGIONAL FAULT MAP
Riverside City College New Cosmetology Building
Riverside, California

MTGL Project No.:	Date:	Author:
8073A06	November, 2025	NA

Geotechnical Engineering
Construction Inspection
Materials Testing
Environmental

Figure No.
6



EXPLANATION:

LEGEND

- VERY LOW
- LOW
- MODERATE
- HIGH
- VERY HIGH
- RIVERSIDE CITY BOUNDARY
- RIVERSIDE PROPOSED SPHERE OF INFLUENCE

Reference:
Transportation and Land Management Agency (TLMA) Geographic Information Services
County of Riverside, January 1, 2005, https://www.tlma.co.riverside.ca.us/index.html

0' 1 2
SCALE (Miles)

Note: All locations are approximate.

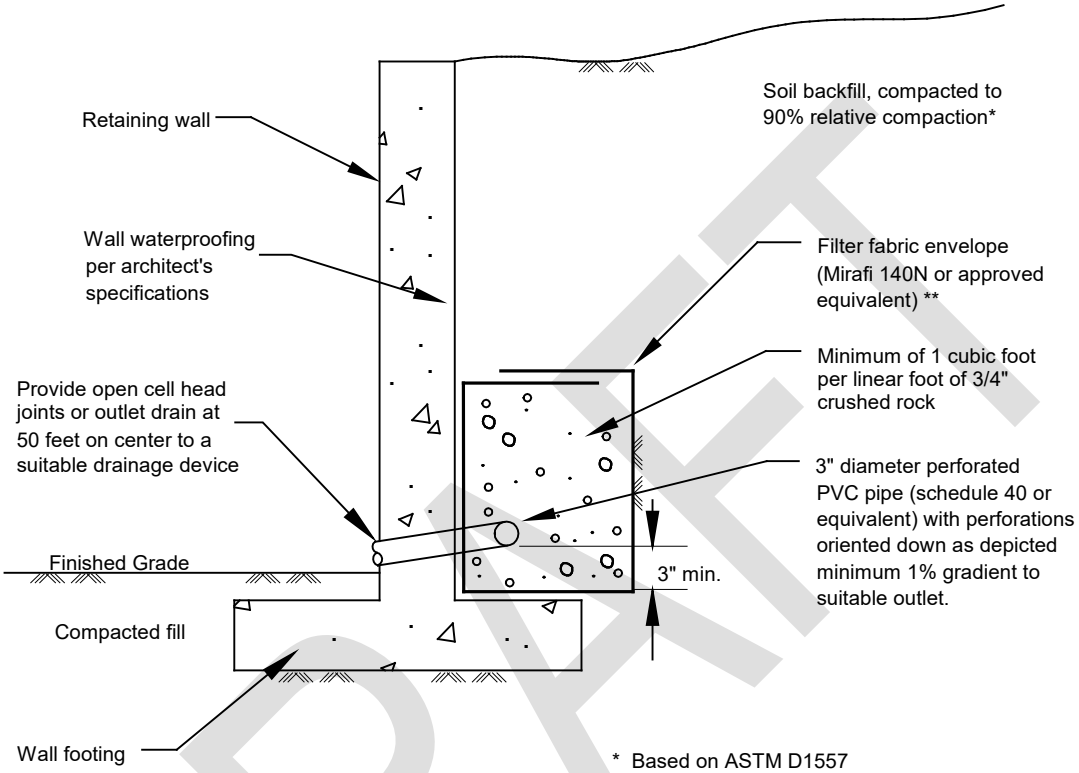
N

CITY OF RIVERSIDE PUBLIC SAFETY ELEMENT
LIQUEFACTION ZONES
Riverside City College New Cosmetology Building
Riverside, California

MTGL Project No.:	Date:	Author:
8073A06	November, 2025	NA

Geotechnical Engineering
Construction Inspection
Materials Testing
Environmental

Figure No.
7



* Based on ASTM D1557

** If class 2 permeable material (See gradation to left) is used in place of 3/4" - 1 1/2" gravel. Filter fabric may be deleted. Class 2 permeable material compacted to 90% relative compaction. *

SPECIFICATIONS FOR CLASS 2 PERMEABLE MATERIAL (CAL TRANS SPECIFICATIONS)

Sieve Size	% Passing
1"	100
3/4"	90-100
3/8"	40-100
No.4	25-40
No.8	18-33
No.30	5-15
No.50	0-7
No.200	0-3

RETAINING WALL DRAINAGE DETAIL

DRAFT

APPENDIX A

REFERENCES

DRAFT

APPENDIX A

REFERENCES

American Society of Civil Engineers, 2022, ASCE 7-22 Minimum Design Loads and Associated Criteria for Building and Other Structures.

Burmister, DM (1948). "Importance & Practical Use of Relative Density in Soil Mechanics", Proc., ASTM, 48, 1249-1268.

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State of California, 2024, Tsunami Hazard Area Map, https://www.conservation.ca.gov/cgs/tsunami/maps/los-angeles.html.

United States Geological Survey, 2025, Areas of Land Subsidence in California, https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html.

United States Geological Survey, 2024, U.S. Quaternary Faults, https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf, accessed February.

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APPENDIX B

FIELD EXPLORATION PROGRAM

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APPENDIX B

FIELD EXPLORATION PROGRAM

The subsurface conditions for this geotechnical investigation were explored by drilling exploratory borings using an 8-inch diameter hollow-stem auger and hand tools, and by advancing cone penetration tests (CPTs). Driven samples were obtained using the Standard Penetration Test (SPT) sampler or a Modified California split-spoon sampler. The approximate boring and CPT locations are shown on the Subsurface Exploration Map (Figure 3). The field exploration was performed under the supervision of an MTGL engineer, who logged the subsurface materials encountered and obtained samples for laboratory testing.

Subsurface conditions are summarized on the accompanying Logs of Borings and Summary of CPT Findings. These logs contain descriptive information and interpretations of subsurface stratigraphy based on the obtained samples and CPT data. The strata indicated on the logs represent approximate boundaries between earth materials; transitions are often gradual rather than sharply defined. The logs depict conditions at the specific locations and dates of exploration and may not represent subsurface conditions at other locations or times.

Soil identification during the subsurface exploration was performed in accordance with the field procedures of the Unified Soil Classification System (ASTM D2488). A legend indicating the symbols and definitions used in this classification system, as well as definitions of relative density, consistency, and firmness terms, is included in this appendix. Bag samples of major earth units were obtained for laboratory inspection and testing, and in-place density (blow count) information was collected where applicable.

The exploratory borings were backfilled with sand cement grout or auger cuttings and patched where appropriate.

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UNIFIED SOIL CLASSIFICATION SYSTEM				
Coarse-grained soil. More than 1/2 of materials is larger than #200 sieve	GRAVELS More than half of coarse fraction larger than #4 sieve	Clean Gravels with less than 15% fines	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with more than 15% fines	GM	Silty Gravels, poorly graded gravel-sand-silt mixtures
			GC	Clayey Gravels, poorly graded gravel-sand-clay mixtures
	SANDS More than half of coarse fraction smaller than #4 sieve	Clean Sands with less than 15% fines	SW	Well-graded sands or Well-graded Sands with Gravel, little or no fines
			SP	Poorly graded sands, or Poorly graded Sands with Gravel, little or no fines
		Sands with more than 15% fines	SM	Silty Sands or Silty Sands with Gravel
			SC	Clayey Sands or Clayey Sands with Gravel
Fine-grained soil. More than 1/2 of material is smaller than #200 sieve	Silts and Clays Liquid limit 50% or less		ML	Inorganic silts of low to med plasticity
			CL	Inorganic clays of low to med plasticity, gravelly, sandy, silty, or lean clays
			OL	Organic silts and clays of low plasticity
	Silts and Clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic silts and clays of medium to high plasticity
Highly Organic Soils			PT	Peat, humus swamp soils with high organic content

GRAIN SIZE					SIZE PROPORTION
Description		Sieve Size	Grain Size	Approximate Size	Trace – Less than 5%
Boulders		>12"	>12"	Larger than basketball-sized	Few – 5% to 10%
Cobbles		3"- 12"	3"- 12"	Fist-sized to basketball-sized	Little – 15% to 20%
Gravel	Coarse	¾"- 3"	¾"- 3"	Thumb-sized	Some – 30% to 45%
	Fine	#4 - ¾"	0.19" - 0.75"	Peat-sized to thumb-sized	Mostly – 50% to 100%
Sand	Coarse	#10 - #4	0.079" - 0.19"	Rock salt-sized to pea-sized	**MOISTURE CONTENT**
	Medium	#40 - #10	0.017" - 0.079"	Sugar-sized to rock salt-sized	Dry – Absence of moisture
	Fine	#200 - #40	0.0029" - 0.017"	Flour-sized to sugar-sized	Moist – Damp but not visible
Fines		Passing #200	<0.0029"	Flour-sized or smaller	Wet – Visible free water

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Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Key to Log of Boring

Sheet 1 of 1

1 Elevation (feet)	2 Depth (feet)	3 Sample Type	4 Sampling Resistance	5 N60	6 Material Type	7 Graphic Log	8 MATERIAL DESCRIPTION	9 Water Content, %	10 Dry Unit Weight, pcf	11 Lab Testing	12 REMARKS

COLUMN DESCRIPTIONS

1 Elevation (feet): Elevation (MSL, feet).
2 Depth (feet): Depth in feet below the ground surface.
3 Sample Type: Type of soil sample collected at the depth interval shown.
4 Sampling Resistance : Number of blows to advance driven sampler one foot (or distance shown) beyond seating interval using the hammer identified on the boring log.
5 N60: Energy corrected SPT blow count per foot.
6 Material Type: Type of material encountered.
7 Graphic Log: Graphic depiction of the subsurface material encountered.
8 MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.
9 Water Content, %: Water content of the soil sample, expressed as percentage of dry weight of sample.
10 Dry Unit Weight, pcf: Dry weight per unit volume of soil sample measured in laboratory, in pounds per cubic foot.
11 Lab Testing : Laboratory Testing
12 REMARKS : Comments and observations regarding drilling or sampling made by driller or field personnel.

FIELD AND LABORATORY TEST ABBREVIATIONS

COR: Corrosive Series Tests (CTM-643,417,422)
MAX: Maximum Dry Density and Optimum Moisture (ASTM D1557)
DS: Direct Shear Test (ASTM, D3080)
EI: Expansion Index, (ASTM, D4829)
AL: Atterberg Limits (ASTM, D4318)
PD: Particle-Size Distribution using Sieve Analysis (ASTM D6913)
CON: One-Dimensional Consolidation (ASTM D2435)
WA: Wash sieve (percent passing No. 200 Sieve)

MATERIAL GRAPHIC SYMBOLS

Asphalt Concrete (AC)
Aggregate Base (AB)
Lean CLAY, CLAY w/SAND, SANDY lean CLAY (CL)
SILTY CLAY (CL-ML)
SILT, SILT w/SAND, SANDY SILT (ML)
Clayey SAND (SC)
SILTY CLAYEY SAND (SC-SM)
Silty SAND (SM)
Poorly graded SAND (SP)
Poorly graded SAND with Clay (SP-SC)
Well graded SAND with Clay (SW-SC)

TYPICAL SAMPLER GRAPHIC SYMBOLS

Bulk Sample
2.5-inch-OD Modified California
2-inch-OD unlined split spoon (SPT)

OTHER GRAPHIC SYMBOLS

Water level (at time of drilling, ATD)
Water level (after waiting, AW)
Minor change in material properties within a stratum
Inferred/gradational contact between strata
Queried contact between strata

GENERAL NOTES

1: Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2: Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

Figure B3

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-1
Sheet 1 of 1

Date(s) Drilled **10/23/2025**	Logged By **NA / IC**	Checked By
Drilling Method **Hand Auger**	Drill Bit Size/Type **4-inch / Hand Auger**	Total Depth of Borehole **1 Foot BGS**
Drill Rig Type **N/A**	Drilling Contractor **MTGL Inc.**	Approximate Surface Elevation **802 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **N/A**	Hammer Data **N/A**
Borehole Backfill **Native Soil Cuttings**	Location **33.970015, -117.379281**	

Elevation (feet)	Depth (feet)	Material Type	MATERIAL DESCRIPTION
802	0	Asphalt	3 inches of Asphalt Concrete (AC) over 3 inches of Aggregate Base (AB).
		Base	**Fill (Qf)**: CLAYEY SAND, loose, brown, moist, fine to medium grained.
		SC	Practical Hand Auger Refusal at Approximately 1 Foot BGS on Geogrid Material. No Groundwater Encountered. Boring Backfilled 10/23/2025. Surface Patched with Aquaphalt.
797	5		
792	10		
787	15		
782	20		

Sample Type | Sampling Resistance | N60 | Graphic Log | Water Content, % | Dry Unit Weight, pcf | Lab Testing | REMARKS

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-2
Sheet 1 of 1

Date(s) Drilled **10/22/2025**	Logged By **NA / TC**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch / Hollow Stem Auger**	Total Depth of Borehole **20 ½ Feet BGS**
Drill Rig Type **Truck Mounted CME-75**	Drilling Contractor **Baja Exploration**	Approximate Surface Elevation **803 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk, Modified California, SPT**	Hammer Data **140 lb / 30" Drop (ETR = 71.9%)**
Borehole Backfill **Sand-Cement Grout**	Location **33.969817, -117.379337**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
803	0				Asphalt Base		3 inches of Asphalt Concrete (AC), over 2 inches of Aggregate Base (AB), over geogrid, over 10 inches (AB), over geogrid.				
					CL-ML		**Fill (Qf)**: SILTY CLAY with SAND, soft, brown, moist, fine grained sands, micaceous.				
					SM		**Young Axial Channel Deposits (Qya)**: SILTY SAND, loose to medium dense, brown, moist, fine grained, trace rootlets, manganese staining, micaceous.			AL, WA, EI, COR	
798	5		2 2 2	5	SC-SM		SILTY CLAYEY SAND, loose, brown, moist, fine grained, micaceous, thin interbeds of sandy lean clay (CL).				
			5 5 5	8				13.1	113.0	DS	
793	10		3 3 4	8	SM		SILTY SAND, loose, light brown, moist, fine to medium grained, slighty micaceous.				
			5 50/6"	>50			Old Alluvial Fan Deposits (Qof): SILTY SAND very dense, light brown to brown to gray, fine to coarse grained, interbedded with well graded sands with silt (SW-SM).				
788	15		42 50/5"	>50	SC		CLAYEY SAND, very dense, light brown to reddish brown to gray, moist, fine to coarse grained, few gravel, micaceous.			PD	
			36 50/4"	>50	SW-SC		Well-graded SAND with CLAY, very dense, light brown to reddish brown to gray, moist, fine to coarse grained, few gravel, micaceous.				
783	20		50/5"	>50			Boring Terminated at Approximately 20 ½ Feet BGS. No Groundwater Encountered. Boring Backfilled 10/23/2025.				

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-3 / P-1

Sheet 1 of 1

Date(s) Drilled **10/22/2025**	Logged By **NA / TC**	Checked By
Drilling Method **Hand Auger**	Drill Bit Size/Type **8-inch / Hand Auger**	Total Depth of Borehole **5 Feet BGS**
Drill Rig Type **N/A**	Drilling Contractor **Baja Exploration**	Approximate Surface Elevation **806 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk**	Hammer Data **N/A**
Borehole Backfill **Native Soil Cuttings**	Location **33.969733, -117.379027**	

Elevation (feet)	Depth (feet)	Material Type	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
806	0	SM	**Fill (Qf)**: SILTY SAND, loose to medium dense, brown to light brown, moist, fine to coarse grained, trace debris, few gravel, few roots.				
		CL-ML	**Young Axial Channel Deposits (Qya)**: SANDY SILTY CLAY, medium stiff, brown, moist, fine grained, micaceous, trace manganese staining.	10.2	121.7	WA, DS, MAX	
		SC-SM	SILTY CLAYEY SAND, loose, brown, moist, fine grained, micaceous.			WA	
801	5		Boring Terminated at Approximately 5 Feet BGS. No Groundwater Encountered. Boring Backfilled 10/22/2025.				
796	10						
791	15						
786	20						

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-4
Sheet 1 of 3

Date(s) Drilled **10/22/2025**	Logged By **NA / TC**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch / Hollow Stem Auger**	Total Depth of Borehole **51 Feet BGS**
Drill Rig Type **Truck Mounted CME-75**	Drilling Contractor **Baja Exploration**	Approximate Surface Elevation **803 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk, Modified California, SPT**	Hammer Data **140 lb / 30" Drop (ETR = 71.9%)**
Borehole Backfill **Sand-cement grout**	Location **33.969578, -117.379333**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
803	0				Asphalt Base		2½ inches of Asphalt Concrete (AC), over 5 inches of Aggregate Base (AB), over geogrid, over 16½ inches of (AB), over geogrid.				
					SC-SM		**Fill (Qf)**: SILTY CLAYEY SAND, loose, brown to dark brown, moist, fine to medium grained, micaceous.			WA	
					SC-SM		**Young Axial Channel Deposits (Qya)**: SILTY CLAYEY SAND, loose, brown to dark brown, moist, fine to medium grained, micaceous, pinhole voids.				
798	5		4 5 7	9	CL-ML		SANDY SILTY CLAY, stiff, brown, moist, fine to medium grained, trace gravel, micaceous, pinhole voids.	8.6	112.3		
			2 3 2	6			Medium stiff.			AL, WA	
793	10		5 7 7	11	SC-SM		SILTY CLAYEY SAND, stiff, brown, moist, thin interbeds of silt and clay.	7.5	105.7	WA	
			3 3 3	7							
788	15		3 8 9	13	CL		SANDY lean CLAY, stiff, brown, moist, fine to medium grained, micaceous, some silt.	12.9	118	WA	
			4 5 6	13	SM		**Old Alluvial Fan Deposits (Qof)**: SILTY SAND, medium dense, brown to light brown, moist, medium to coarse grained, slightly micaceous, decomposed granite.			WA	
783	20		7 10 13	18	SP-SC		Poorly graded SAND with CLAY, medium dense, moist, medium to coarse grained, thin interbeds of clayey sand (SC).	3.1	104.5	WA	

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-4
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
							Very dense, increased difficulty drilling.				
778	25		19 25 30	>50			Light brown to reddish brown, trace silt, increased fines, slight micaceous, subrounded gravels in cuttings from 25 - 26 feet.				
							Increased difficulty drilling.				
773	30		50/4"	>50	SC		CLAYEY SAND, very dense, brown to reddish brown to light brown to olive, moist, fine to coarse grained, interbeds of poorly graded sand with clay.				
768	35		23 50/6"	>50	SP		Poorly graded SAND, very dense, light brown, moist, medium grained, trace silt.				
							Driller remarks feels like gravel at 38 feet.				
763	40		21 50/6"	>50	SW-SC		Well-graded SAND with CLAY and GRAVEL, very dense, grayish brown, moist, fine to coarse grained.				

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-4
Sheet 3 of 3

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
758	45		50/3"	>50	SW-SC		Well-graded SAND with CLAY and GRAVEL, very dense, grayish brown, moist, fine to coarse grained.				
753	50		27 50/3"	>50			Slight decrease in fines.				
							Boring Terminated at Approximately 51 Feet BGS. No Groundwater Encountered. Boring Backfilled 10/22/2025 With Portland-Cement Grout. Surface Patched with Aquaphalt.				
748	55										
743	60										
738	65										

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-5
Sheet 1 of 1

Date(s) Drilled **10/22/2025**	Logged By **NA / TC**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch / Hollow Stem Auger**	Total Depth of Borehole **21 ½ Feet BGS**
Drill Rig Type **Truck Mounted CME-75**	Drilling Contractor **Baja Exploration**	Approximate Surface Elevation **803 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk, Modified California, SPT**	Hammer Data **140 lb / 30" Drop (ETR = 71.9%)**
Borehole Backfill **Sand-cement grout**	Location **33.969514, -117.378993**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
803	0				Asphalt Base		3 inches of Asphalt Concrete (AC), over 10 inches of Aggregate Base (AB), over geogrid, over 19 inches of (AB), over geogrid.				
					CL-ML		**Fill (Qf)**: SANDY SILTY CLAY, medium stiff, brown, moist, fine grained sands.			PD	
					SC-SM		**Young Axial Channel Deposits (Qya)**: CLAYEY SILTY SAND, loose, brown, moist, fine grained.				
798	5		4 4 5	7				11.2	101.0	CON	
			2 2 1	4	CL-ML		SANDY SILTY CLAY, medium stiff, brown, moist, fine grained sands.				
793	10		4 4 4	6							
			3 5 7	14			Stiff.				
788	15		8 20 31	40	SC		**Old Alluvial Fan Deposits (Qof)**: CLAYEY SAND, dense, brown, moist, fine to coarse grained, manganese staining.				
			6 11 17	34	CL		SANDY LEAN CLAY, hard, brown, moist.				
783	20		18 27 50/5"	>50	SC		CLAYEY SAND, very dense, brown, moist, fine to medium grained.				
							Boring Terminated at Approximately 21 ½ Feet BGS.\ No Groundwater Encountered. Boring Backfilled 10/23/2025.				

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-6
Sheet 1 of 1

Date(s) Drilled **10/23/2025**	Logged By **NA / IC**	Checked By
Drilling Method **Hand Auger**	Drill Bit Size/Type **4-inch / Hand Auger**	Total Depth of Borehole **5 Feet BGS**
Drill Rig Type **N/A**	Drilling Contractor **MTGL Inc.**	Approximate Surface Elevation **802 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **N/A**	Hammer Data **N/A**
Borehole Backfill **Native Soil Cuttings**	Location **33.969541, -117.378629**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
802	0				SC		**Fill (Qf)**: CLAYEY SAND, loose to medium dense, brown, moist, fine to medium grained, some roots.				
							Light brown.				
797	5						Boring Terminated at Approximately 5 Feet BGS. No Groundwater Encountered. Boring Backfilled 10/23/2025.				
792	10										
787	15										
782	20										

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-7
Sheet 1 of 1

Date(s) Drilled **10/23/2025**	Logged By **NA / IC**	Checked By
Drilling Method **Hand Auger**	Drill Bit Size/Type **4-inch / Hand Auger**	Total Depth of Borehole **2 Feet BGS**
Drill Rig Type **N/A**	Drilling Contractor **MTGL Inc.**	Approximate Surface Elevation **804 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **N/A**	Hammer Data **N/A**
Borehole Backfill **Native Soil Cuttings**	Location **33.969354, -117.379334**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
804	0				SC		**Fill (Qf)**: CLAYEY SAND, brown, moist, fine to medium grained, lots of roots, very hard drilling.				
							Practical Hand Auger Refusal at Approximately 2 Feet BGS. No Groundwater Encountered. Boring Backfilled 10/23/2025.				
799	5										
794	10										
789	15										
784	20										

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

Log of Boring B-8
Sheet 1 of 1

Date(s) Drilled **10/23/2025**	Logged By **NA / IC**	Checked By
Drilling Method **Hand Auger**	Drill Bit Size/Type **4-inch / Hand Auger**	Total Depth of Borehole **5 Feet BGS**
Drill Rig Type **N/A**	Drilling Contractor **MTGL Inc.**	Approximate Surface Elevation **803 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **N/A**	Hammer Data **N/A**
Borehole Backfill **Native Soil Cuttings**	Location **33.969412, -117.378722**	

Elevation (feet)	Depth (feet)	Material Type	MATERIAL DESCRIPTION
803	0	SC	**Fill (Qf)**: CLAYEY SAND, loose, brown, moist, fine to medium grained.
798	5		Boring Terminated at Approximately 5 Feet BGS. No Groundwater Encountered. Boring Backfilled 10/23/2025.
793	10		
788	15		
783	20		

Sample Type | Sampling Resistance | N60 | Graphic Log | Water Content, % | Dry Unit Weight, pcf | Lab Testing | REMARKS

Project: **Riverside City College Cosmetology Building**
Project Location: **4800 Magnolia Avenue, Riverside, CA 92506**
Project Number: **8073A06**

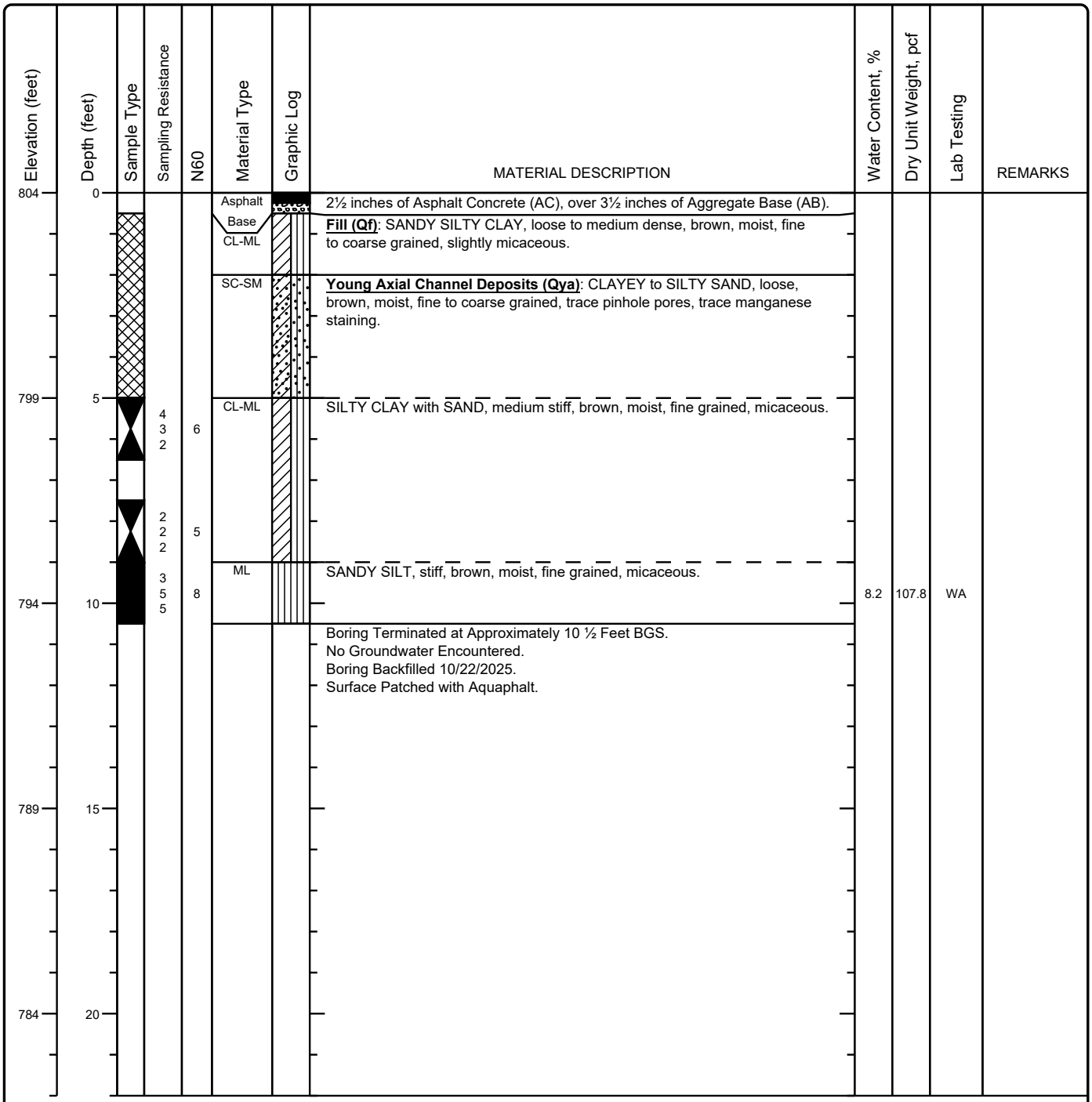
Log of Boring B-9
Sheet 1 of 1

Date(s) Drilled **10/22/2025**	Logged By **NA / TC**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch / Hollow Stem Auger**	Total Depth of Borehole **11 ½ Feet BGS**
Drill Rig Type **Truck Mounted CME-75**	Drilling Contractor **Baja Exploration**	Approximate Surface Elevation **804 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk, SPT**	Hammer Data **140 lb / 30" Drop (ETR = 71.9%)**
Borehole Backfill **Native Soil Cuttings**	Location **33.969271, -117.379122**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
804	0				Asphalt Base		3 inches of Asphalt Concrete over 14 inches of Aggregate Base (AB), over geogrid.				
					CL		**Fill (Qf)**: SANDY LEAN CLAY, medium stiff, brown to dark brown, moist, fine to medium grained sands, micaceous.				
					SC-SM		**Young Axial Channel Deposits (Qya)**: SILTY CLAYEY SAND, loose, brown, moist, fine grained, micaceous, trace pinhole voids.				
799	5		2 2 2	5							
794	10		3 2 2	5			Slightly less fines.				
							Boring Terminated at Approximately 11 ½ Feet BGS. No Groundwater Encountered. Boring Backfilled 10/22/2025. Surface Patched with Aquaphalt.				
789	15										
784	20										

Project: **Riverside City College Cosmetology Building** Project Location: **4800 Magnolia Avenue, Riverside, CA 92506** Project Number: **8073A06**	Log of Boring B-10 / P-2 **Sheet 1 of 1**

Date(s) Drilled **10/22/2025**	Logged By **NA / TC**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch / Hollow Stem Auger**	Total Depth of Borehole **10 ½ Feet BGS**
Drill Rig Type **Truck Mounted CME-75**	Drilling Contractor **Baja Exploration**	Approximate Surface Elevation **804 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk, Modified California, SPT**	Hammer Data **140 lb / 30" Drop (ETR = 71.9%)**
Borehole Backfill **Native Soil Cuttings**	Location **33.969318, -117.378854**	



SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

RCC Cosmetology Geo Inv
Terracina Drive & Saunders Street
Riverside, CA
October 23, 2025

Prepared for:

Mr. Trevor Carter
MTGL, Inc.
7742 Arjons Drive
San Diego, CA 92126
Office (858) 537-3999 / Fax (858) 537-3990

Prepared by:

KEHOE TESTING & ENGINEERING
5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

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1. **INTRODUCTION**
2. **SUMMARY OF FIELD WORK**
3. **FIELD EQUIPMENT & PROCEDURES**
4. **CONE PENETRATION TEST DATA & INTERPRETATION**

APPENDIX

- CPT Plots
- CPT Classification/Soil Behavior Chart
- Summary of Shear Wave Velocities
- CPT Data Files (sent via email)

SUMMARY
OF
CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the RCC Cosmetology Geo Inv project located at Terracina Drive & Saunders Street in Riverside, California. The work was performed by Kehoe Testing & Engineering (KTE) on October 23, 2025. The scope of work was performed as directed by MTGL, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at two locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	28	Refusal
CPT-2	37	Refusal

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone with a cone net area ratio of 0.83. The following parameters were recorded at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

At location CPT-1 & CPT-2, shear wave measurements were obtained at various depths. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, "Interpretation of Cone Penetration Test…", 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING



Steven P. Kehoe
President

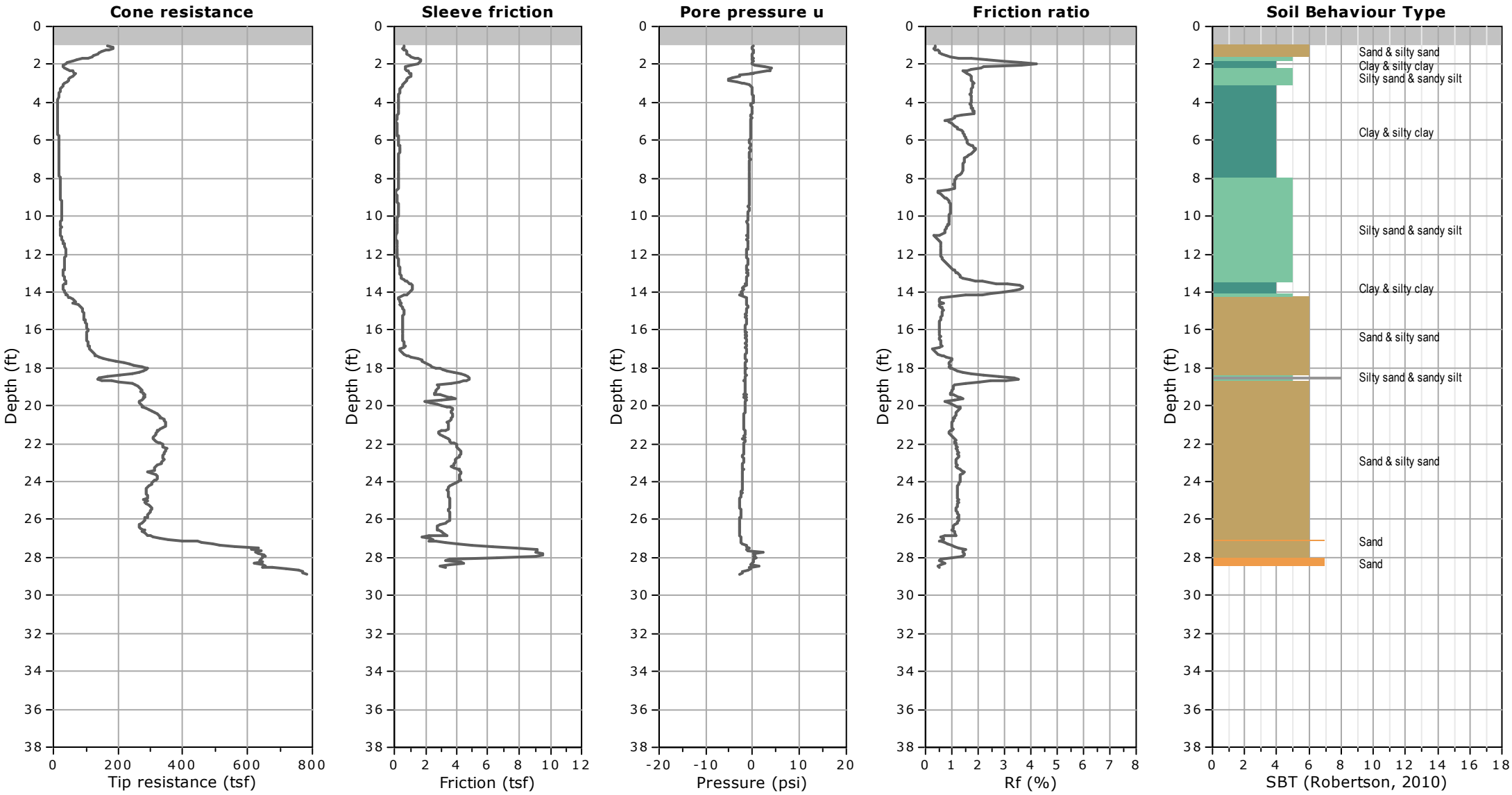
11/04/25-ca-8161

APPENDIX

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **MTGL / RCC Cosmetology Geo Inv**
Location: **Terracina Dr, Riverside, CA**

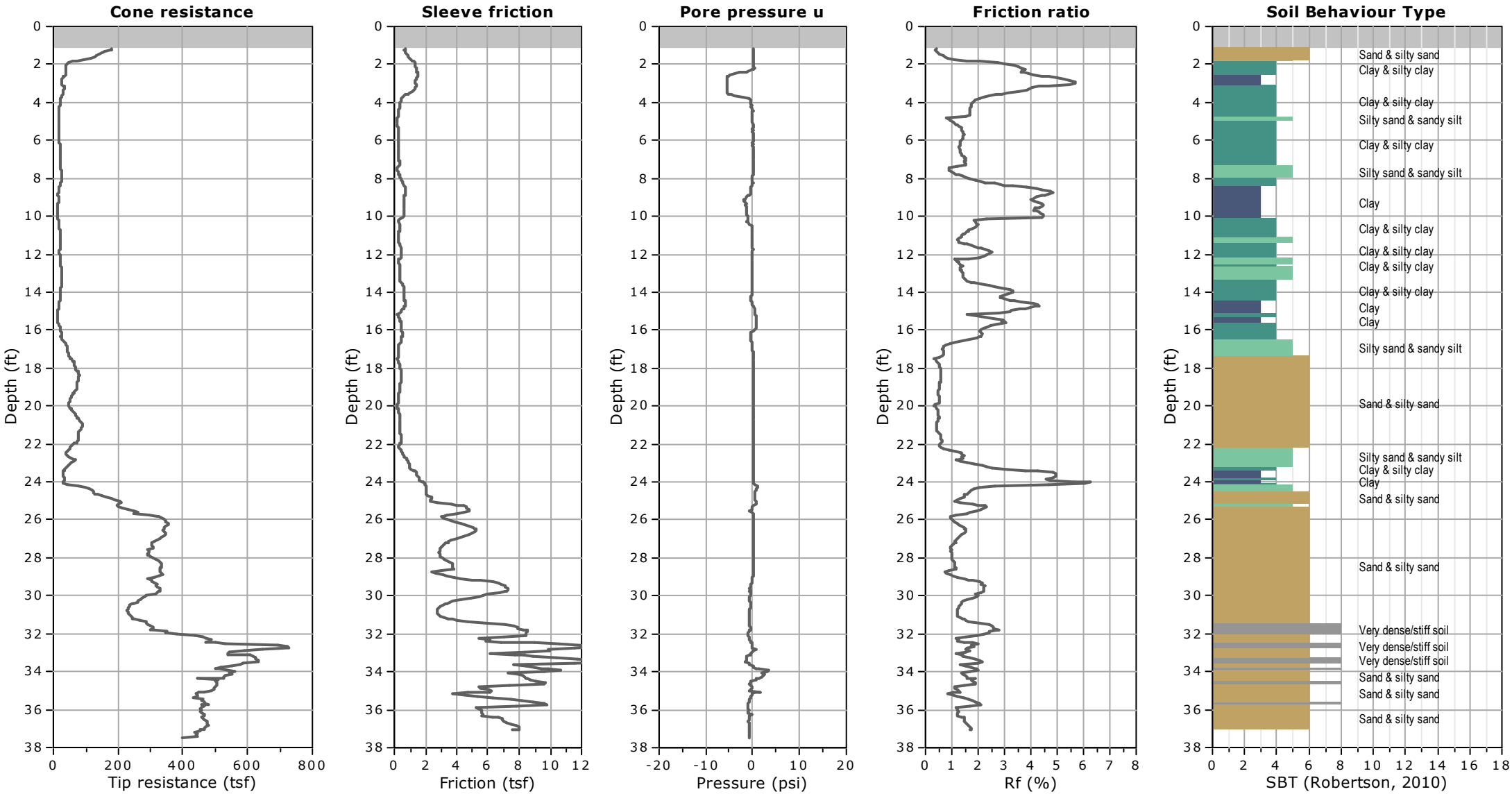
CPT-1
Total depth: 28.88 ft, Date: 10/23/2025



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: MTGL / RCC Cosmetology Geo Inv
Location: Terracina Dr, Riverside, CA

CPT-2
Total depth: 37.47 ft, Date: 10/23/2025



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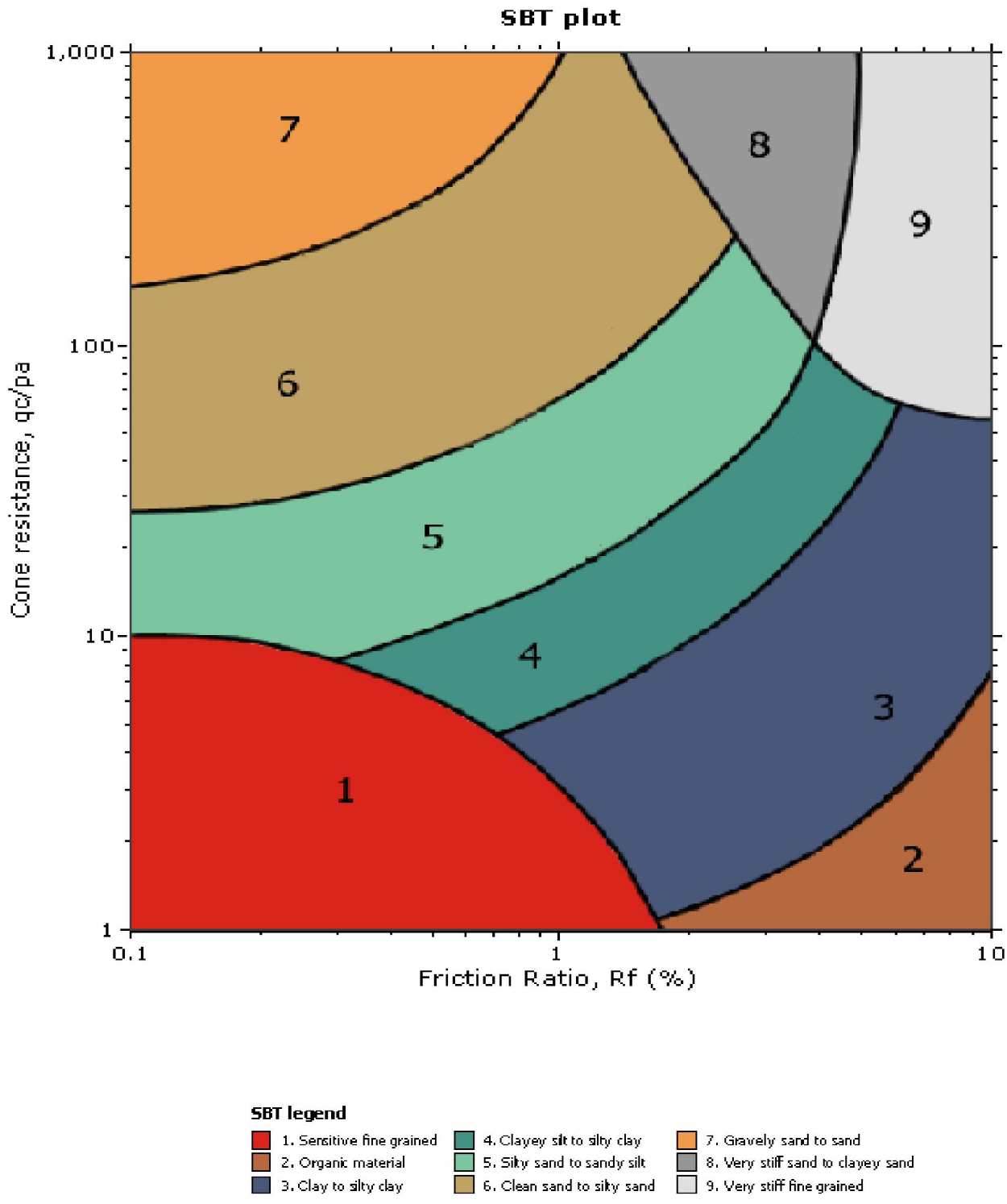


MTGL
RCC Cosmetology Geo Inv
Riverside, CA

CPT Shear Wave Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-1	4.99	3.99	4.46	4.28	1043	
	10.01	9.01	9.23	12.72	726	565
	14.99	13.99	14.13	17.24	820	1085
	20.01	19.01	19.11	22.04	867	1038
	25.00	24.00	24.08	26.32	915	1161
	28.87	27.87	27.94	28.90	967	1496
CPT-2	5.02	4.02	4.49	4.04	1111	
	10.01	9.01	9.23	12.06	765	591
	14.99	13.99	14.13	18.84	750	723
	20.01	19.01	19.11	24.40	783	896
	25.00	24.00	24.08	29.76	809	927
	29.99	28.99	29.06	32.64	890	1728
	35.01	34.01	34.07	34.96	975	2159
	37.53	36.53	36.58	36.18	1011	2062

Shear Wave Source Offset - 2 ft

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

APPENDIX C

LABORATORY TEST RESULTS

DRAFT

APPENDIX C

LABORATORY TESTING PROCEDURES

1. Particle Size Analysis

   Particle size analysis on representative soil samples were determined using the standard test method of the ASTM D6913.

2. Atterberg Limits

   The liquid limit, plastic limit, and the plasticity index of the major soil types encountered were determined using the standard test methods of the ASTM D4318.

3. Percent Finer

   Determining the amount of material finer than No. 200 sieve in soils by washing was performed on soil samples using the standard test methods of the ASTM D1140.

4. Expansion Index

   Expansion index of materials encountered was determined using the standard test methods of the ASTM D4829.

5. Maximum Density

   Maximum density tests were performed on a representative bag sample of the near surface soils in accordance with ASTM D1557.

6. Direct Shear

   Direct Shear Tests were performed on in-place samples of site soils in accordance with ASTM D3080.

8. Corrosion

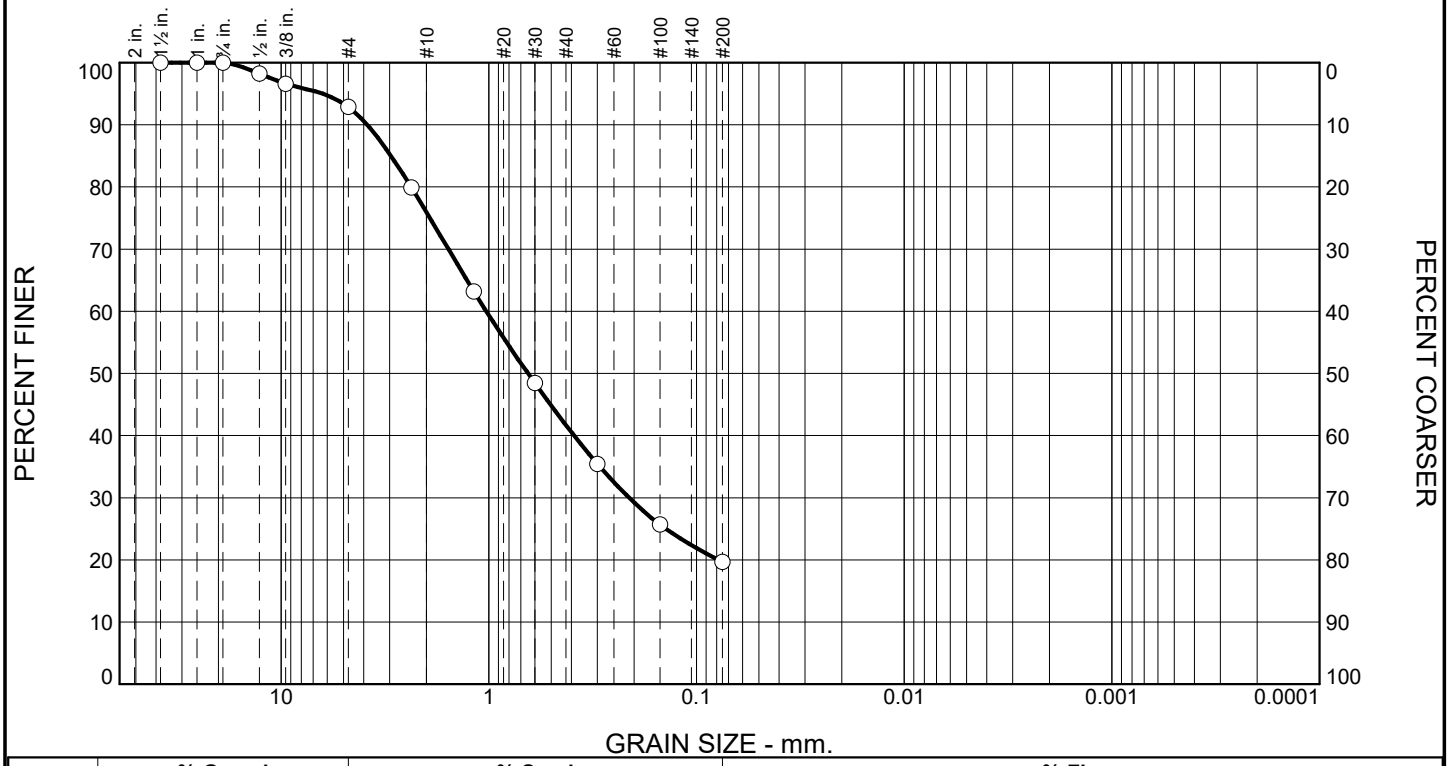
   Chemical testing was performed on representative samples to determine the corrosion potential of the onsite soils. Testing consisted of pH, chlorides (CTM 422), soluble sulfates (CTM 417), and resistivity (CTM 643).

9. Consolidation

   Testing for one-dimensional consolidation properties of soils were tested in general accordance with ASTM D2435

DRAFT

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	7	17	34	22	20	

Test Results (ASTM C 136 & ASTM C 117)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100		
1"	100		
3/4"	100		
1/2"	98		
3/8"	97		
#4	93		
#8	80		
#16	63		
#30	48		
#50	35		
#100	26		
#200	20		

Material Description

CLAYEY SAND (SC)
Light brown to reddish brown

Coefficients

D_{90}= 3.8539 D_{85}= 2.9591 D_{60}= 1.0289
D_{50}= 0.6475 D_{30}= 0.2112 D_{15}=
D_{10}= C_u= C_c=

Date Received: _____ **Date Tested:** 10/30/25

Tested By: J. Alvarez

Checked By: I. Chun

Title: PE.GE.

Location: B - 2 at 15 to 16 Feet
Sample Number: 25-7412

Date Sampled: 10/22/25

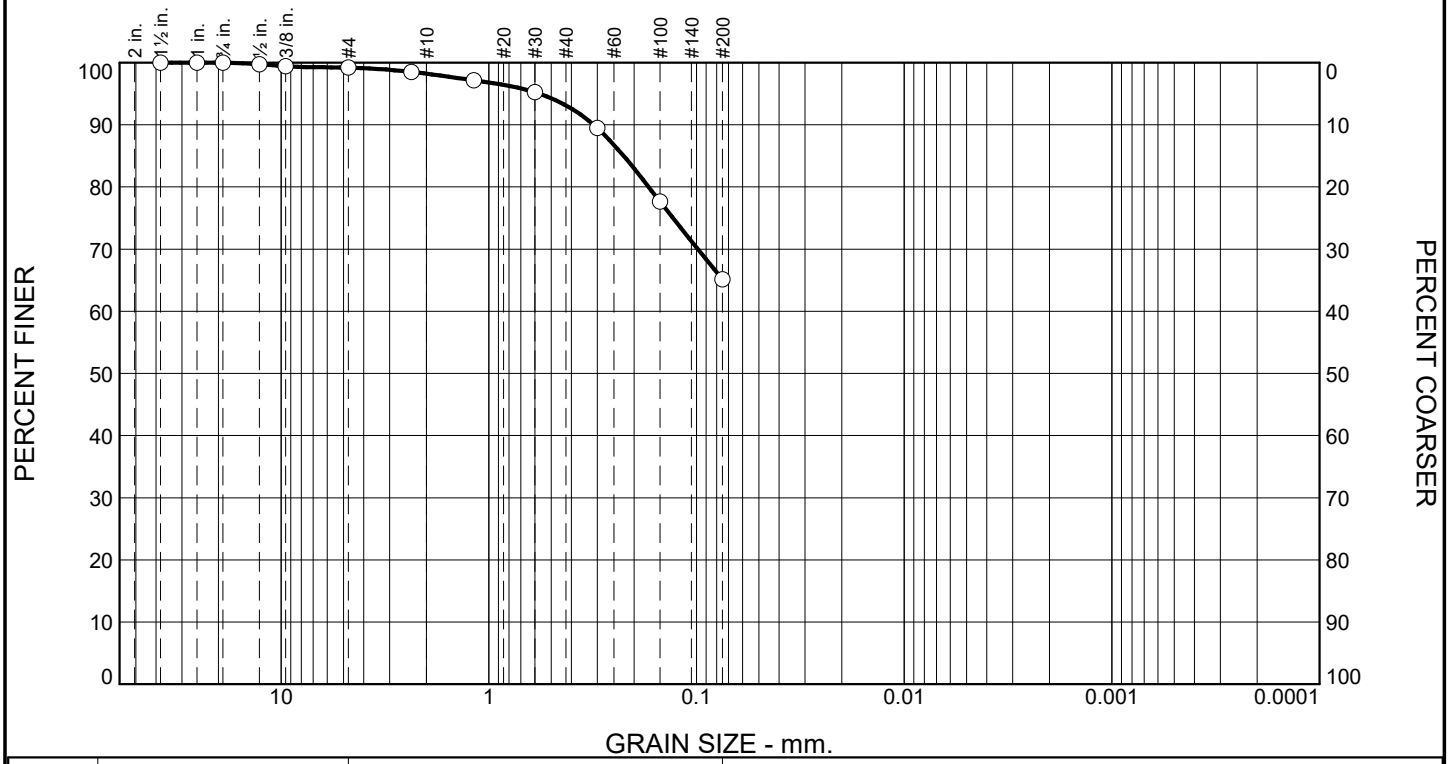
MTGL, Inc.

Anaheim, CA

Client: Riverside Community College District
Project: RCC Cosmetology GI

Project No: 8073A06 **Figure** C-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	1	1	5	28	65	

Test Results (ASTM C 136 & ASTM C 117)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100		
1"	100		
3/4"	100		
1/2"	100		
3/8"	99		
#4	99		
#8	99		
#16	97		
#30	95		
#50	89		
#100	78		
#200	65		

Material Description

SANDY SILTY CLAY (CL-ML)

Brown

Coefficients

D_{90}= 0.3129 D_{85}= 0.2244 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Date Received: _____ **Date Tested:** 10/30/25

Tested By: J. Alvarez

Checked By: I. Chun

Title: PE.GE.

Location: B - 5 at 2 to 3 Feet
Sample Number: 25-7412

Date Sampled: 10/22/25

MTGL, Inc.

Anaheim, CA

Client: Riverside Community College District
Project: RCC Cosmetology GI

Project No: 8073A06 **Figure** C-2

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT

ASTM D1557

SAMPLE	DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-3 at 2 to 4 Feet	SANDY SILTY CLAY (CL-ML)	121.7	10.2

EXPANSION INDEX

ASTM D4829

SAMPLE	DESCRIPTION	EXPANSION INDEX
B-2 at 2 to 5 Feet	SILTY SAND	29

Classification of Expansive Soil[1]

Expansion Index	Expansion Potential
1-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

1. ASTM - D4829

RESISTIVITY, pH, SOLUBLE CHLORIDE and SOLUBLE SULFATE

Resistivity (CT.643), Soluble Sulfates (CT.417) Soluble Chlorides (CT.442)

SAMPLE	pH	RESISTIVITY (Ω-cm)	SULFATE (ppm)	CHLORIDE (ppm)
B-2 at 2 to 5 Feet	7.6	5,400	102	34

ATTERBERG LIMITS

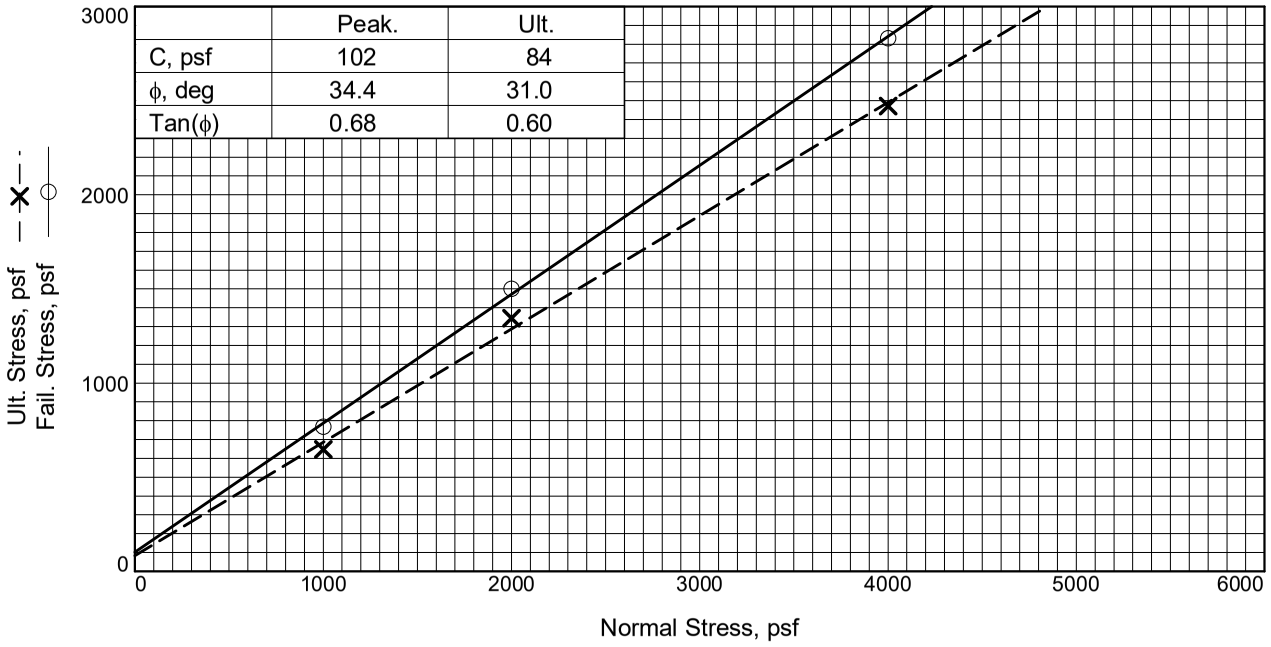
ASTM D4318

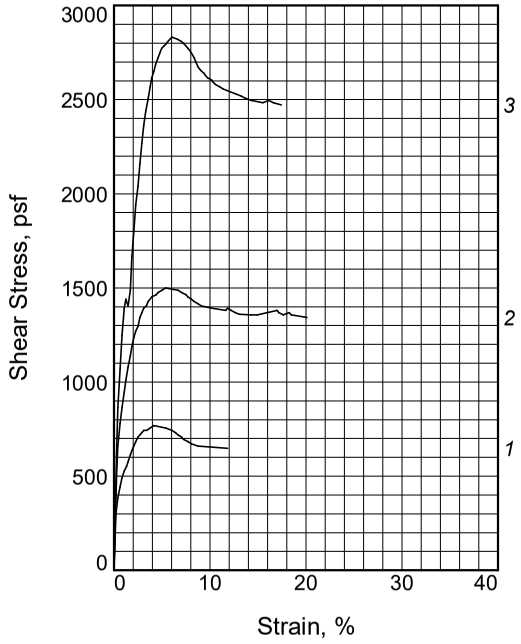
SAMPLE	DESCRIPTION	Liquid Limit	Plastic Limit	Plastic Index
B-2 at 2 to 5 Feet	SILTY SAND (SM)	21	18	3
B-4 at 8 to 9 Feet	SANDY SILTY CLAY (CL-ML)	21	15	6

PERCENT FINER

ASTM D1140

SAMPLE	DESCRIPTION	% Finer
B-2 at 2 to 5 Feet	SILTY SAND (SM)	46.4
B-3 at 2 to 4 Feet	SANDY SILTY CLAY (CL-ML)	58.7
B-3 at 4 to 5 Feet	SILTY CLAYEY SAND (SC-SM)	39.5
B-4 at 2 to 4 Feet	SILTY CLAYEY SAND (SC-SM)	47.0
B-4 at 8 to 9 Feet	SANDY SILTY CLAY (CL-ML)	65.3
B-4 at 11 to 11½ Feet	SILTY CLAYEY SAND (SC-SM)	31.9
B-4 at 16 to 16½ Feet	SANDY SILTY CLAY (CL-ML)	53.4
B-4 at 18 to 19 Feet	SILTY SAND (SM)	12.8
B-4 at 21 to 21½ Feet	Poorly Graded SAND with CLAY (SP-SC)	8.1
B-10 at 10 to 10½ Feet	SANDY SILT (ML)	50.1





Sample No.		1	2	3
Initial	Water Content, %	13.1	13.1	13.1
	Dry Density, pcf	113.3	111.8	113.8
	Saturation, %	72.2	69.5	73.2
	Void Ratio			
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	17.3	17.5	16.0
	Dry Density, pcf	113.7	112.6	114.8
	Saturation, %	97.0	95.2	92.4
	Void Ratio			
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	0.99	0.99
Normal Stress, psf		1000	2000	4000
Fail. Stress, psf		768	1500	2832
Strain, %		4.2	5.4	6.0
Ult. Stress, psf		648	1344	2472
Strain, %		11.8	20.1	17.4
Strain at peak, %		4.2	5.4	6.0

Sample Type:
Mod. CAL (Ring)

Material Description:
Young Axial Channel Deposits (Qya_a)
CLAYEY SAND (SC), Brown

Client: Riverside Community College District

Project: Riverside City College Cosmetology Bldg.

Location: B-2 at 8½ to 9 Feet

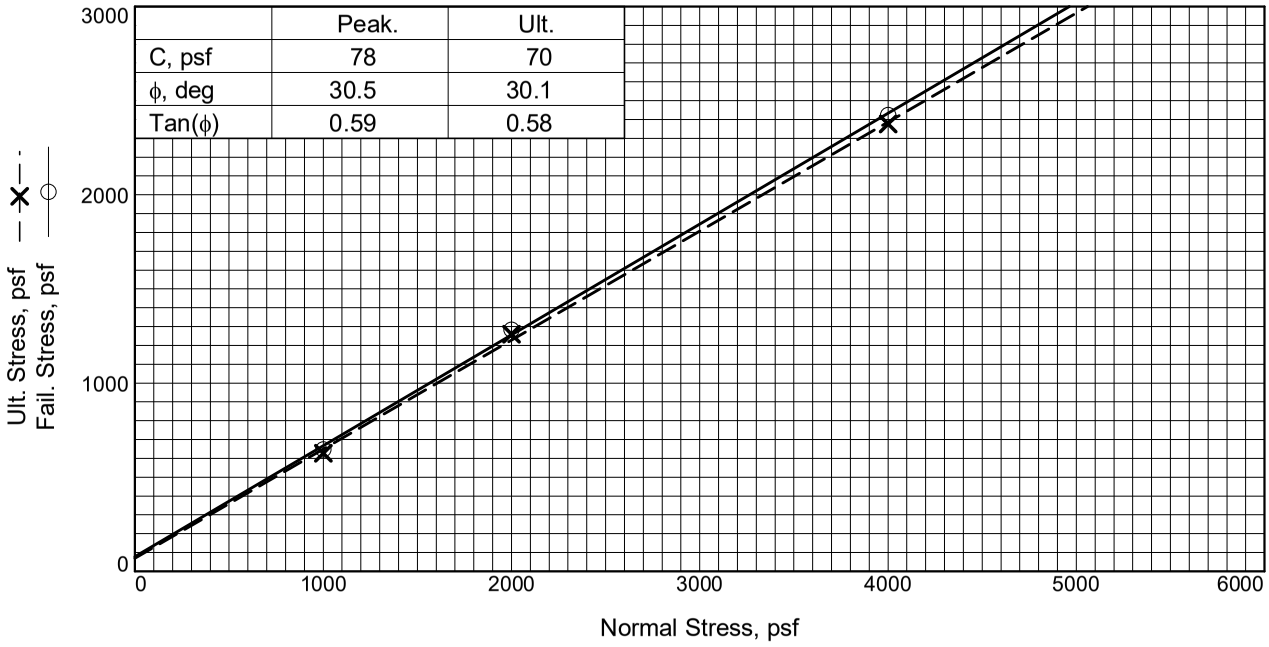
Sample Number: 25-7412

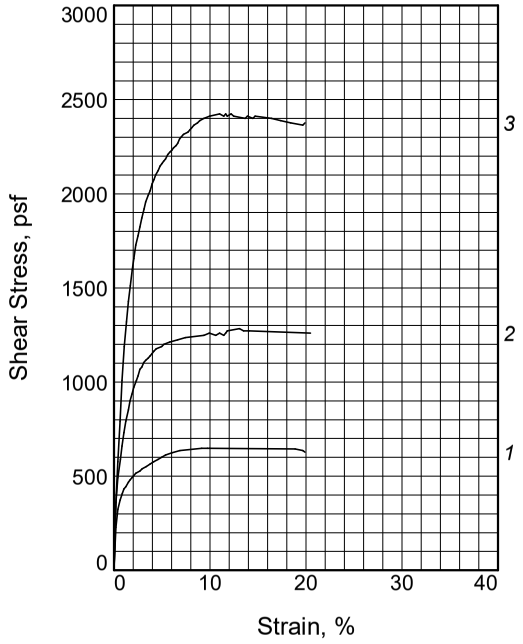
Proj. No.: 8073A06 **Date Sampled:** 10/22/25

DIRECT SHEAR TEST REPORT
MTGL, Inc.
Anaheim, CA

Tested By: J. Alvarez

Figure: C-4





Sample No.		1	2	3
Initial	Water Content, %	9.2	9.2	9.2
	Dry Density, pcf	109.5	109.7	109.9
	Saturation, %	46.1	46.3	46.5
	Void Ratio			
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	20.1	18.6	17.3
	Dry Density, pcf	108.8	109.0	109.5
	Saturation, %	98.5	92.1	86.7
	Void Ratio			
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.01	1.01	1.00
Normal Stress, psf		1000	2000	4000
Fail. Stress, psf		648	1284	2424
Strain, %		9.1	13.1	11.0
Ult. Stress, psf		628	1260	2376
Strain, %		19.9	20.5	19.9
Strain at peak, %		9.1	13.1	11.0

Sample Type:
Bulk (Remolded 90% Rel. Comp. ASTM D1557)
Description:
Compacted Fill (Qfc)
SANDY SILTY CLAY (CL-ML), Brown

Client: Riverside Community College District

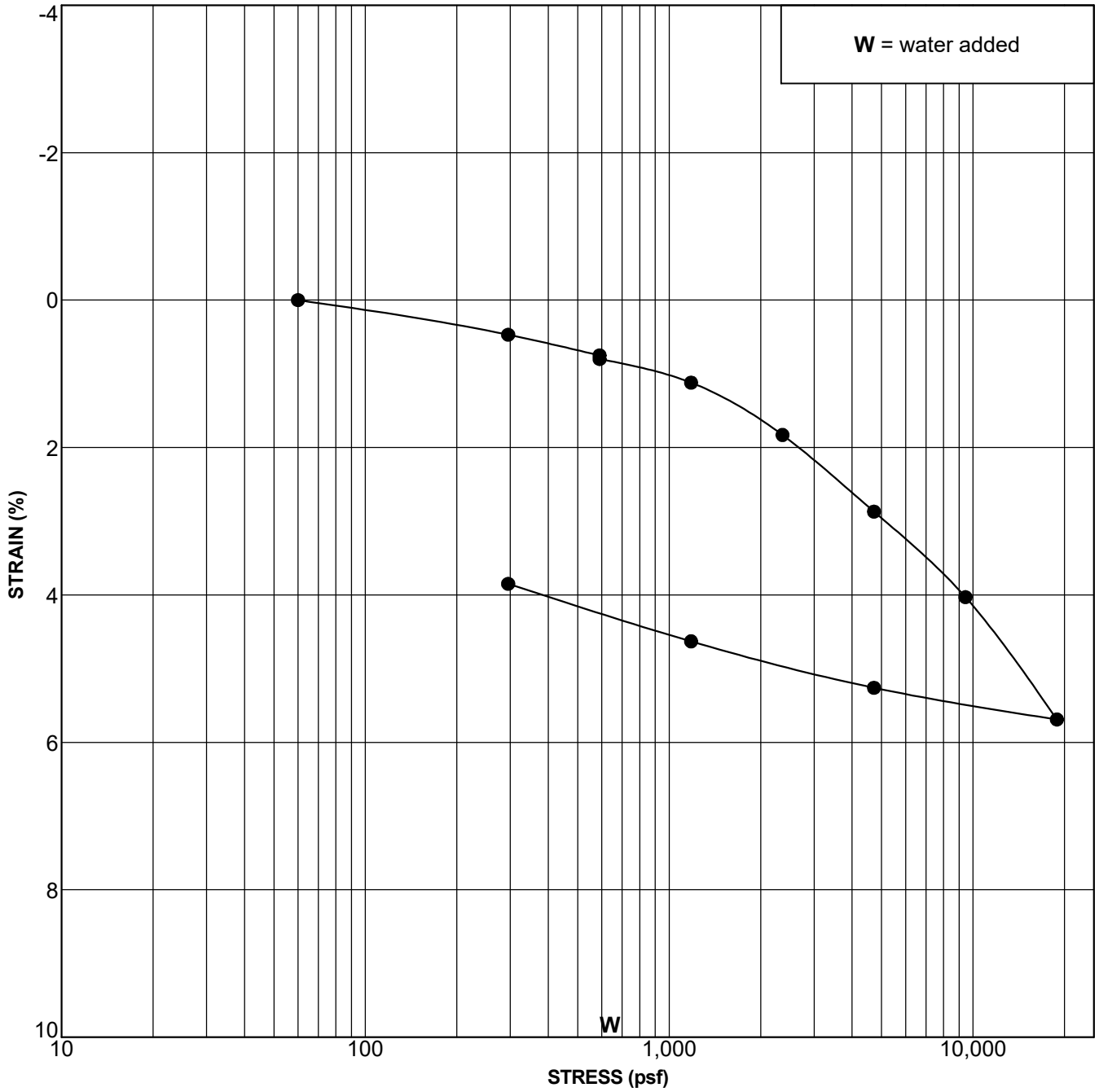
Project: Riverside City College Cosmetology Bldg.

Location: B-3 at 2 to 4 Feet
Sample Number: 25-7412
Proj. No.: 8073A06 **Date Sampled:** 10/22/25

DIRECT SHEAR TEST REPORT
MTGL, Inc.
Anaheim, CA

Tested By: J. Alvarez

Figure: C-5



Boring Number	Depth (feet)	Geologic Unit	Symbol	In Situ or Remolded Sample	% Hydro-Collapse	Classification
B-5	6.0		●	In Situ	0.05	SILTY SAND (SM)

CONSOLIDATION TEST DATA

Project: MTGL New Cosmetology Building Riverside CA

Project No. LA.112.25

GMU

GMU_CONSOL_LA.112.25 MTGL.GPJ GM&U.GDT 11/10/25

TABLE B-1
SUMMARY OF SOIL LABORATORY DATA

Sample Information			Geologic Unit	USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	In Situ Saturation, %	Sieve/Hydrometer				Atterberg Limits			Compaction		Expansion Index	R-Value	Chemical Test Results			
Boring Number	Depth, feet	Elevation, feet						Gravel, %	Sand, %	<#200, %	<2μ, %	LL	PL	PI	Maximum Dry Unit Weight, pcf	Optimum Water Content, %			pH	Sulfate (ppm)	Chloride (ppm)	Min. Resistivity (ohm/cm)
B-5	6	NA		SM	11.2	101	46															

Project: MTGL New Cosmetology Building Riverside CA
Project No. LA.112.25

APPENDIX D

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

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APPENDIX D

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

GENERAL

These specifications present general procedures and requirements for grading and earthwork as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, and excavations. The recommendations contained in the attached geotechnical report are a part of the earthwork and grading specifications and shall supersede the provisions contained herein in the case of conflict. Evaluations performed by the Consultant during the course of grading may result in new recommendations, which could supersede these specifications, or the recommendations of the geotechnical report.

EARTHWORK OBSERVATION AND TESTING

Prior to the start of grading, a qualified Geotechnical Consultant (Geotechnical Engineer and Engineering Geologist) shall be employed for the purpose of observing earthwork procedures and testing the fills for conformance with the recommendations of the geotechnical report and these specifications. It will be necessary that the Consultant provide adequate testing and observation so that he may determine that the work was accomplished as specified. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that he may schedule his personnel accordingly.

It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications, and the approved grading plans.

Maximum dry density tests used to determine the degree of compaction will be performed in accordance with the American Society for Testing and Materials Test Method (ASTM) D1557.

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PREPARATION OF AREAS TO BE FILLED

Clearing and Grubbing: All brush, vegetation and debris shall be removed or piled and otherwise disposed of.

Processing: The existing ground which is determined to be satisfactory for support of fill shall be scarified to a minimum depth of 8 inches. Existing ground, which is not satisfactory, shall be over-excavated as specified in the following section.
Over-excavation: Soft, dry, spongy, highly fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, shall be over-excavated down to firm ground, approved by the Consultant.

Moisture conditioning: Over-excavated and processed soils shall be watered, dried-back, blended, and mixed as required to have a relatively uniform moisture content near the optimum moisture content as determined by ASTM D1557.

Re-compaction: Over-excavated and processed soils, which have been mixed, and moisture conditioned uniformly shall be recompacted to a minimum relative compaction of 90% of ASTM D1557.

Benching: Where soils are placed on ground with slopes steeper than 5:1 (horizontal to vertical), the ground shall be stepped or benched. Benches shall be excavated in firm material for a minimum width of 4 feet.

FILL MATERIAL

General: Material to be placed as fill shall be free of organic matter and other deleterious substances and shall be approved by the Consultant.

Oversize: Oversized material defined as rock, or other irreducible material with a maximum dimension greater than 6 inches, shall not be buried or placed in fill, unless the location, material, and disposal methods are specifically approved by the Consultant. Oversize disposal operations shall be such that nesting of oversized material does not occur, and such that the oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet vertically of finish grade or within the range of future utilities or underground construction, unless specifically approved by the Consultant.

Import: If importing of fill material is required for grading, the import material shall meet the general requirements.

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FILL PLACEMENT AND COMPACTION

Fill Lifts: Approved fill material shall be placed in areas prepared to receive fill in near-horizontal layers not exceeding 8 inches in compacted thickness. The Consultant may approve thicker lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to attain uniformity of material and moisture in each layer.

Fill Moisture: Fill layers at a moisture content less than optimum shall be watered and mixed, and wet fill layers shall be aerated by scarification or shall be blended with drier material. Moisture conditioning and mixing of fill layers shall continue until the fill material is at uniform moisture content at or near optimum.

Compaction of Fill: After each layer has been evenly spread, moisture conditioned, and mixed, it shall be uniformly compacted to not less than 90% of maximum dry density in accordance with ASTM D1557. Compaction equipment shall be adequately sized and shall be either specifically designed for soil compaction or of proven reliability, to efficiently achieve the specified degree of compaction.

Fill Slopes: Compacting on slopes shall be accomplished, in addition to normal compacting procedures, by backrolling of slopes with sheepsfoot rollers at frequent increments of 2 to 3 feet as the fill is placed, or by other methods producing satisfactory results. At the completion of grading, the relative compaction of the slope out to the slope face shall be at least 90% in accordance with ASTM D1557.

Compaction Testing: Field tests to check the fill moisture and degree of compaction will be performed by the consultant. The location and frequency of tests shall be at the consultant's discretion. In general, these tests will be taken at an interval not exceeding 2 feet in vertical rise, and/or 1,000 cubic yards of fill placed. In addition, on slope faces, at least one test shall be taken for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope.

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SUBDRAIN INSTALLATION

Subdrain systems, if required, shall be installed in approved ground to conform to the approximate alignment and details shown on the plans or herein. The subdrain location or materials shall not be changed or modified without the approval of the Consultant. The Consultant, however, may recommend and, upon approval, direct changes in subdrain line, grade, or materials. All subdrains should be surveyed for line and grade after installation and sufficient time shall be allowed for the surveys, prior to commencement of fill over the subdrain.

EXCAVATION

Excavations and cut slopes will be examined during grading. If directed by the Consultant, further excavation or over-excavation and refilling of cut areas, and/or remedial grading of cut slopes shall be performed. Where fill over cut slopes are to be graded, unless otherwise approved, the cut portion of the slope shall be made and approved by the Consultant prior to placement of materials for construction of the fill portion of the slope.

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APPENDIX E

BOREHOLE PERCOLATION TEST RESULTS

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PERCOLATION TEST DATA

Project:	RCC New Cosmetology Building			**Project No:**	8073A06		
TEST INFORMATION							
Test Hole No.:	P-1	**Tested by:**	NA/IC	**Date:**	10/23/2025		
Depth of Hole (in), D_T:	60	**USCS Soil Classification:**	CLAYEY SAND (SC)	**Gravel Pack:**	Y	**Size:**	3/4 inch

Test Hole Dimensions (in):	**Diameter**	**Length**	**Width**
	8	N/A	N/A

SANDY SOIL CRITERIA*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in)	Final Depth to Water (in)	Change in Water Level (in)	Greater than or Equal to 6" (y/n)	Pass/Fail
1	9:53 AM	10:18 AM	25	42.0	57.0	15.0	Yes	Pass
2	10:20 AM	10:45 AM	25	42.0	55.2	13.2	Yes	Pass

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

TEST DATA

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_o Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)	Tested Infiltration Rate (in/hr)	*Reliable Infiltration Rate (in/hr)	**Gravel-Adjusted Reliable Infiltration Rate (in/hr)
1	10:45 AM	10:55 AM	10	41.4	48.0	6.6	1.52	4.58	2.29	**0.71**
2	10:55 AM	11:05 AM	10	41.4	46.8	5.4	1.85	3.62	1.81	**0.56**
3	11:05 AM	11:15 AM	10	43.2	48.0	4.8	2.08	3.51	1.76	**0.55**
4	11:15 AM	11:25 AM	10	41.4	48.0	6.6	1.52	4.58	2.29	**0.71**
5	11:25 AM	11:35 AM	10	43.2	48.0	4.8	2.08	3.51	1.76	**0.55**
6	11:35 AM	11:45 AM	10	40.8	47.4	6.6	1.52	4.42	2.21	**0.69**
7	11:45 AM	11:55 AM	10	39.6	45.6	6.0	1.67	3.71	1.86	**0.58**
8										
9										
10										
11										
12										
13										
14										
15										

*The reliable infiltration rate is calculated by applying a factor of safety of 2 to the tested infiltration rate.

**The gravel-adjusted infiltration rate is the reliable infiltration rate mutliplied by a gravel pack porosity factor.

Note: Infiltration Rate Calculated by Porchet Method

$$I_t = \frac{\Delta H\,(60r)}{\Delta t\,(r + 2H_{avg})}$$

Where:

- ΔH = Change in height over time
- Δt = Time interval
- r = Test hole radius
- H_{avg} = Average head over time interval
- I_t = Tested infiltration rate

PERCOLATION TEST DATA

Project:	RCC New Cosmetology Building			**Project No:**	8073A06		
TEST INFORMATION							
Test Hole No.:	P-2	**Tested by:**	NA/IC	**Date:**	10/23/2025		
Depth of Hole (in), D_T:	120	**USCS Soil Classification:**	CLAYEY SAND (SC)	**Gravel Pack:**	Y	**Size:**	3/4 inch

Test Hole Dimensions (in):	**Diameter**	**Length**	**Width**
	8	N/A	N/A

SANDY SOIL CRITERIA*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in)	Final Depth to Water (in)	Change in Water Level (in)	Greater than or Equal to 6" (y/n)	Pass/Fail
1	9:48 AM	10:13 AM	25	94.8	97.8	3.0	No	Fail
2	10:13 AM	10:38 AM	25	95.4	98.4	3.0	No	Fail

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurement sper hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

TEST DATA

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_o Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)	Tested Infiltration Rate (in/hr)	*Reliable Infiltration Rate (in/hr)	**Gravel-Adjusted Reliable Infiltration Rate (in/hr)
1	10:38 AM	11:08 AM	30	94.3	96.6	2.3	13.04	0.35	0.17	**0.05**
2	11:08 AM	11:38 AM	30	93.6	96.0	2.4	12.50	0.35	0.18	**0.05**
3	11:38 AM	12:08 PM	30	93.6	96.0	2.4	12.50	0.35	0.18	**0.05**
4	12:08 PM	12:38 PM	30	93.0	95.4	2.4	12.50	0.35	0.17	**0.05**
5	12:38 PM	1:08 PM	30	92.4	94.8	2.4	12.50	0.34	0.17	**0.05**
6	1:08 PM	1:38 PM	30	94.8	97.2	2.4	12.50	0.37	0.18	**0.06**
7	1:38 PM	2:08 PM	30	93.6	96.6	3.0	10.00	0.45	0.22	**0.07**
8	2:08 PM	2:38 PM	30	93.0	95.6	2.6	11.54	0.38	0.19	**0.06**
9	2:38 PM	3:08 PM	30	93.6	96.6	3.0	10.00	0.45	0.22	**0.07**
10	3:08 PM	3:38 PM	30	93.6	96.0	2.4	12.50	0.35	0.18	**0.05**
11	3:38 PM	4:08 PM	30	93.0	96.6	3.6	8.33	0.53	0.26	**0.08**
12	4:08 PM	4:38 PM	30	93.6	96.6	3.0	10.00	0.45	0.22	**0.07**
13										
14										
15										

*The reliable infiltration rate is calculated by applying a factor of safety of 2 to the tested infiltration rate.

**The gravel-adjusted infiltration rate is the reliable infiltration rate mutliplied by a gravel pack porosity factor.

Note: Infiltration Rate Calculated by Porchet Method

$$I_t = \frac{\Delta H\,(60r)}{\Delta t\,(r + 2H_{avg})}$$

Where:

- ΔH = Change in height over time
- Δt = Time interval
- r = Test hole radius
- H_{avg} = Average head over time interval
- I_t = Tested infiltration rate

APPENDIX F

LIQUEFACTION ANALYSIS RESULTS

DRAFT

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Lateral displacements summary report 10

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Transition layer aglorithm summary report 18
Vertical settlements summary report 19
Lateral displacements summary report 20

MTGL, Inc.
2992 E. La Palma Ave.
Anaheim, CA 92806
www.mtlginc.com

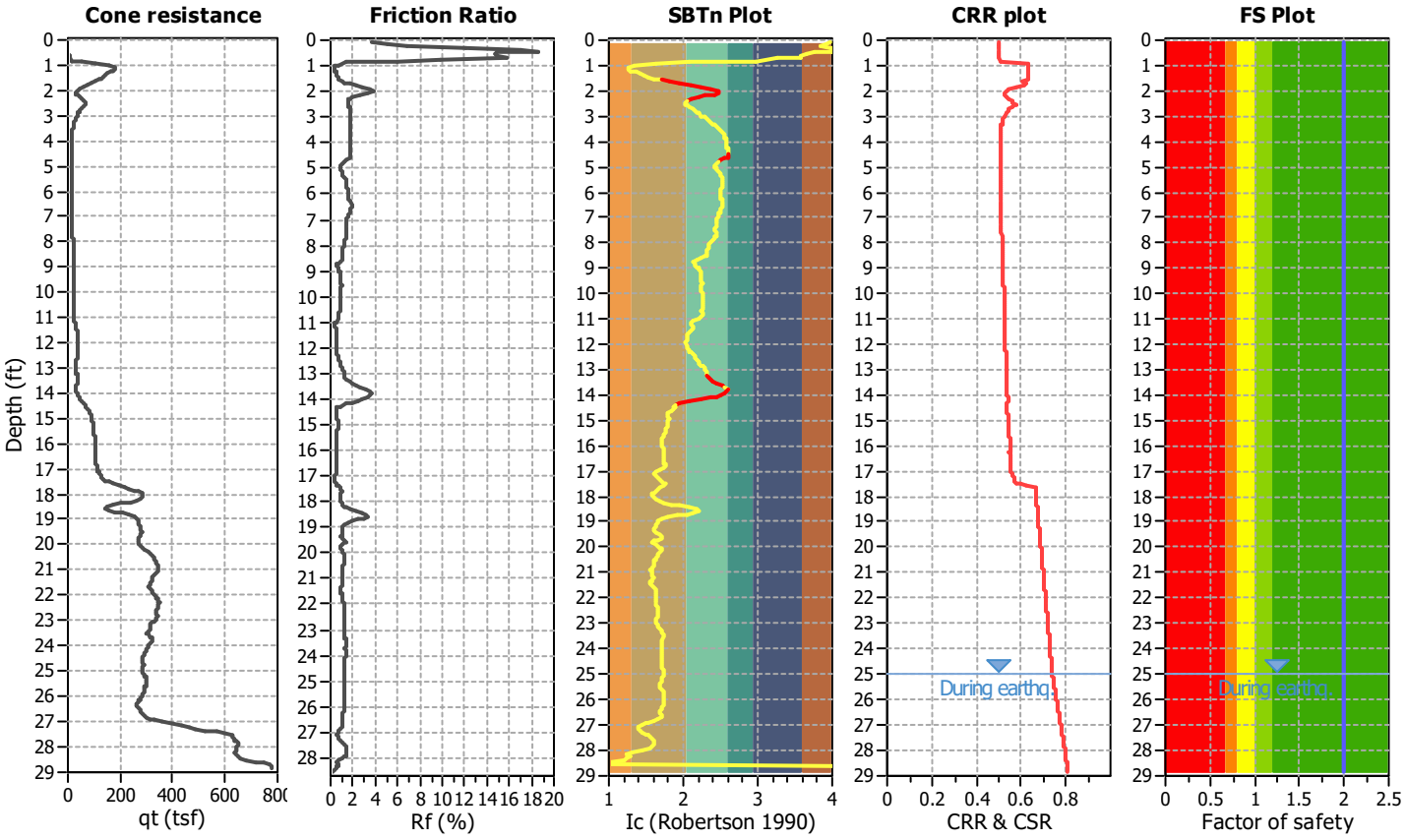
LIQUEFACTION ANALYSIS REPORT

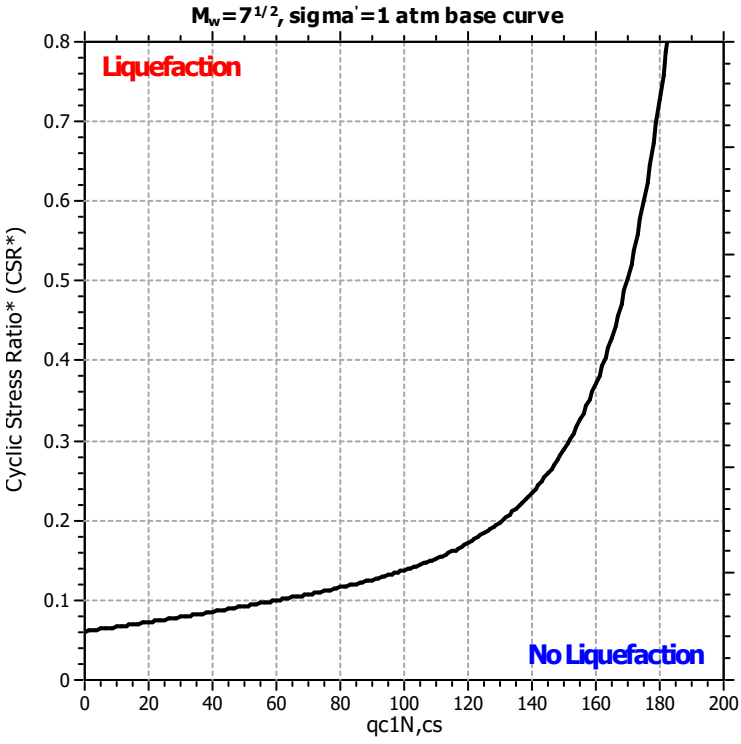
Project title : Riverside City College Cosmetology Building **Location : Riverside City College, Riverside, CA**

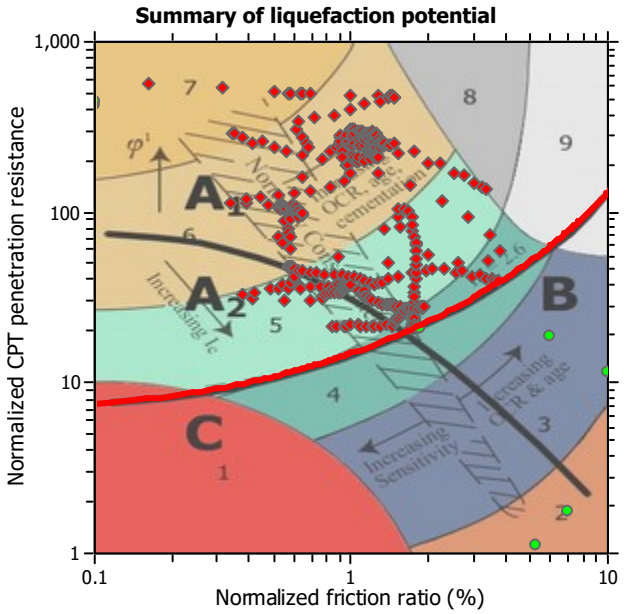
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	50.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	25.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

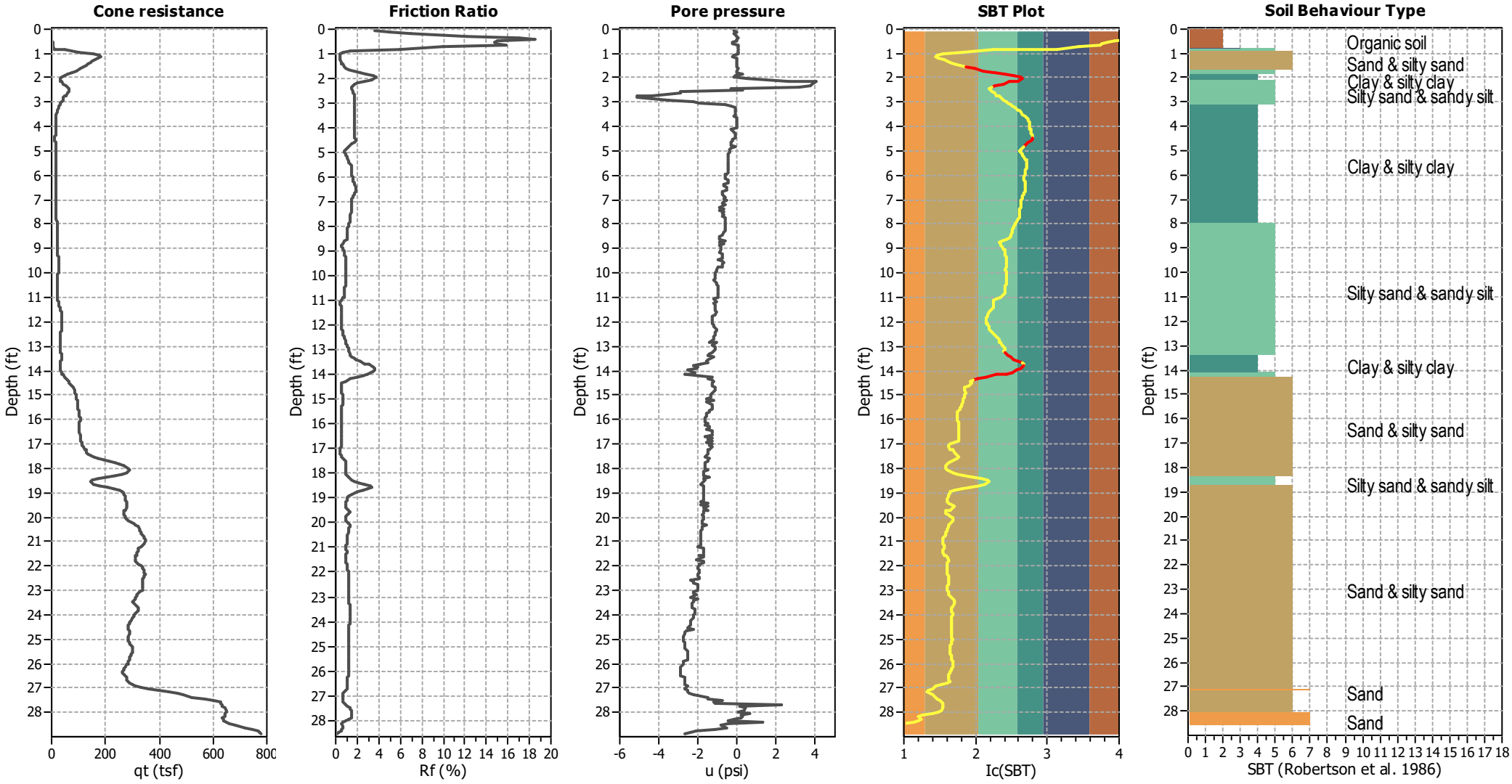






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



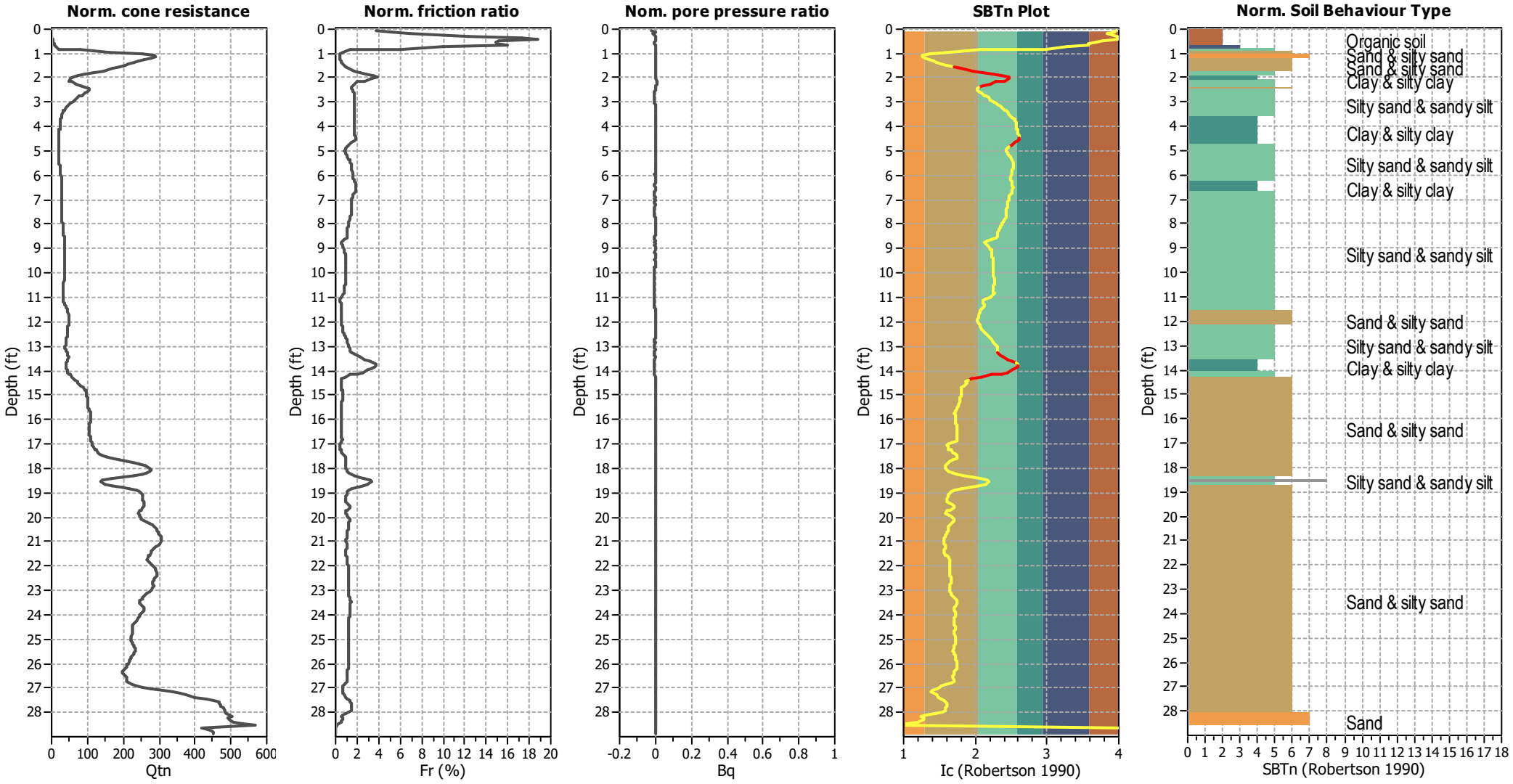
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

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)



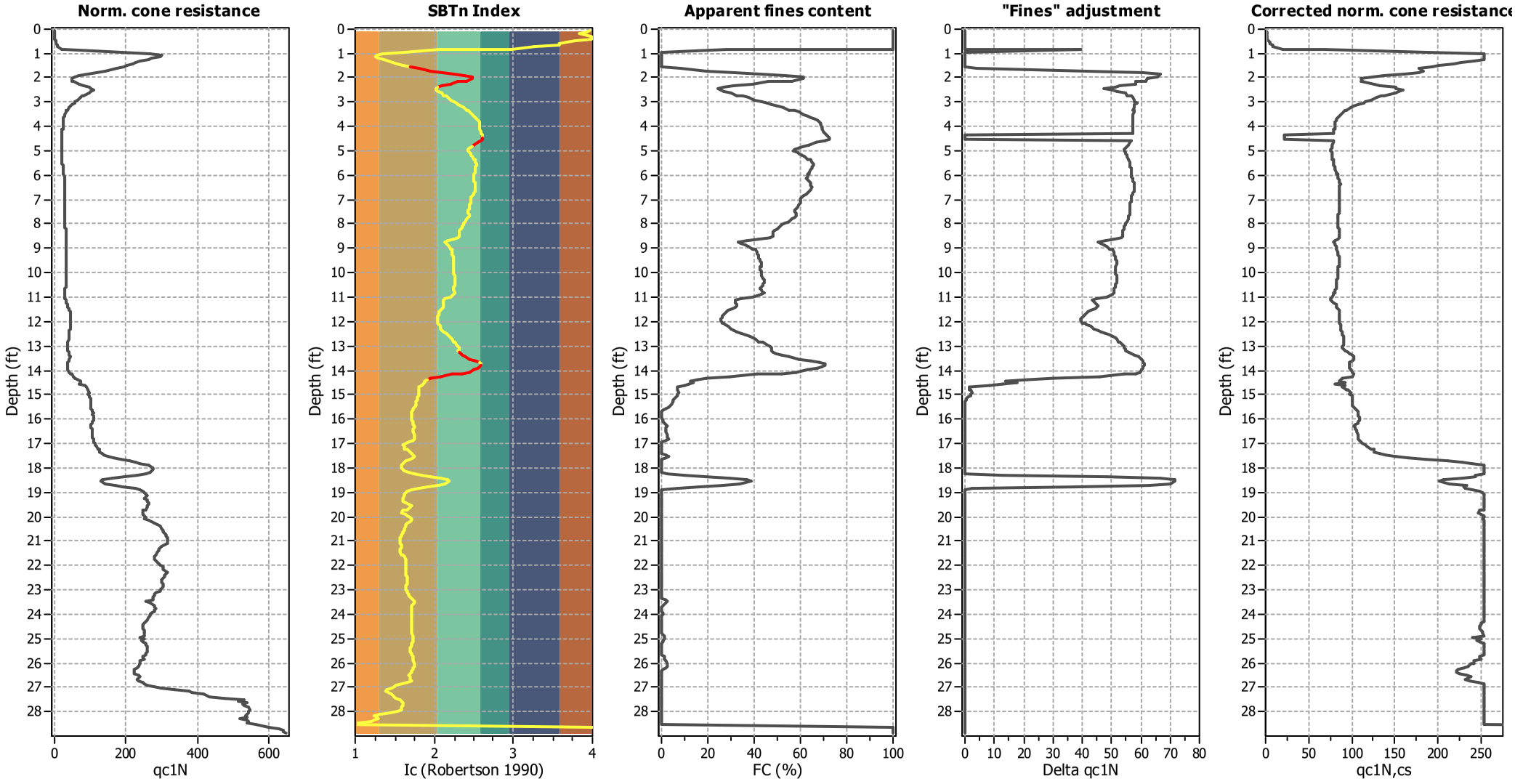
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

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

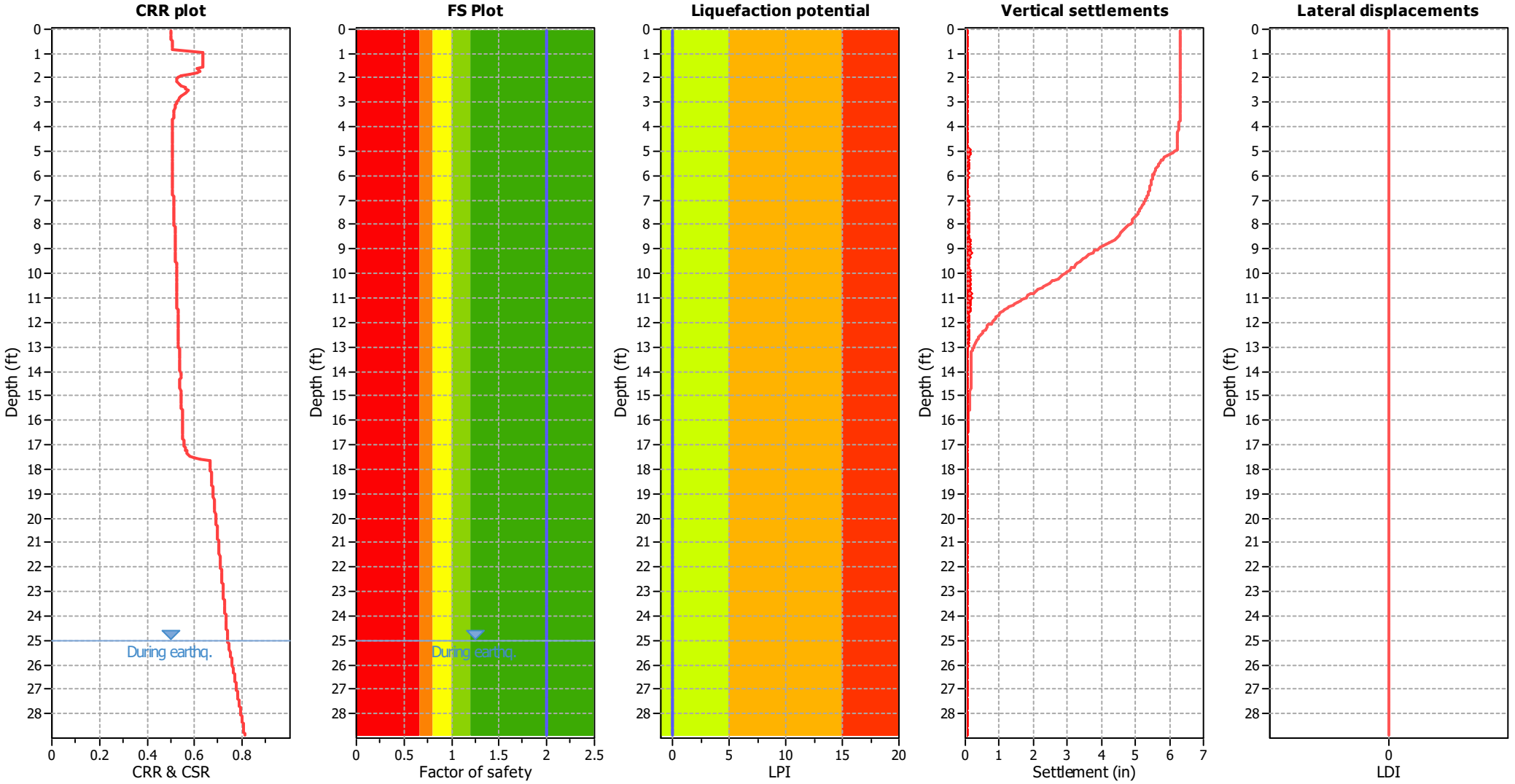
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

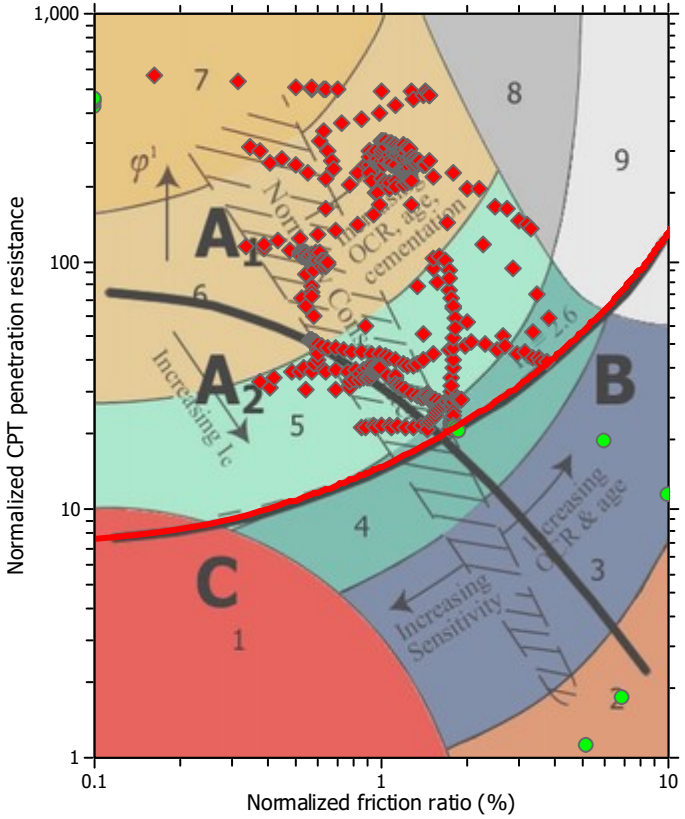
F.S. color scheme

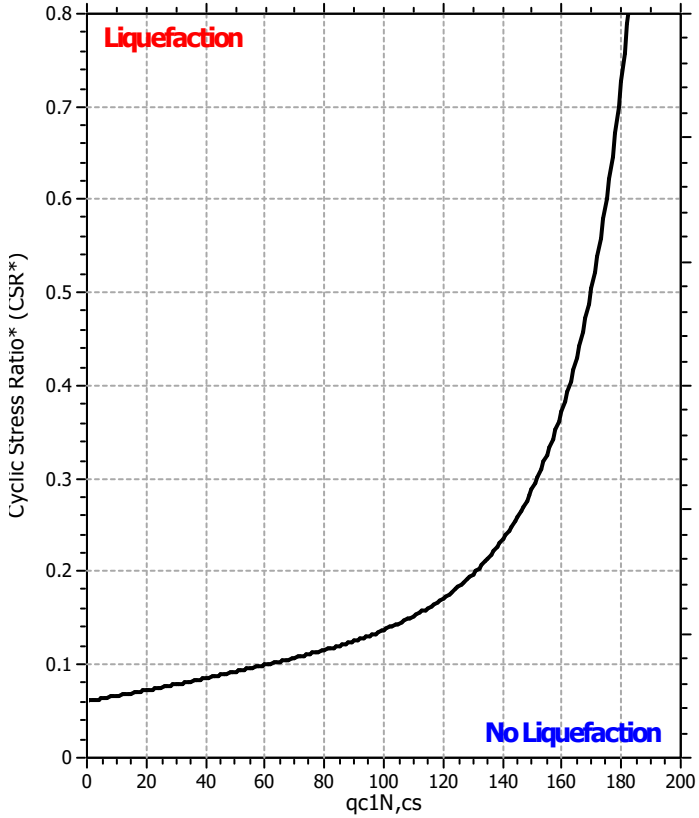
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

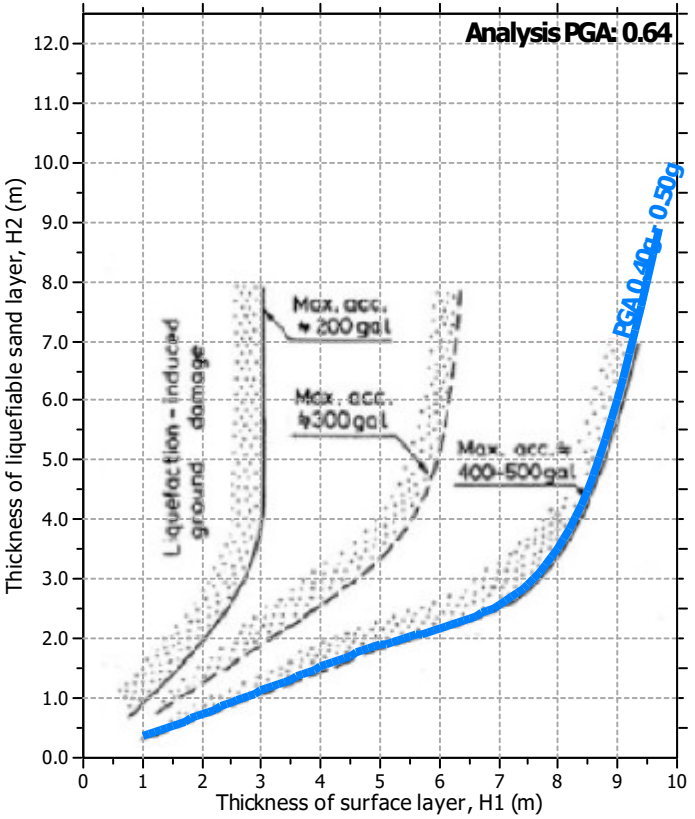
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



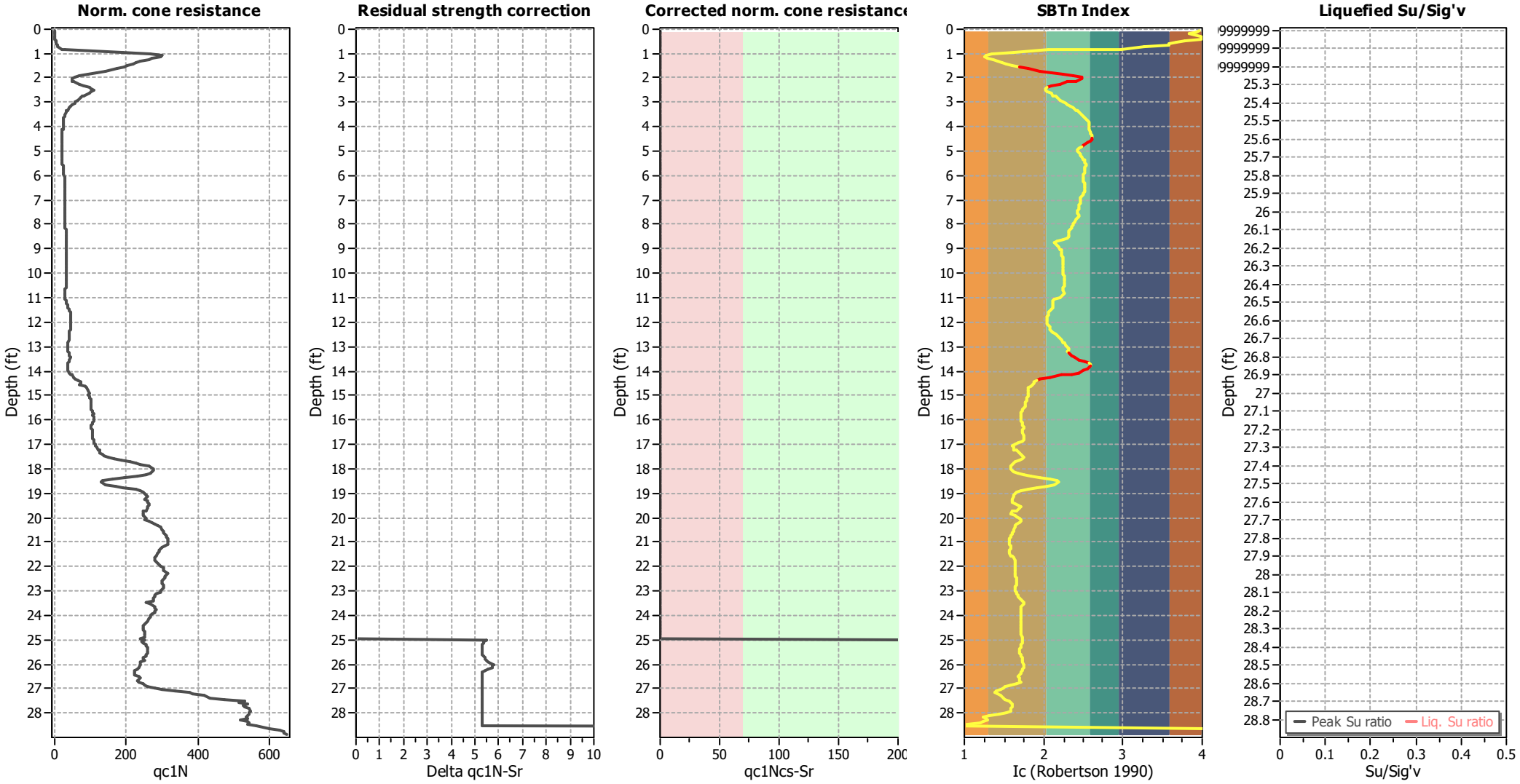




Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

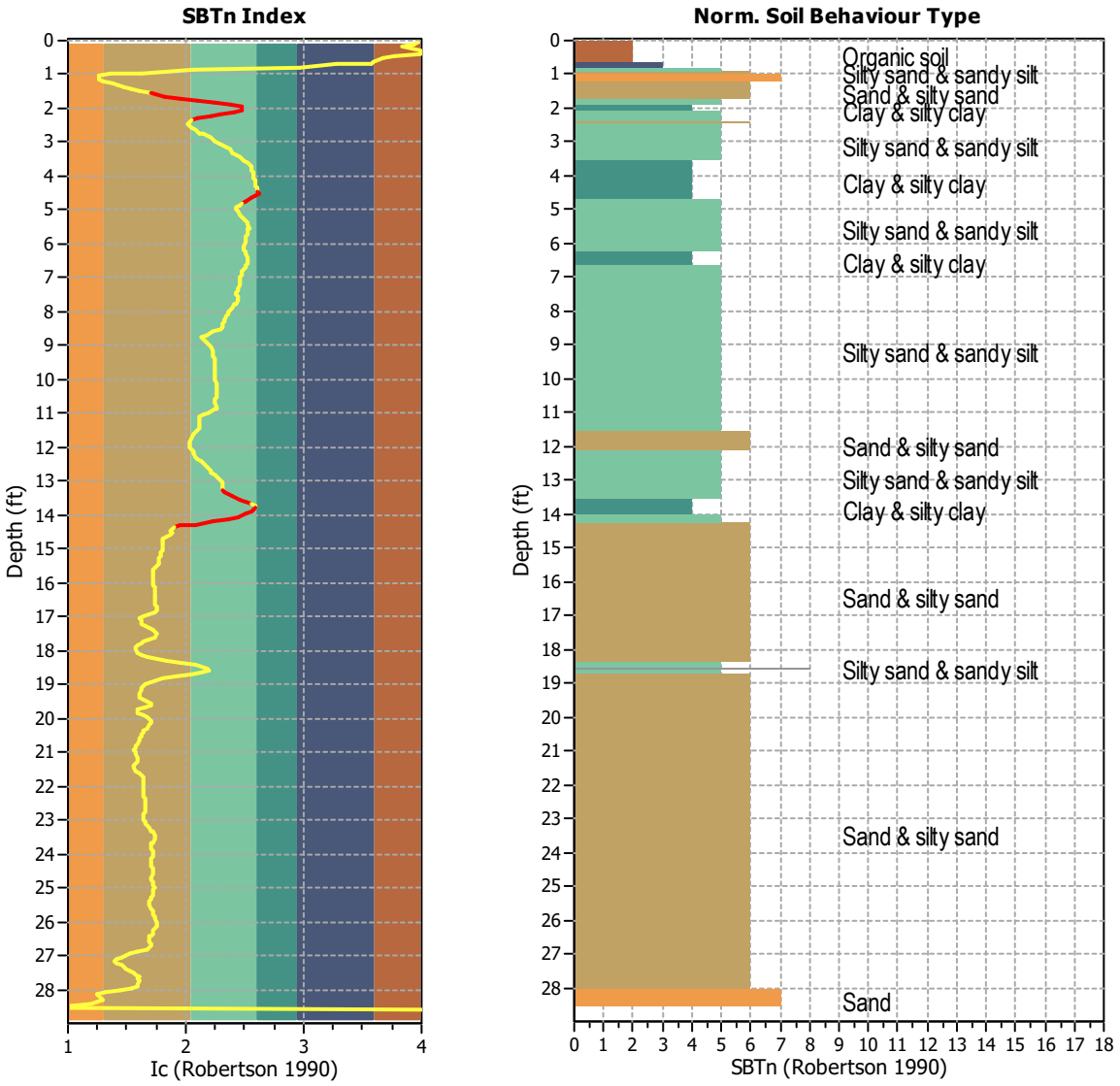
Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

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.



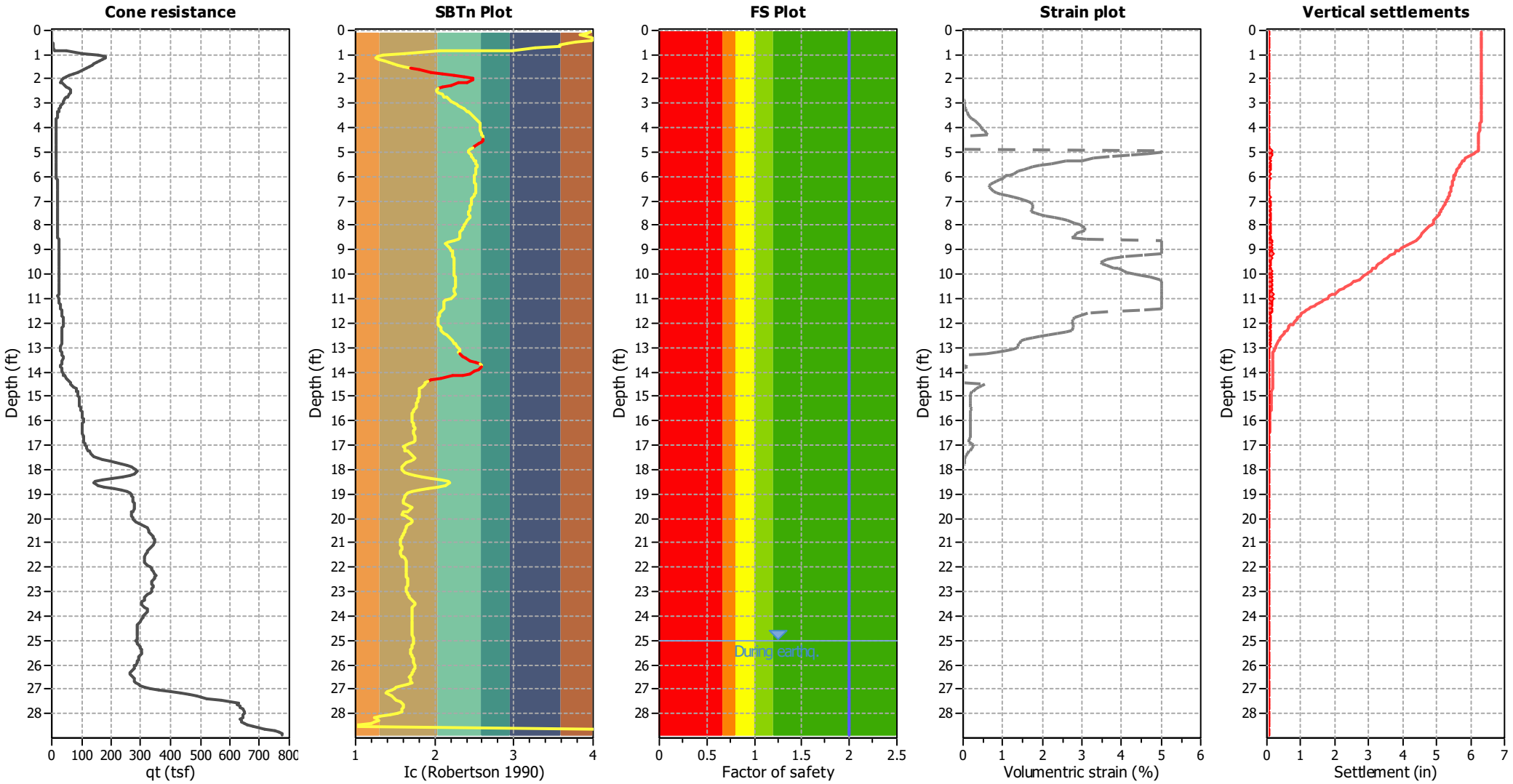
Transition layer algorithm properties

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:	440
Total points excluded:	36
Exclusion percentage:	8.18%
Number of layers detected:	5

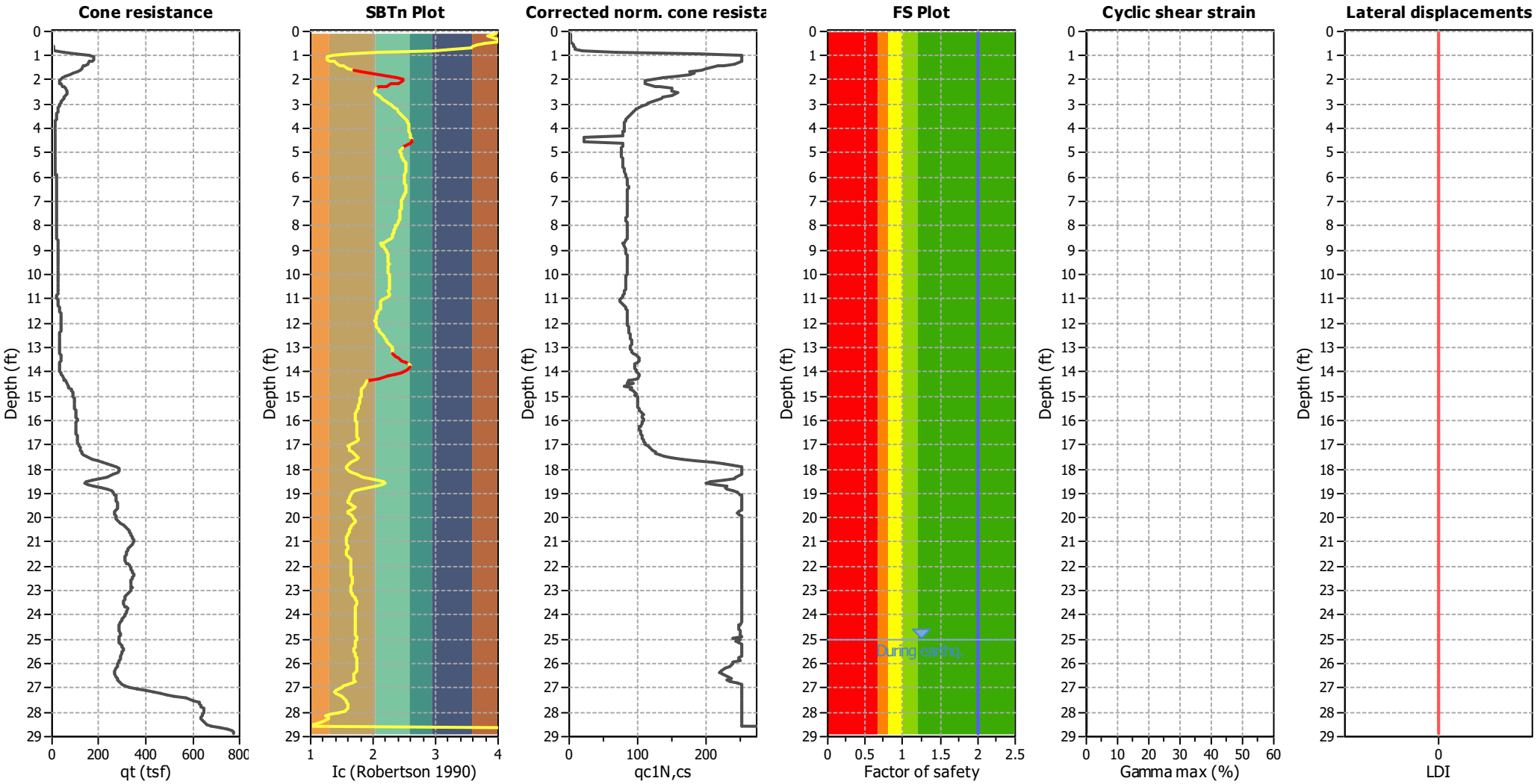
Estimation of post-earthquake settlements



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

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

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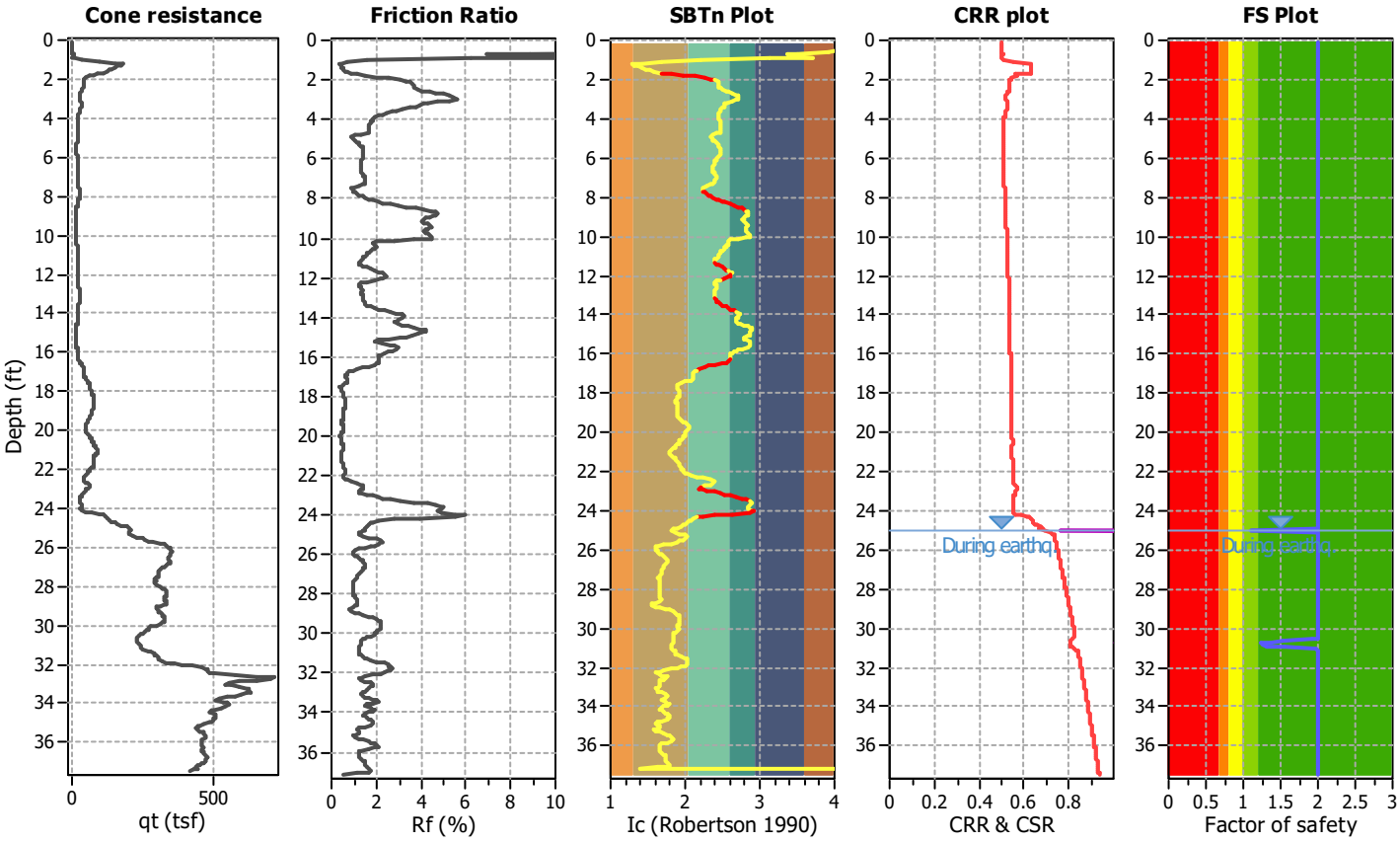
LIQUEFACTION ANALYSIS REPORT

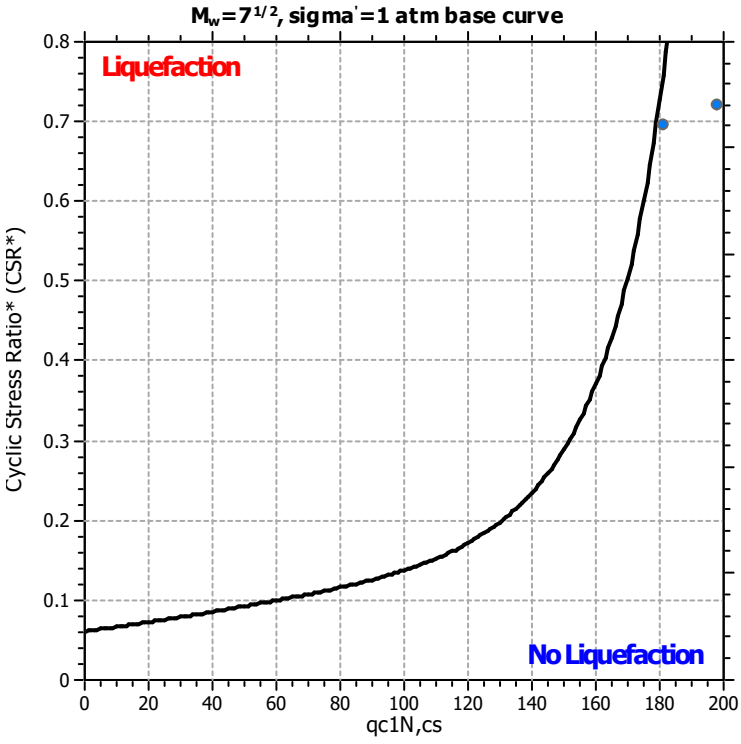
Project title : Riverside City College Cosmetology Building **Location : Riverside City College, Riverside, CA**

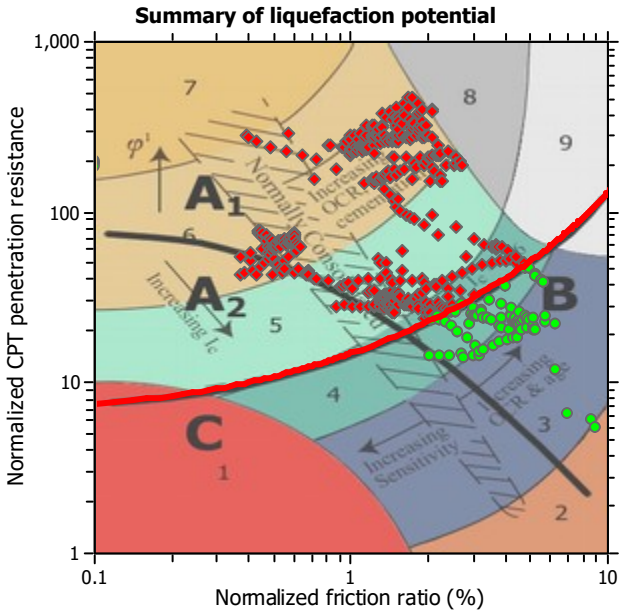
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	50.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	25.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.64	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

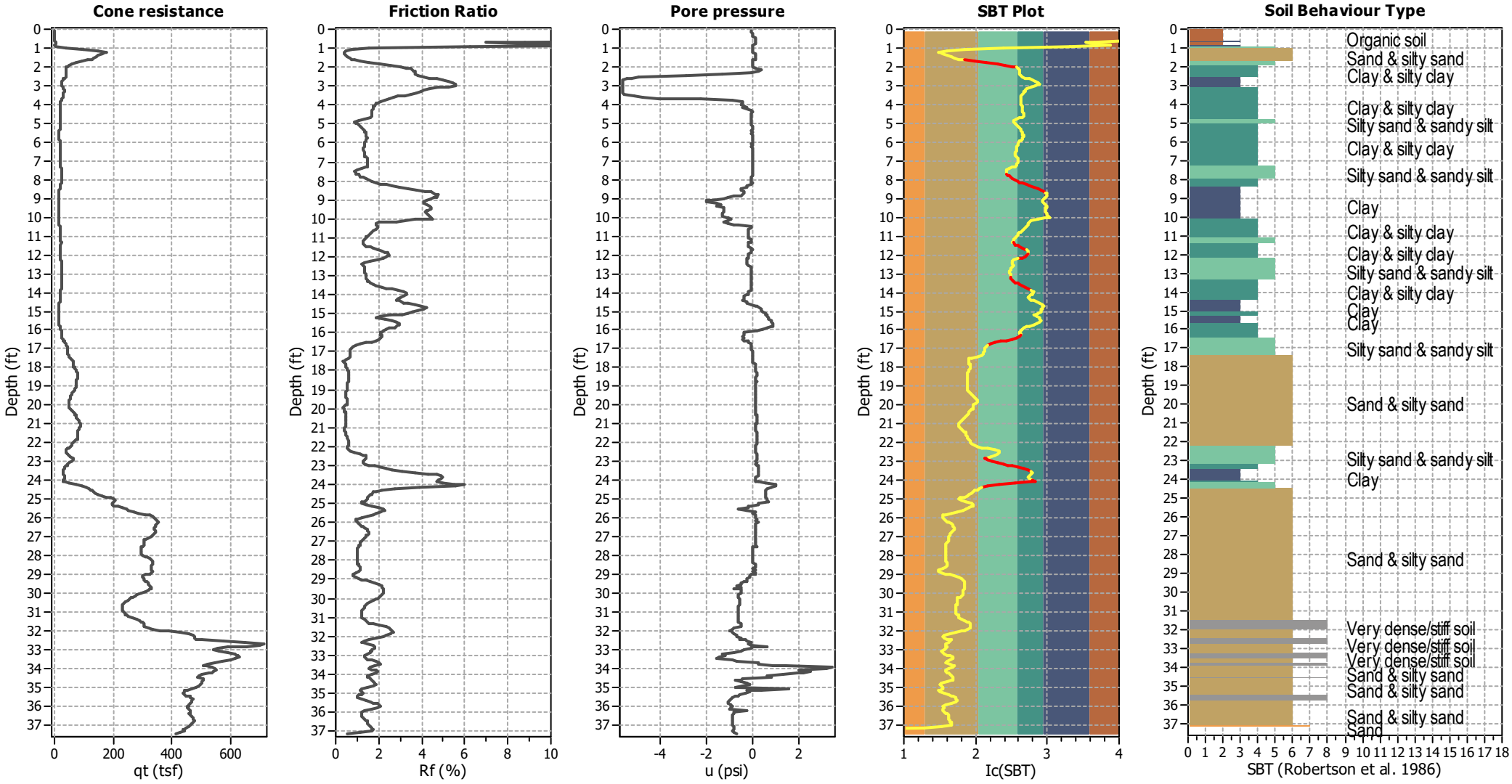






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



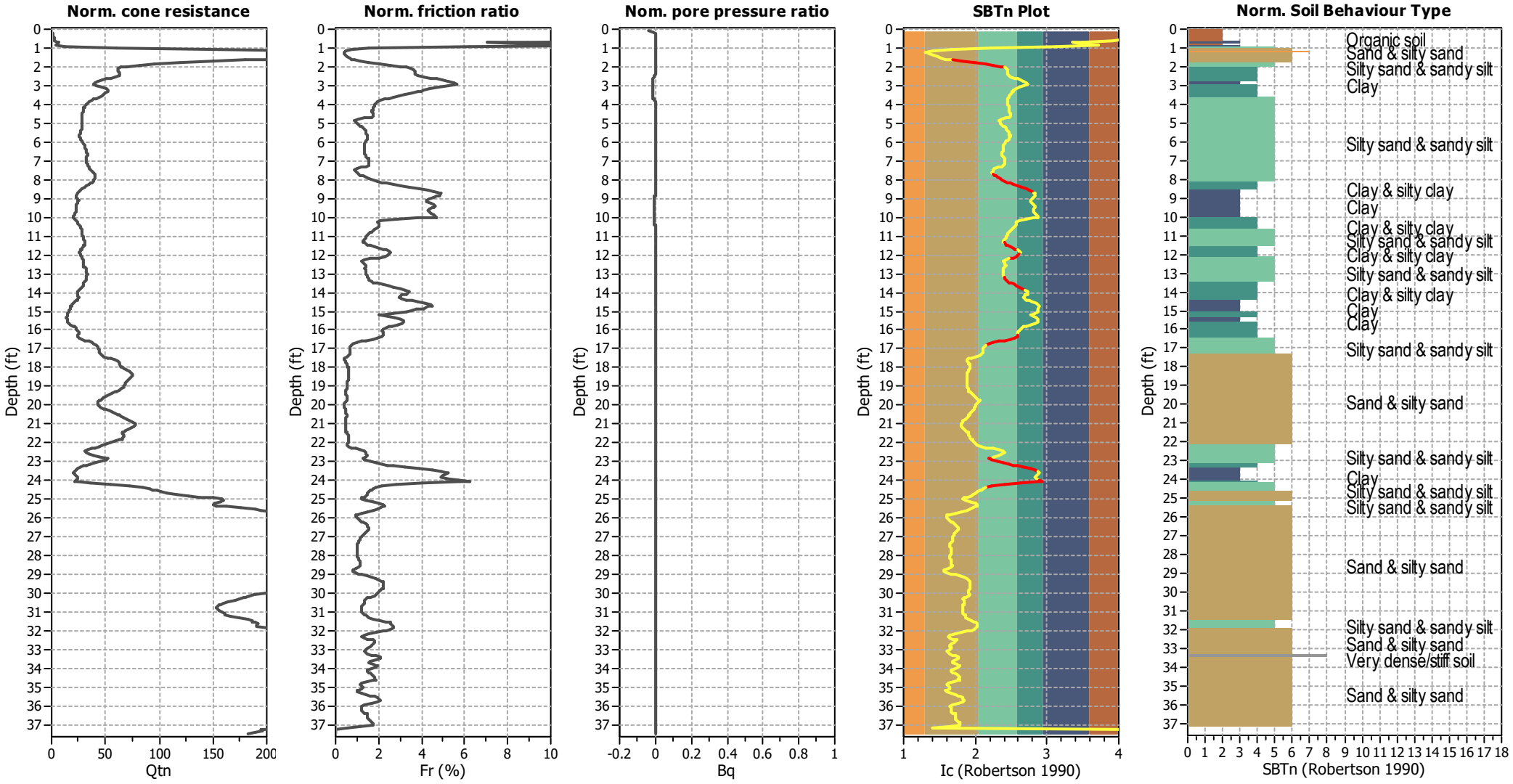
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

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)



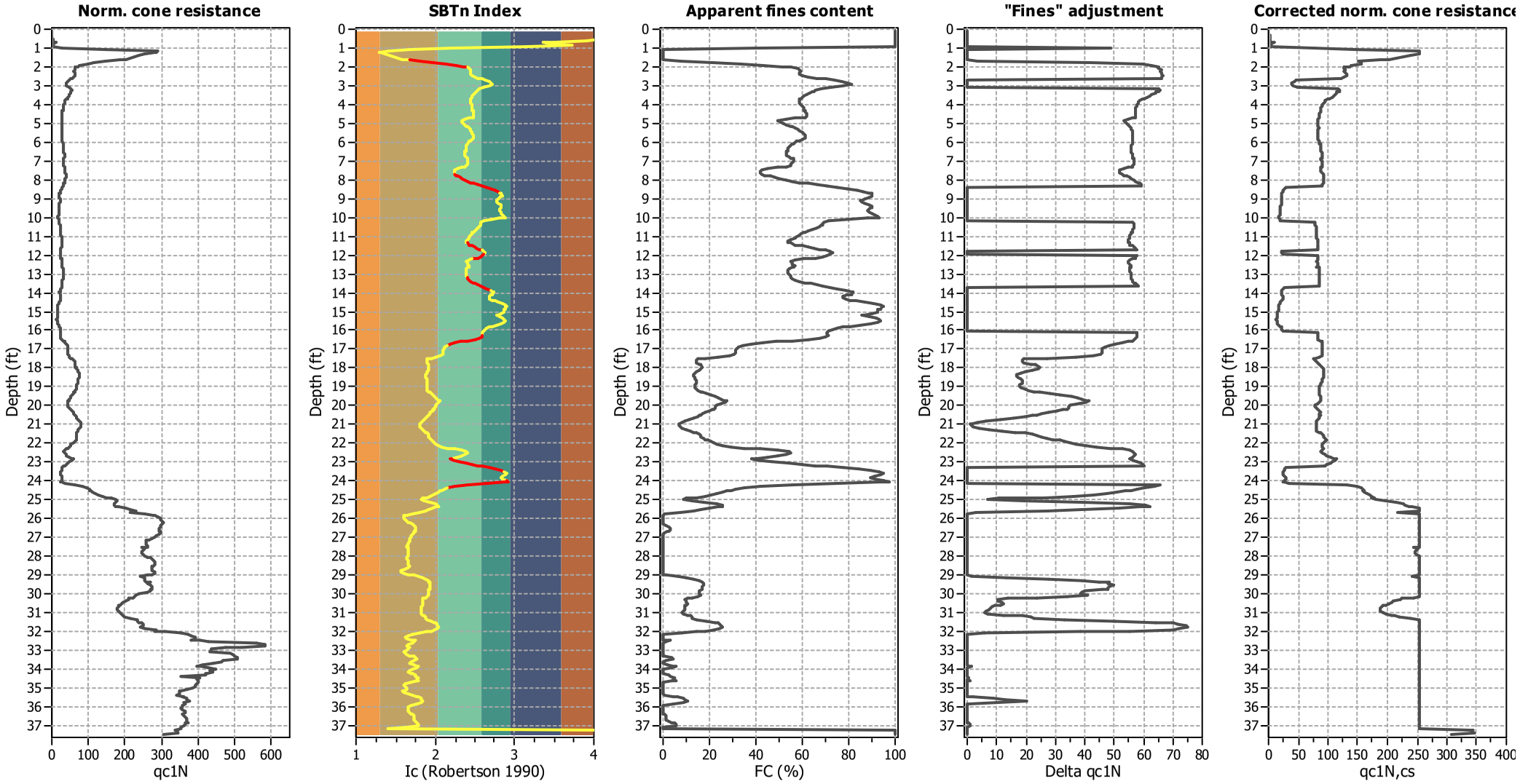
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

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

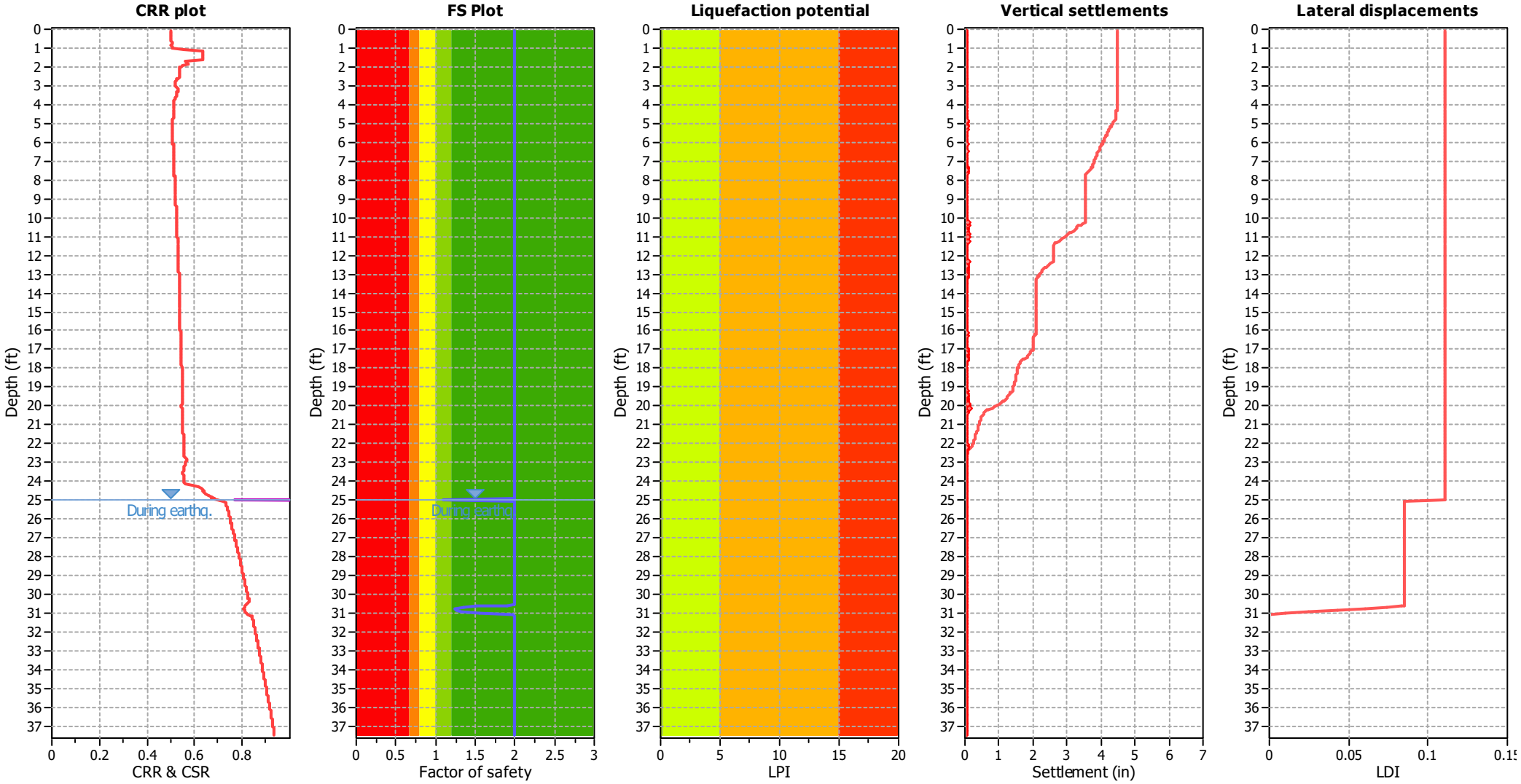
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

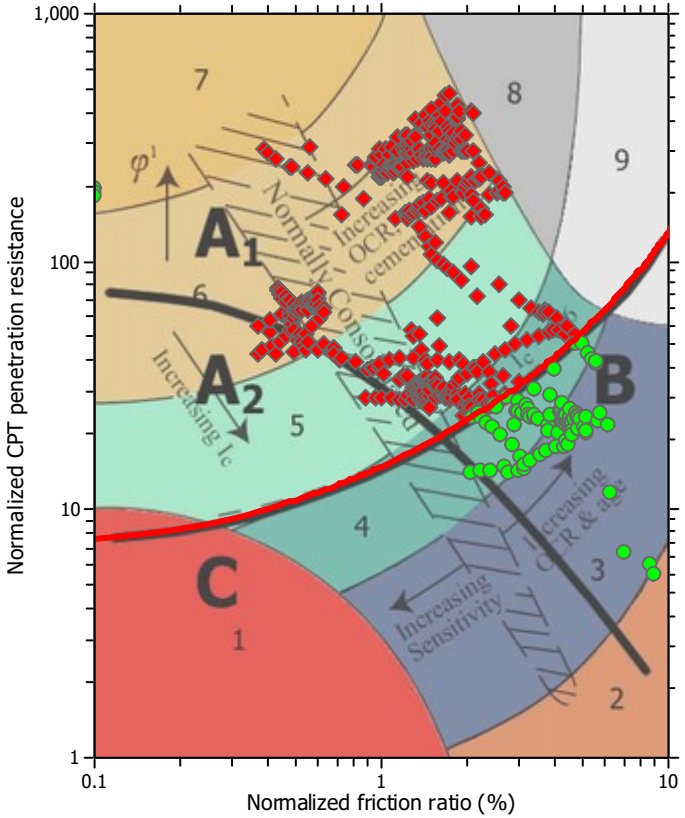
F.S. color scheme

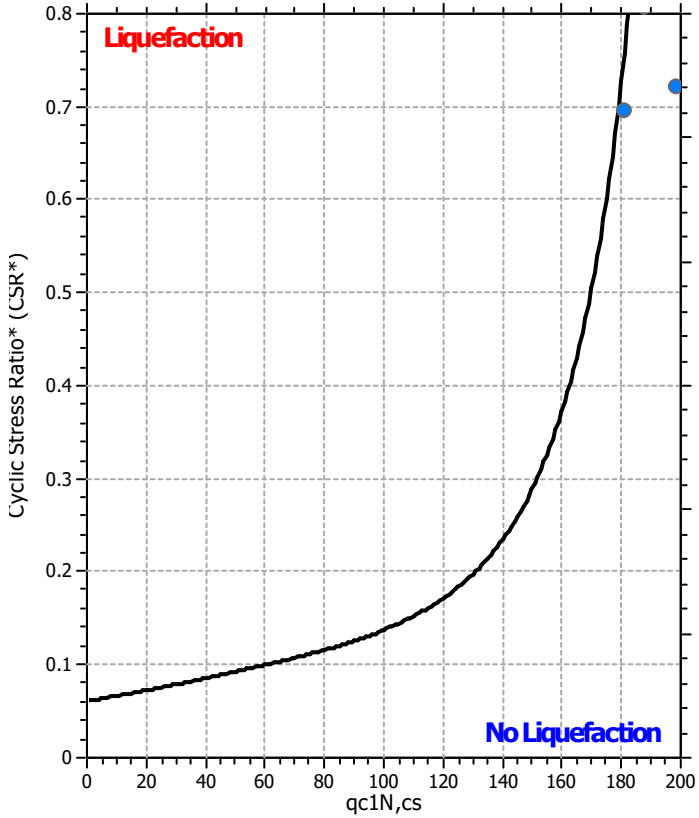
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

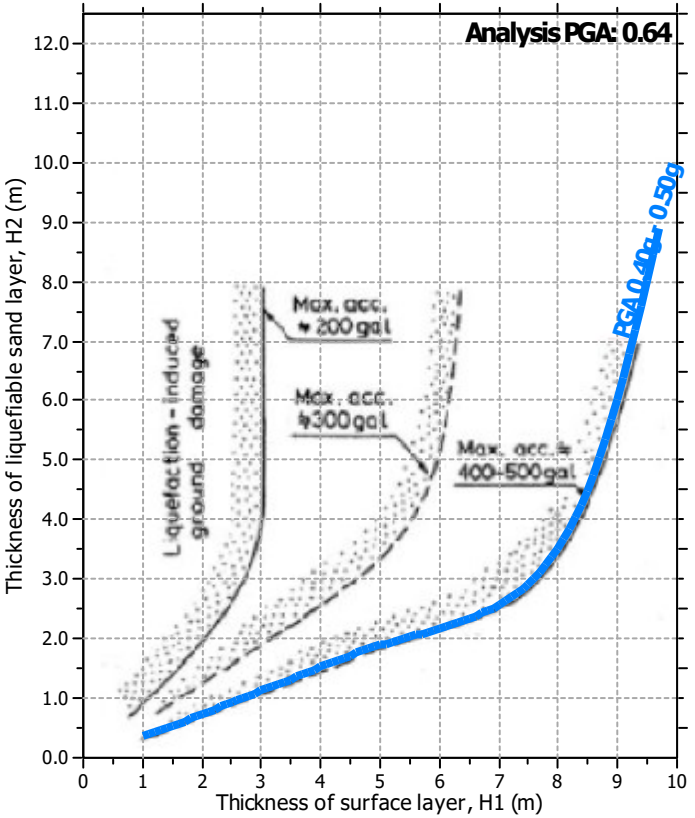
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



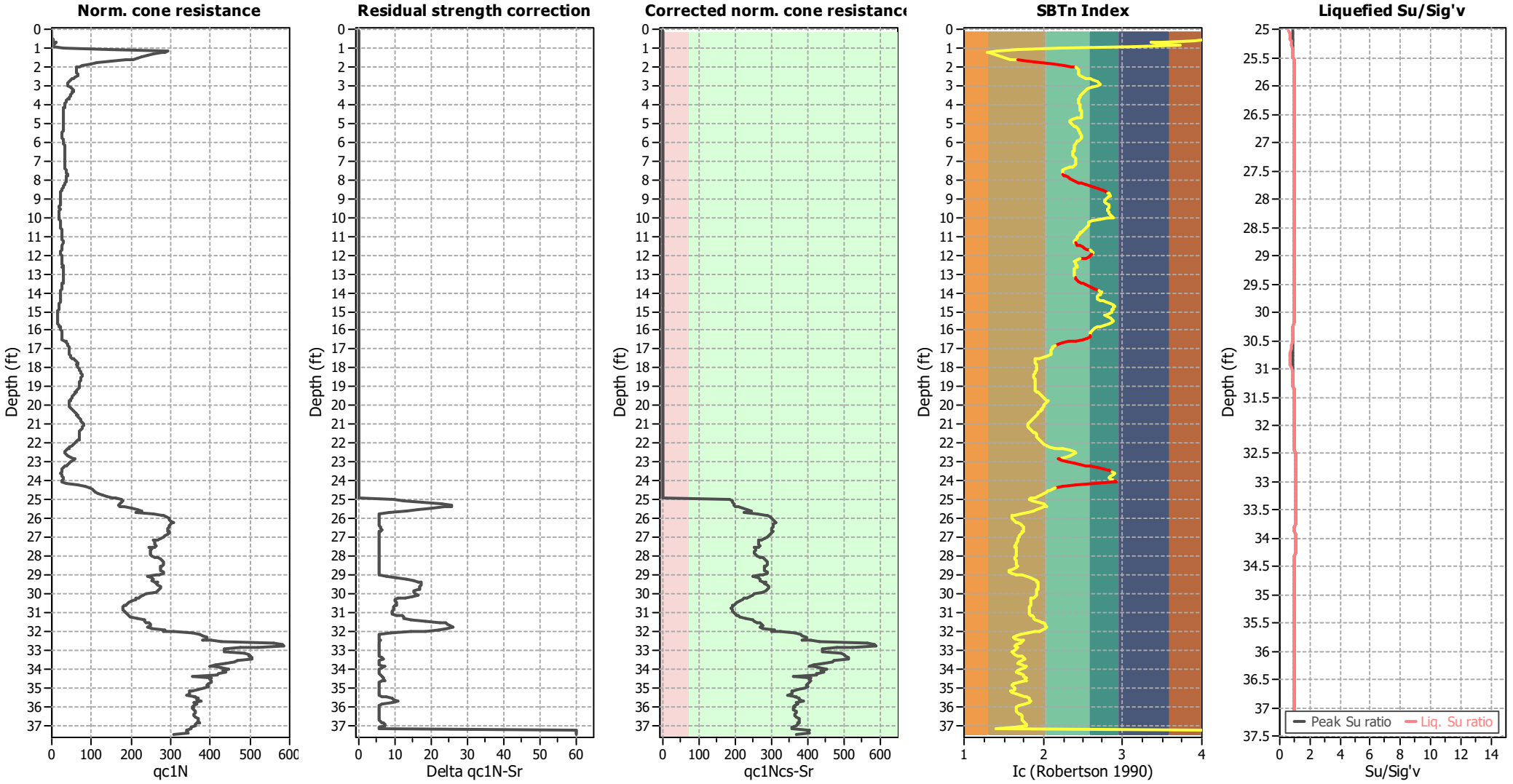




Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

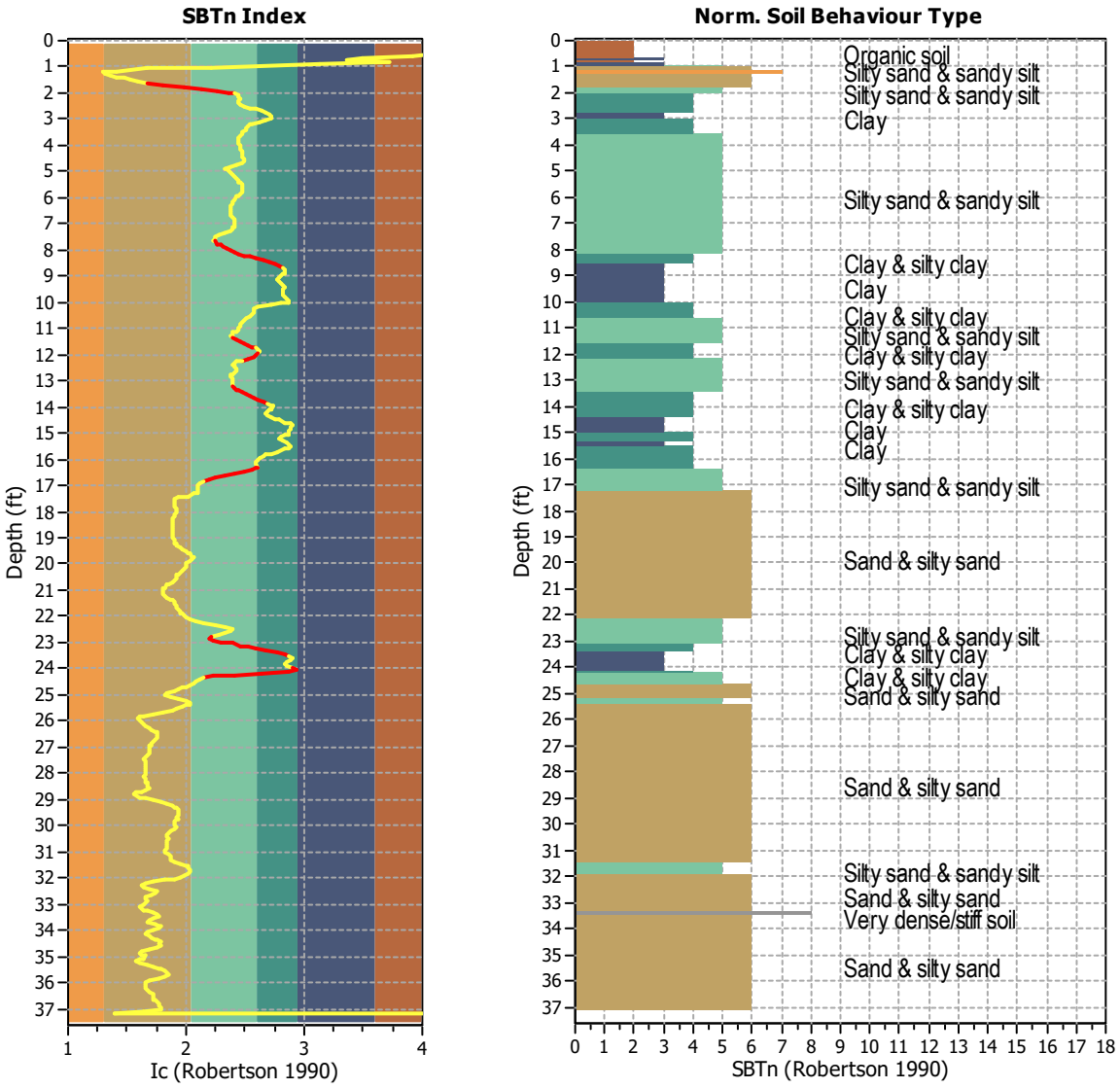
Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.64	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	50.00 ft	Fill height:	N/A	Limit depth:	N/A

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.



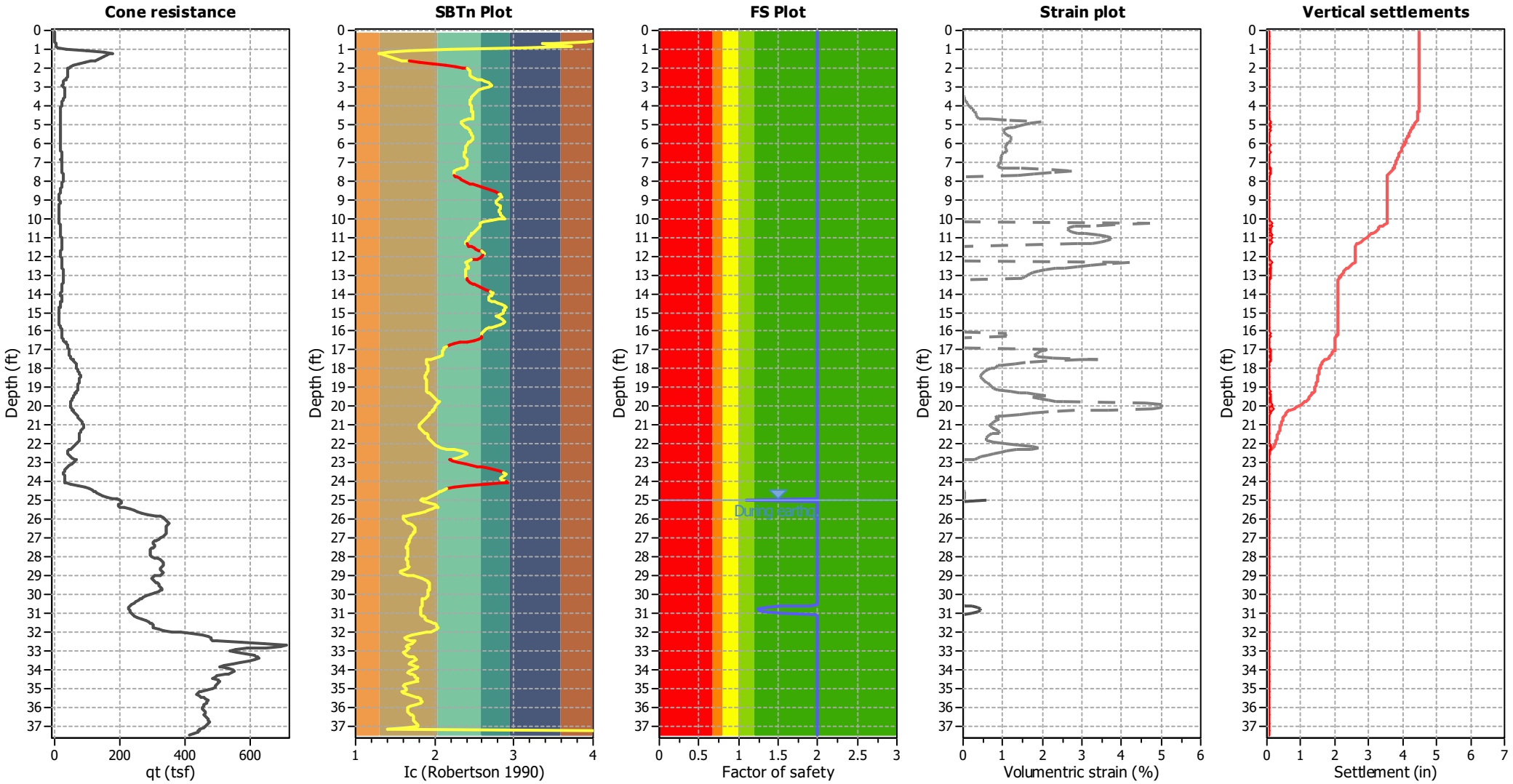
Transition layer algorithm properties

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:	571
Total points excluded:	74
Exclusion percentage:	12.96%
Number of layers detected:	8

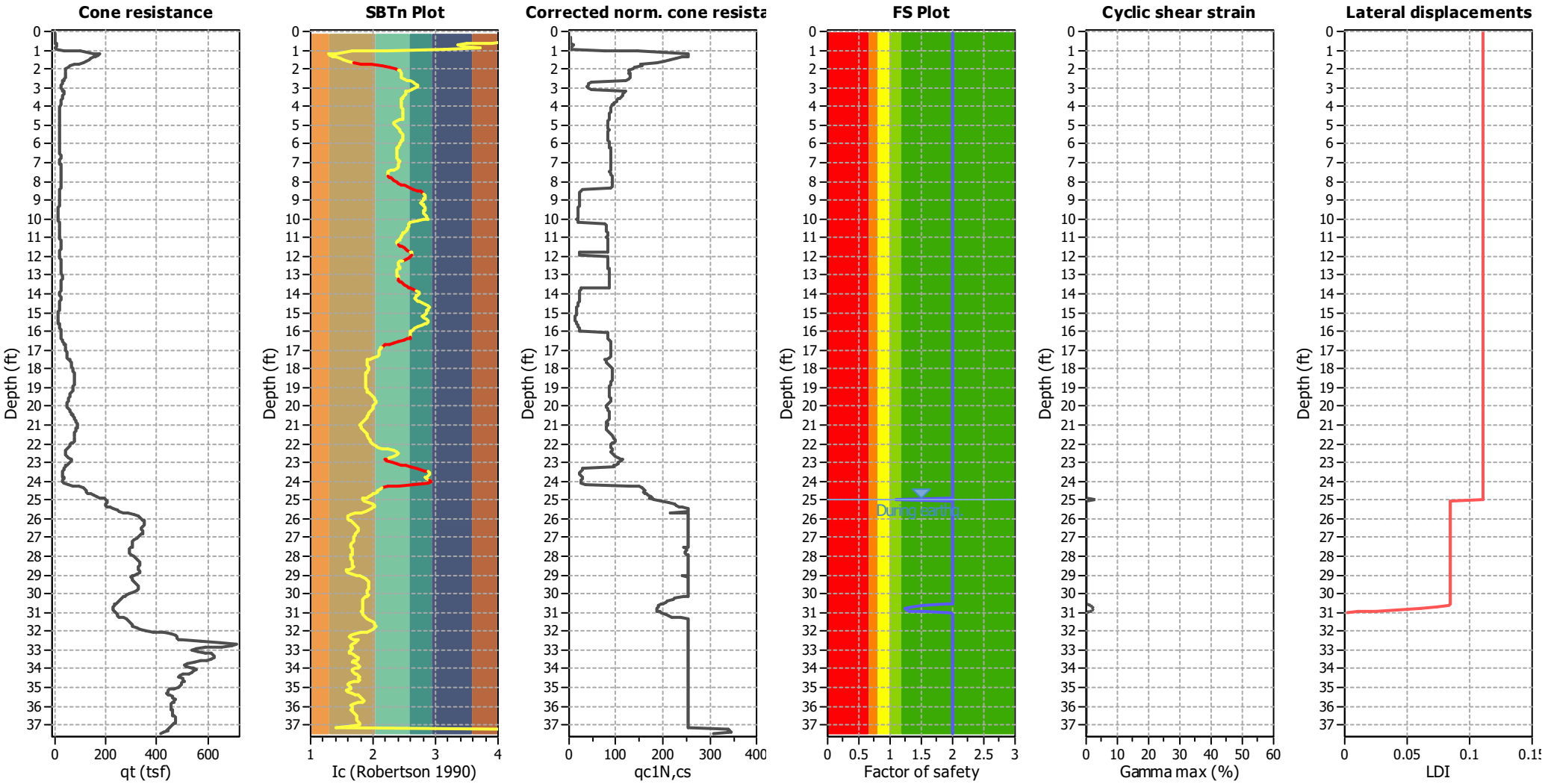
Estimation of post-earthquake settlements



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

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

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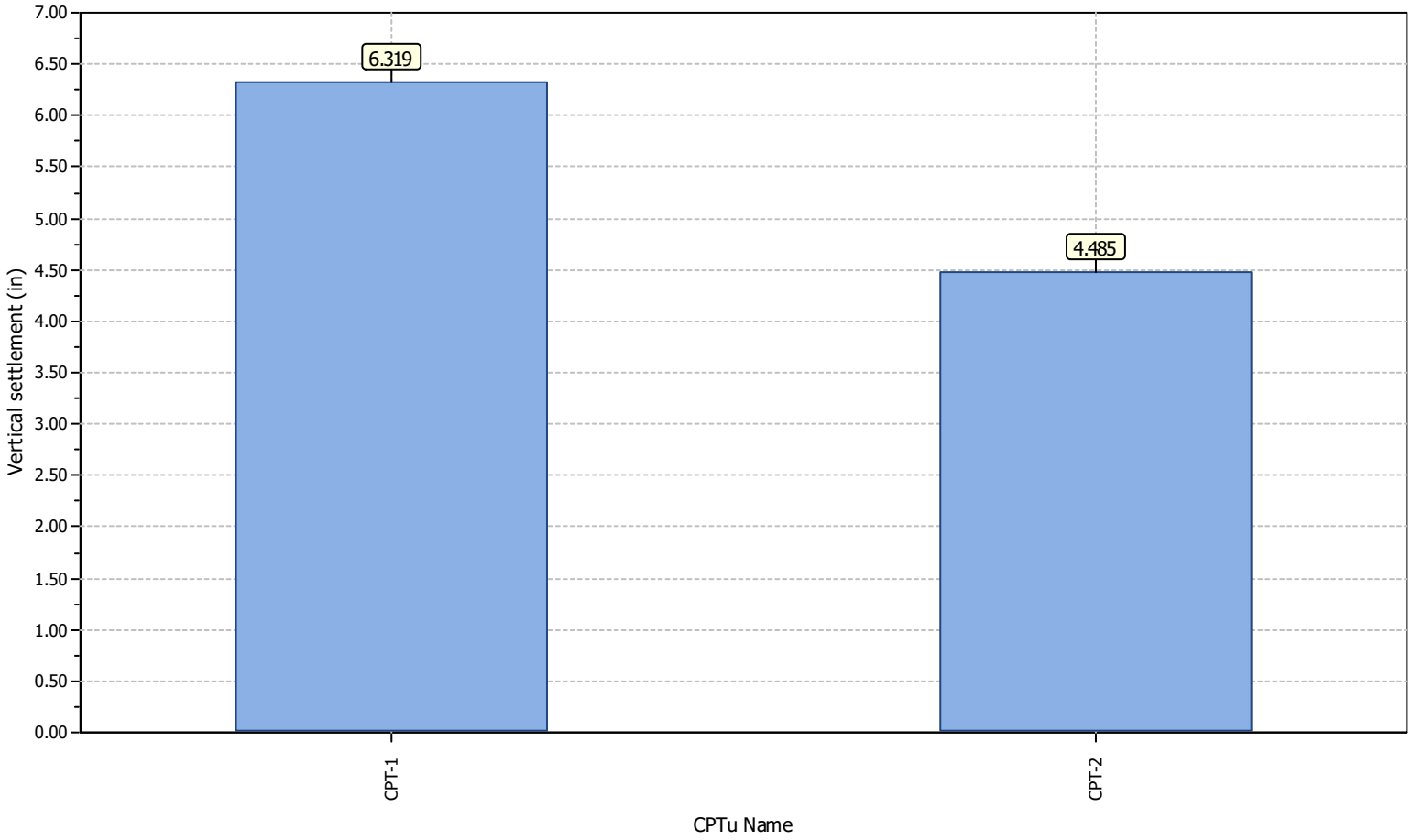
Project title : Riverside City College Cosmetology Building

Location : Riverside City College, Riverside, CA

Overall vertical settlements report



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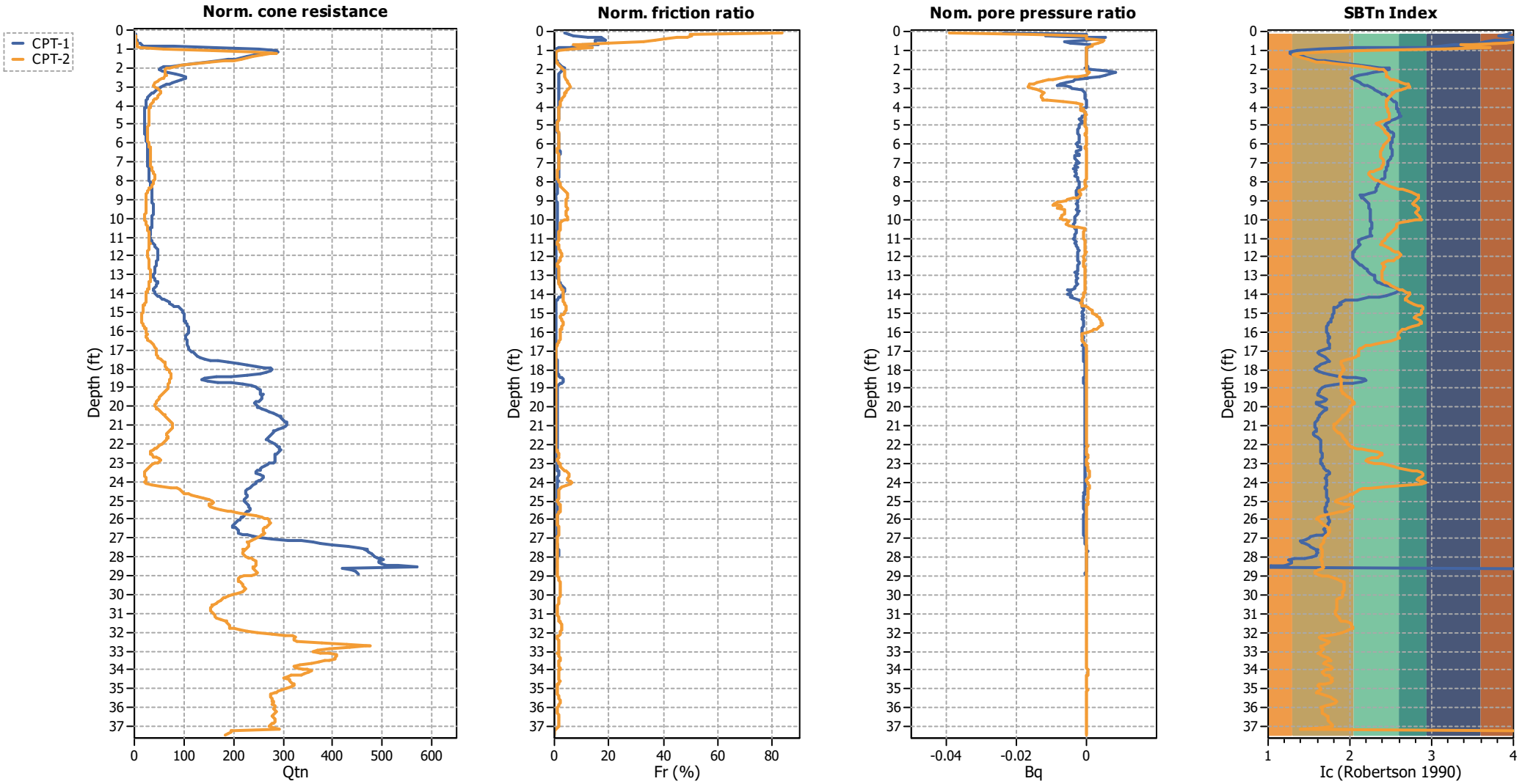
Project: Riverside City College Cosmetology Building

Overlay Normalized Plots



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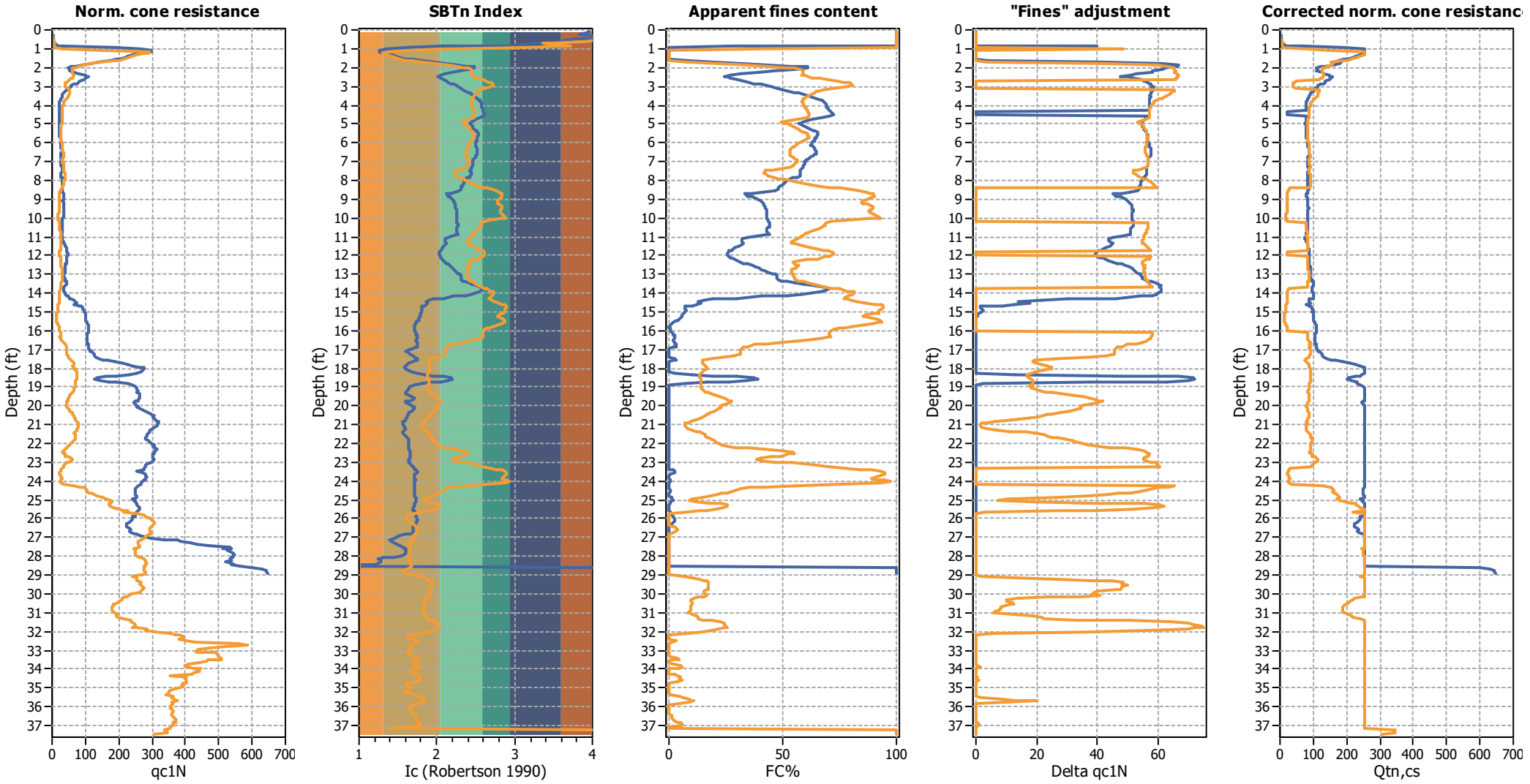
Overlay Intermediate Results



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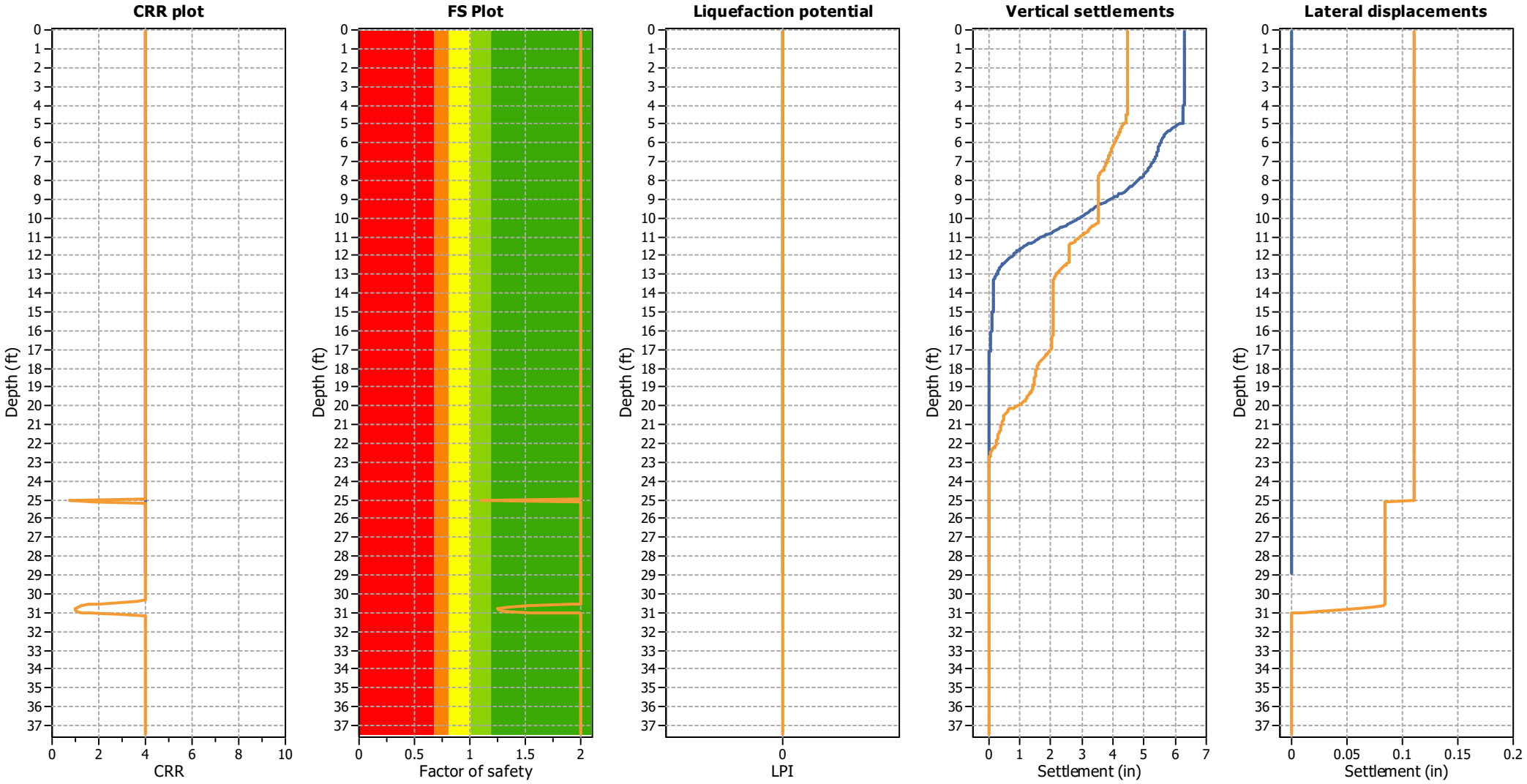
Overlay Cyclic Liquefaction Plots



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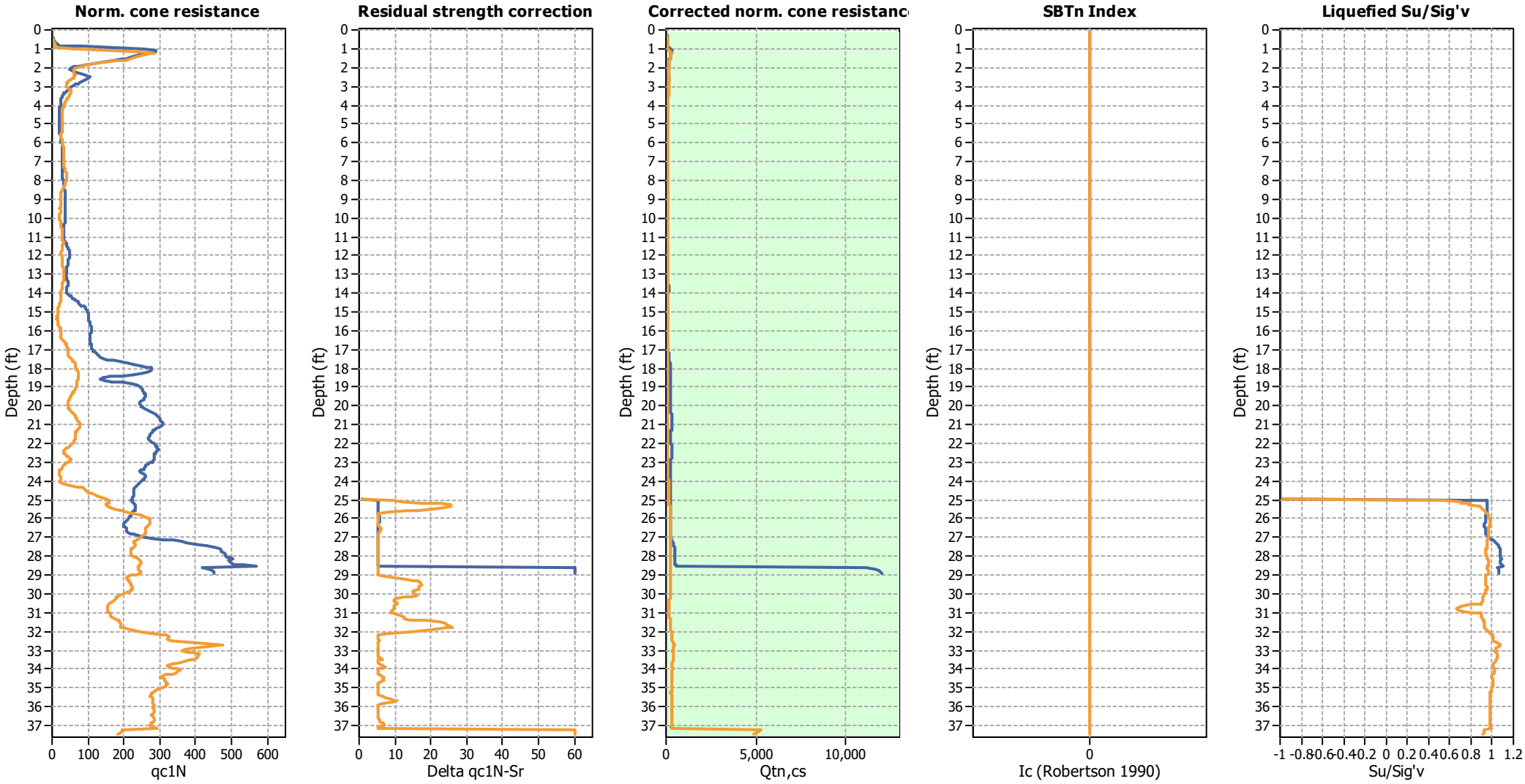
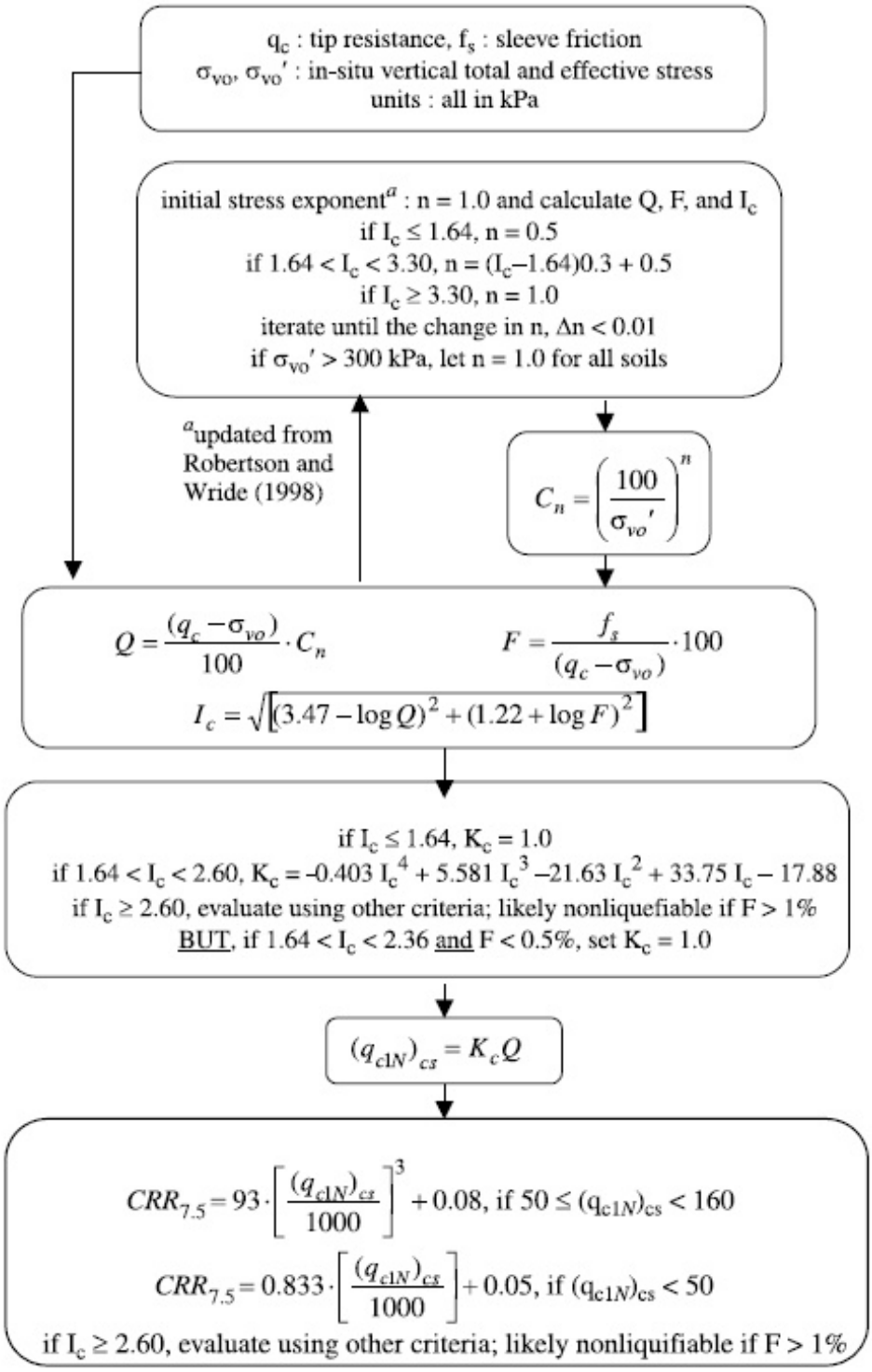
Overlay Strength Loss Plots



Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

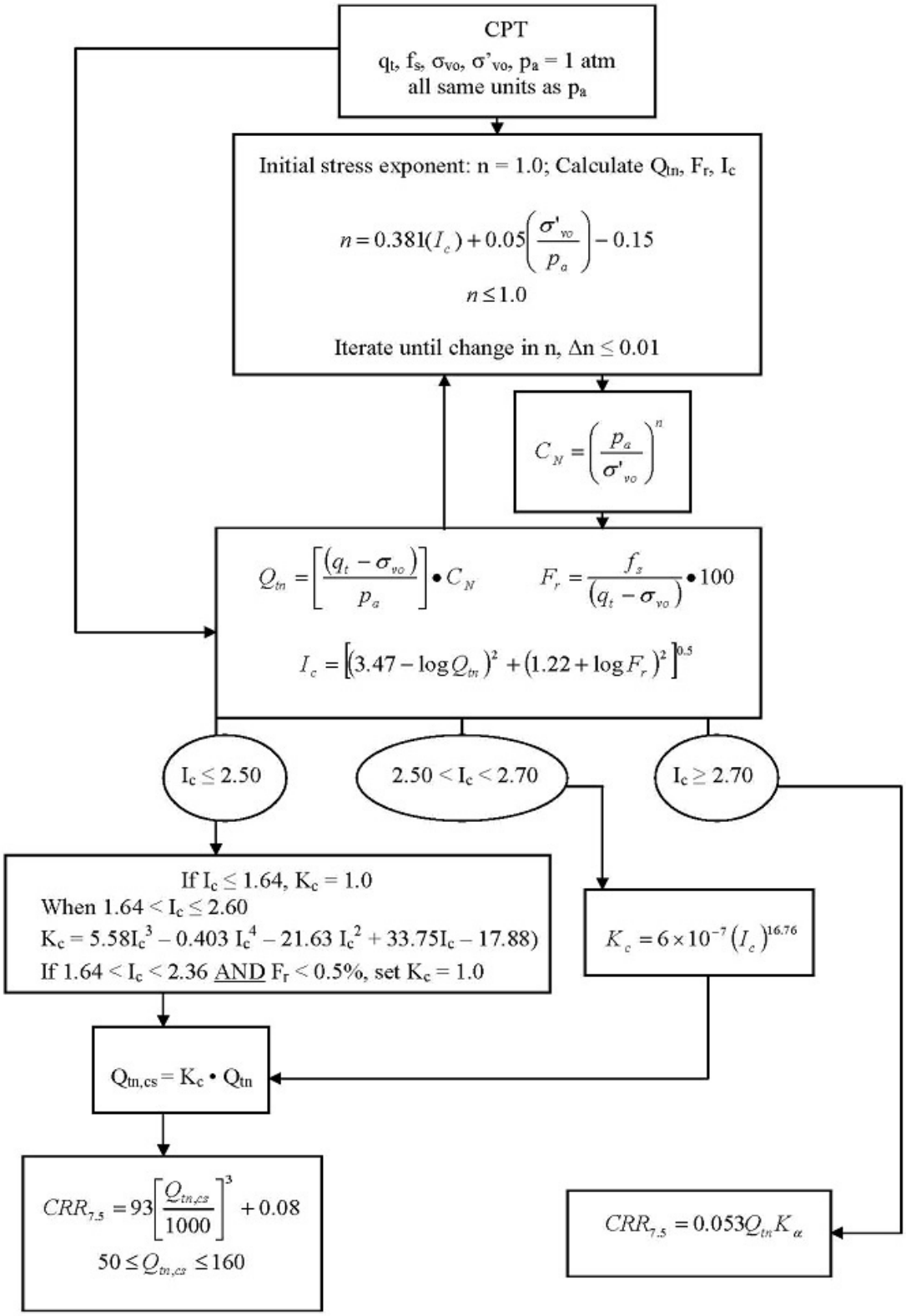
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of 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

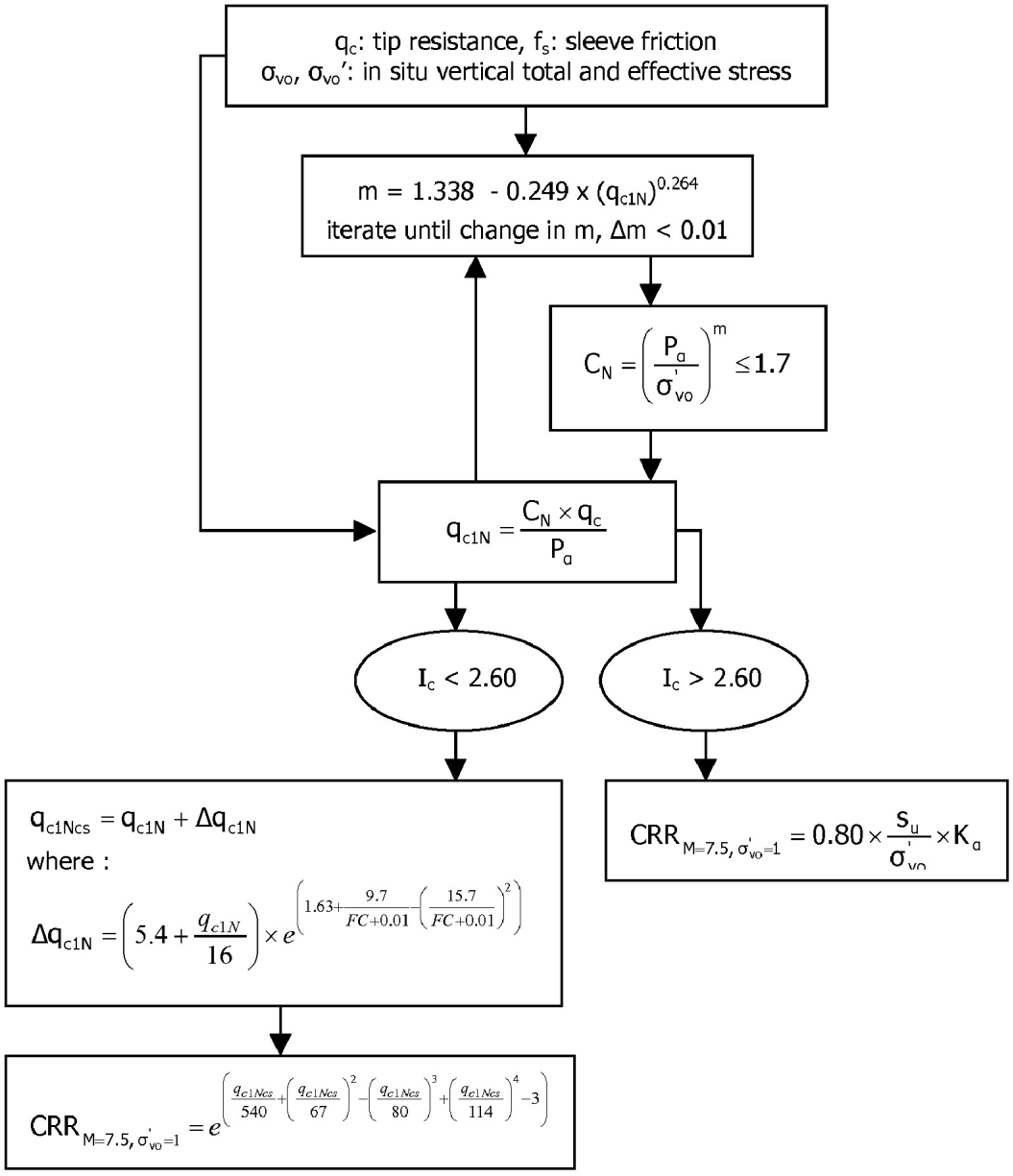
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This 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]:

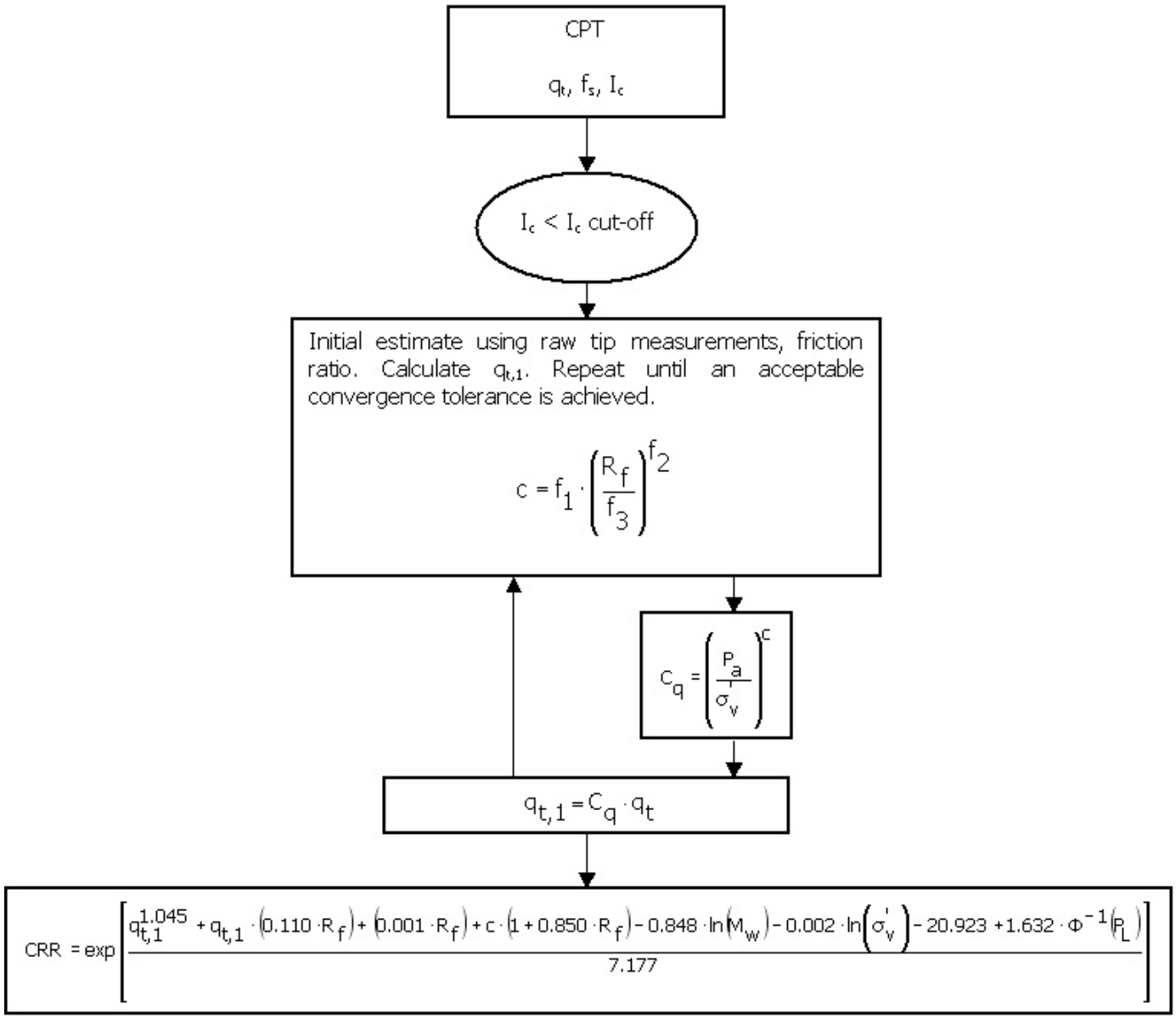


[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

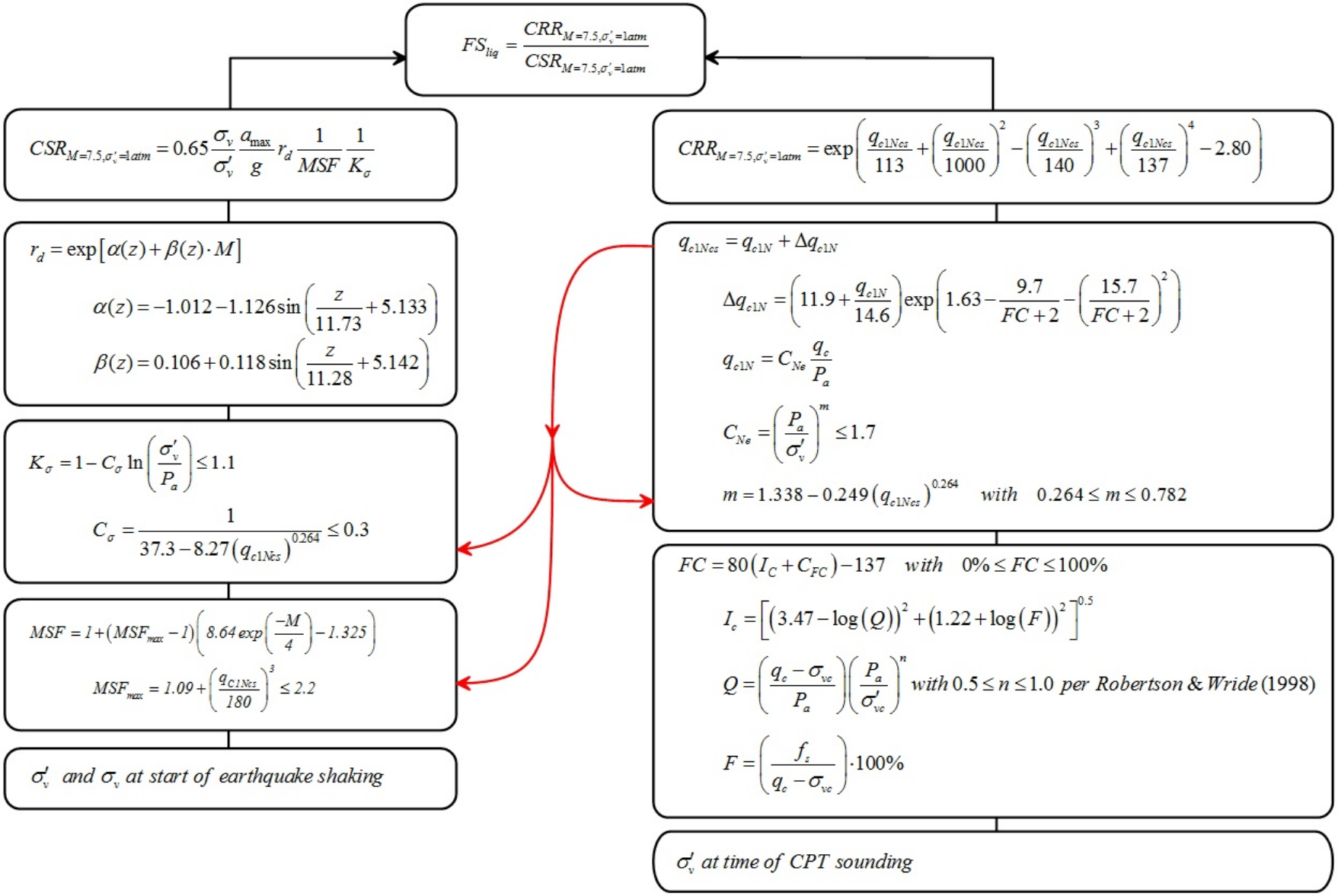
Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



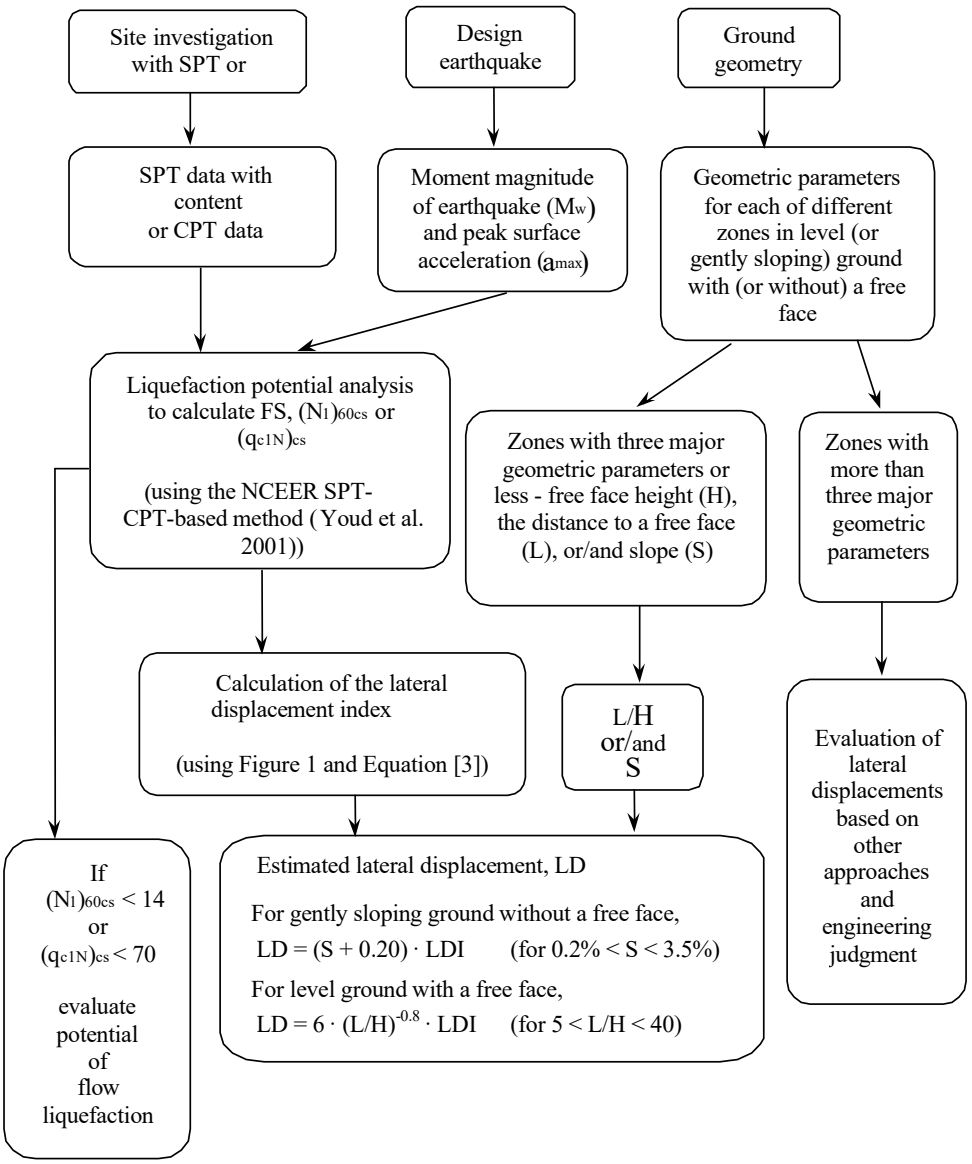
Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)



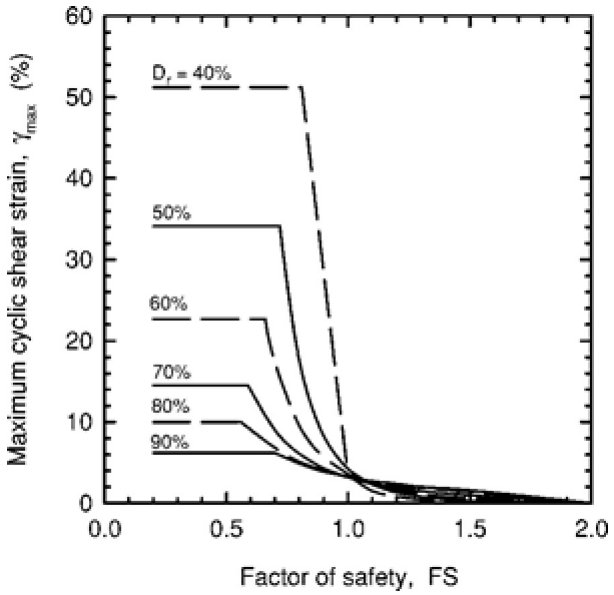
Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)



Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



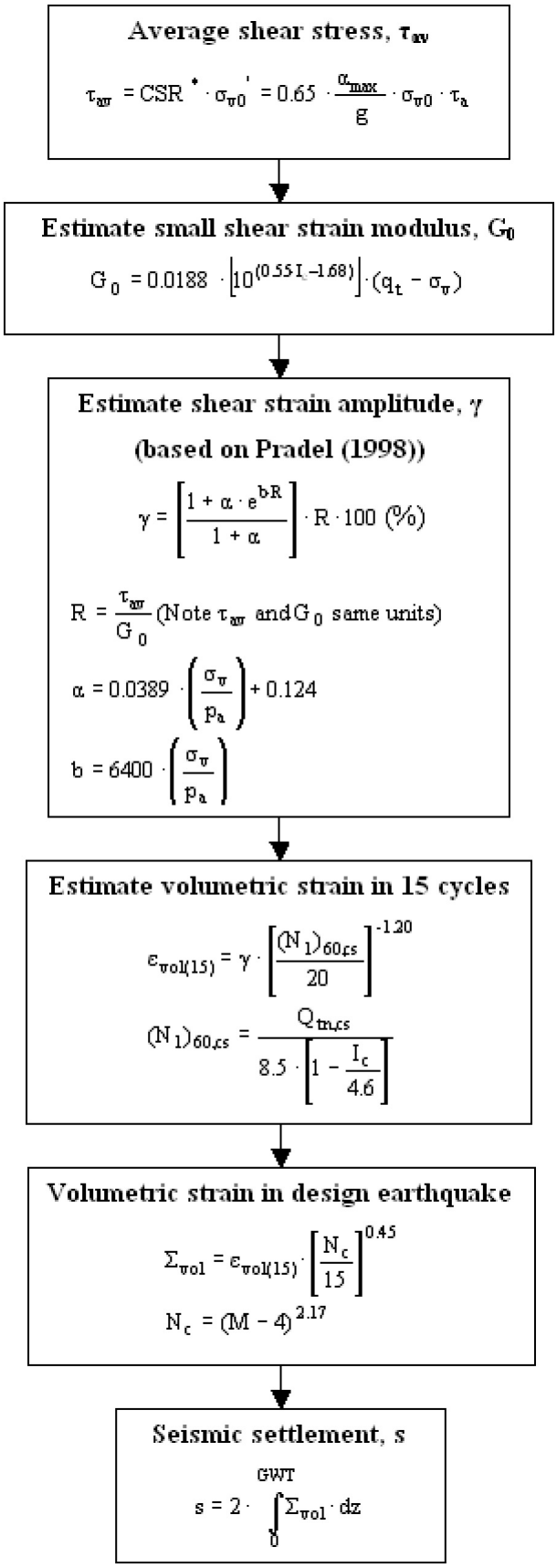
[1] Figure 1

$$LDI = \int_0^{z_{max}} \gamma_{max} dz$$

[1] Equation [3]

[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



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.

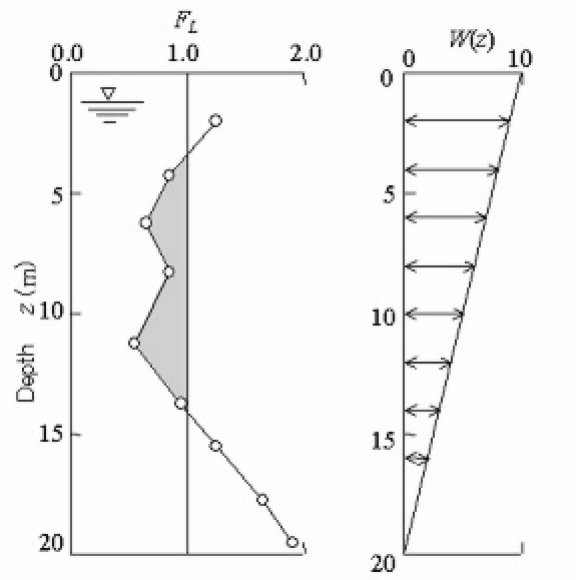
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\mathbf{LPI} = \int_0^{20} (10 - 0{,}5z) \times F_L \times dz$$

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

Shear-Induced Building Settlement (Ds) calculation procedure

The shear-induced building settlement (Ds) due to liquefaction below the building can be estimated using the relationship developed by Bray and Macedo (2017):

$$Ln(Ds) = c1 + c2 * LBS + 0.58 * Ln\left(Tanh\left(\frac{HL}{6}\right)\right) + 4.59 * Ln(Q) - 0.42 * Ln(Q)^2 - 0.02 * B + 0.84 * Ln(CAVdp) + 0.41 * Ln(Sa1) + \varepsilon$$

where Ds is in the units of mm, c1= -8.35 and c2= 0.072 for LBS ≤ 16, and c1= -7.48 and c2= 0.014 otherwise. Q is the building contact pressure in units of kPa, HL is the cumulative thickness of the liquefiable layers in the units of m, B is the building width in the units of m, CAVdp is a standardized version of the cumulative absolute velocity in the units of g-s, Sa1 is 5%-damped pseudo-acceleration response spectral value at a period of 1 s in the units of g, and ε is a normal random variable with zero mean and 0.50 standard deviation in Ln units. The liquefaction-induced building settlement index (LBS) is:

$$LBS = \sum W * \frac{\varepsilon_{shear}}{z} dz$$

where z (m) is the depth measured from the ground surface > 0, W is a foundation-weighting factor wherein W = 0.0 for z less than Df, which is the embedment depth of the foundation, and W = 1.0 otherwise. The shear strain parameter (ε_shear) is the liquefaction-induced free-field shear strain (in %) estimated using Zhang et al. (2004). It is calculated based on the estimated Dr of the liquefied soil layer and the calculated safety factor against liquefaction triggering (FSL).

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APPENDIX G

AXIAL PILE ANALYSIS

DRAFT

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=============================================================================

                  SHAFT for Windows, Version 2012.7.17

                   Serial Number :  154813412

                VERTICALLY LOADED DRILLED SHAFT ANALYSIS
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                       All Rights Reserved

=============================================================================

     Path to file locations      : C:\Users\MFC\OneDrive - MTGL,
Inc\Documents\8073A06_RCC Cosmetology GI\SHAFT Downdrag\
     Name of input data file     : 8073A06_RCC Cosmetology GI Axial Capacity.sfd
     Name of output file         : 8073A06_RCC Cosmetology GI Axial Capacity.sfo
     Name of plot output file    : 8073A06_RCC Cosmetology GI Axial Capacity.sfp
     Name of runtime file        : 8073A06_RCC Cosmetology GI Axial Capacity.sfr

     -----------------------------------------------------------------------
                         Time and Date of Analysis
     -----------------------------------------------------------------------

               Date:  November 20, 2025     Time:  18:43:47

     8073A06_RCC Cosmetology GI


     PROPOSED DEPTH =      50.0 FT
     ----------------


     NUMBER OF LAYERS =    4
     ------------------


     WATER TABLE DEPTH =      25.0 FT.
     -------------------



     FACTOR OF SAFETY APPLIED TO THE ULTIMATE SIDE FRICTION CAPACITY = 2.50
     ---------------------------------------------------------
     FACTOR OF SAFETY APPLIED TO THE ULTIMATE BASE CAPACITY = 3.00
     ----------------------------------------------------


     SOIL INFORMATION
```

LAYER NO 1----CLAY

AT THE TOP

STRENGTH REDUCTION FACTOR-ALPHA	= 0.500E+00
END BEARING COEFFICIENT-Nc	= 0.600E+01
UNDRAINED SHEAR STRENGTH, LB/SQ FT	= 0.400E+03
INTERNAL FRICTION ANGLE, DEG.	= 0.000E+00
BLOWS PER FOOT FROM STANDARD PENETRATION TEST	= 0.600E+01
SOIL UNIT WEIGHT, LB/CU FT	= 0.120E+03
MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT	= 0.100E+11
DEPTH, FT	= 0.000E+00

AT THE BOTTOM

STRENGTH REDUCTION FACTOR-ALPHA	= 0.500E+00
END BEARING COEFFICIENT-Nc	= 0.900E+01
UNDRAINED SHEAR STRENGTH, LB/SQ FT	= 0.400E+03
INTERNAL FRICTION ANGLE, DEG.	= 0.000E+00
BLOWS PER FOOT FROM STANDARD PENETRATION TEST	= 0.600E+01
SOIL UNIT WEIGHT, LB/CU FT	= 0.120E+03
MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT	= 0.100E+11
DEPTH, FT	= 0.100E+02

LAYER NO 2----SAND

AT THE TOP

SKIN FRICTION COEFFICIENT- BETA	= 0.501E+00
UNDRAINED SHEAR STRENGTH, LB/SQ FT	= 0.000E+00
INTERNAL FRICTION ANGLE, DEG.	= 0.290E+02
BLOWS PER FOOT FROM STANDARD PENETRATION TEST	= 0.700E+01
SOIL UNIT WEIGHT, LB/CU FT	= 0.125E+03
MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT	= 0.100E+11
DEPTH, FT	= 0.100E+02

AT THE BOTTOM

SKIN FRICTION COEFFICIENT- BETA	= 0.456E+00
UNDRAINED SHEAR STRENGTH, LB/SQ FT	= 0.000E+00
INTERNAL FRICTION ANGLE, DEG.	= 0.290E+02
BLOWS PER FOOT FROM STANDARD PENETRATION TEST	= 0.700E+01
SOIL UNIT WEIGHT, LB/CU FT	= 0.125E+03
MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT	= 0.100E+11
DEPTH, FT	= 0.150E+02

LAYER NO 3----CLAY

AT THE TOP

STRENGTH REDUCTION FACTOR-ALPHA	= 0.500E+00
END BEARING COEFFICIENT-Nc	= 0.900E+01
UNDRAINED SHEAR STRENGTH, LB/SQ FT	= 0.500E+03
INTERNAL FRICTION ANGLE, DEG.	= 0.000E+00
BLOWS PER FOOT FROM STANDARD PENETRATION TEST	= 0.130E+02
SOIL UNIT WEIGHT, LB/CU FT	= 0.125E+03
MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT	= 0.100E+11
DEPTH, FT	= 0.150E+02

AT THE BOTTOM

STRENGTH REDUCTION FACTOR-ALPHA	= 0.500E+00
END BEARING COEFFICIENT-Nc	= 0.900E+01
UNDRAINED SHEAR STRENGTH, LB/SQ FT	= 0.500E+03
INTERNAL FRICTION ANGLE, DEG.	= 0.000E+00
BLOWS PER FOOT FROM STANDARD PENETRATION TEST	= 0.130E+02
SOIL UNIT WEIGHT, LB/CU FT	= 0.125E+03
MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT	= 0.100E+11
DEPTH, FT	= 0.200E+02

LAYER NO 4----SAND

AT THE TOP

SKIN FRICTION COEFFICIENT- BETA	= 0.896E+00
UNDRAINED SHEAR STRENGTH, LB/SQ FT	= 0.000E+00
INTERNAL FRICTION ANGLE, DEG.	= 0.370E+02
BLOWS PER FOOT FROM STANDARD PENETRATION TEST	= 0.500E+02
SOIL UNIT WEIGHT, LB/CU FT	= 0.125E+03
MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT	= 0.100E+11
DEPTH, FT	= 0.200E+02

AT THE BOTTOM

SKIN FRICTION COEFFICIENT- BETA	= 0.545E+00
UNDRAINED SHEAR STRENGTH, LB/SQ FT	= 0.000E+00
INTERNAL FRICTION ANGLE, DEG.	= 0.370E+02
BLOWS PER FOOT FROM STANDARD PENETRATION TEST	= 0.500E+02
SOIL UNIT WEIGHT, LB/CU FT	= 0.125E+03
MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT	= 0.100E+11
DEPTH, FT	= 0.500E+02

DRILLED SHAFT INFORMATION

DIAMETER OF STEM	=	2.000	FT.
DIAMETER OF BASE	=	2.000	FT.
END OF STEM TO BASE	=	0.000	FT.
ANGLE OF BELL	=	0.000	DEG.
IGNORED TOP PORTION	=	20.000	FT.
IGNORED BOTTOM PORTION	=	0.000	FT.
AREA OF ONE PERCENT STEEL	=	4.524	SQ.IN.
ELASTIC MODULUS, Ec	=	0.360E+07	LB/SQ IN
VOLUME OF UNDERREAM	=	0.000	CU.YDS.

PREDICTED RESULTS

QS = ULTIMATE SIDE RESISTANCE;
QB = ULTIMATE BASE RESISTANCE;
WT = WEIGHT OF DRILLED SHAFT (FOR UPLIFT CAPACITY ONLY);
QU = TOTAL ULTIMATE RESISTANCE;
QBD = TOTAL ALLOWABLE LOAD USING A FACTOR OF SAFETY
APPLIED TO THE ULTIMATE BASE RESISTANCE;
QDN = TOTAL ALLOWABLE LOAD USING FACTORS OF SAFETY
APPLIED TO THE ULTIMATE SIDE RESISTANCE AND
THE ULTIMATE BASE RESISTANCE.

LENGTH (FEET)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	QBD (TONS)	QDN (TONS)	QU/VOLUME (TONS/CU.YDS)
21.0	2.44	6.96	65.35	72.31	28.74	24.57	29.59
22.0	2.56	14.14	65.35	79.49	35.93	27.44	31.05
23.0	2.68	21.54	65.35	86.89	43.33	30.40	32.47
24.0	2.79	29.15	65.35	94.50	50.93	33.44	33.84
25.0	2.91	36.96	65.35	102.31	58.74	36.57	35.17
26.0	3.03	44.88	65.35	110.23	66.66	39.74	36.43
27.0	3.14	52.83	65.35	118.18	74.61	42.92	37.61
28.0	3.26	60.81	65.35	126.16	82.59	46.11	38.72
29.0	3.37	68.81	65.35	134.16	90.59	49.31	39.75
30.0	3.49	76.83	65.35	142.18	98.61	52.52	40.73
31.0	3.61	84.87	65.35	150.22	106.65	55.73	41.64
32.0	3.72	92.92	65.35	158.28	114.71	58.95	42.50
33.0	3.84	100.99	65.35	166.35	122.78	62.18	43.32
34.0	3.96	109.07	65.35	174.43	130.86	65.41	44.08
35.0	4.07	117.16	65.35	182.51	138.94	68.65	44.81

36.0	4.19	125.25	65.35	190.60	147.03	71.88	45.50
37.0	4.31	133.34	65.35	198.70	155.13	75.12	46.15
38.0	4.42	141.44	65.35	206.79	163.22	78.36	46.76
39.0	4.54	149.53	65.35	214.88	171.31	81.60	47.35
40.0	4.65	157.62	65.35	222.97	179.40	84.83	47.90
41.0	4.77	165.69	65.35	231.05	187.48	88.06	48.43
42.0	4.89	173.76	65.35	239.12	195.55	91.29	48.92
43.0	5.00	181.82	65.35	247.17	203.60	94.51	49.40
44.0	5.12	189.86	65.35	255.21	211.64	97.73	49.84
45.0	5.24	197.88	65.35	263.23	219.67	100.94	50.27

RESULT FROM TREND (AVERAGED) LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.3370E-01	0.2588E-04	0.9531E-03	0.1000E-04
0.1685E+00	0.1294E-03	0.4765E-02	0.5000E-04
0.3370E+00	0.2588E-03	0.9531E-02	0.1000E-03
0.1707E+02	0.1302E-01	0.4765E+00	0.5000E-02
0.2567E+02	0.1956E-01	0.7148E+00	0.7500E-02
0.3428E+02	0.2611E-01	0.9531E+00	0.1000E-01
0.8072E+02	0.6333E-01	0.2383E+01	0.2500E-01
0.1316E+03	0.1133E+00	0.4765E+01	0.5000E-01
0.1573E+03	0.1514E+00	0.7148E+01	0.7500E-01
0.1744E+03	0.1851E+00	0.9531E+01	0.1000E+00
0.2130E+03	0.3568E+00	0.2331E+02	0.2500E+00
0.2266E+03	0.6160E+00	0.3761E+02	0.5000E+00
0.2319E+03	0.7446E+00	0.4305E+02	0.6250E+00
0.2389E+03	0.9055E+00	0.5021E+02	0.7812E+00
0.2547E+03	0.1335E+01	0.6633E+02	0.1200E+01

RESULT FROM UPPER-BOUND LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.4795E-01	0.3237E-04	0.1362E-02	0.1000E-04
0.2397E+00	0.1618E-03	0.6808E-02	0.5000E-04
0.4795E+00	0.3237E-03	0.1362E-01	0.1000E-03
0.2447E+02	0.1638E-01	0.6808E+00	0.5000E-02
0.3679E+02	0.2461E-01	0.1021E+01	0.7500E-02
0.4906E+02	0.3281E-01	0.1362E+01	0.1000E-01
0.1121E+03	0.7795E-01	0.3404E+01	0.2500E-01
0.1712E+03	0.1326E+00	0.6808E+01	0.5000E-01
0.1933E+03	0.1695E+00	0.1021E+02	0.7500E-01
0.2048E+03	0.2007E+00	0.1362E+02	0.1000E+00
0.2299E+03	0.3666E+00	0.3240E+02	0.2500E+00

0.2444E+03	0.6263E+00	0.4722E+02	0.5000E+00
0.2479E+03	0.7537E+00	0.5076E+02	0.6250E+00
0.2541E+03	0.9140E+00	0.5690E+02	0.7812E+00
0.2678E+03	0.1342E+01	0.7058E+02	0.1200E+01

RESULT FROM LOWER-BOUND LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.2079E-01	0.1990E-04	0.5446E-03	0.1000E-04
0.1040E+00	0.9949E-04	0.2723E-02	0.5000E-04
0.2079E+00	0.1990E-03	0.5446E-02	0.1000E-03
0.1044E+02	0.9963E-02	0.2723E+00	0.5000E-02
0.1573E+02	0.1497E-01	0.4085E+00	0.7500E-02
0.2101E+02	0.1998E-01	0.5446E+00	0.1000E-01
0.5115E+02	0.4940E-01	0.1362E+01	0.2500E-01
0.9097E+02	0.9362E-01	0.2723E+01	0.5000E-01
0.1198E+03	0.1328E+00	0.4085E+01	0.7500E-01
0.1428E+03	0.1690E+00	0.5446E+01	0.1000E+00
0.1961E+03	0.3470E+00	0.1421E+02	0.2500E+00
0.2088E+03	0.6057E+00	0.2799E+02	0.5000E+00
0.2159E+03	0.7355E+00	0.3535E+02	0.6250E+00
0.2238E+03	0.8970E+00	0.4352E+02	0.7812E+00
0.2415E+03	0.1328E+01	0.6209E+02	0.1200E+01

DRILLED SHAFT INFORMATION

DIAMETER OF STEM	=	2.500	FT.
DIAMETER OF BASE	=	2.500	FT.
END OF STEM TO BASE	=	0.000	FT.
ANGLE OF BELL	=	0.000	DEG.
IGNORED TOP PORTION	=	20.000	FT.
IGNORED BOTTOM PORTION	=	0.000	FT.
AREA OF ONE PERCENT STEEL	=	7.069	SQ.IN.
ELASTIC MODULUS, Ec	=	0.360E+07	LB/SQ IN
VOLUME OF UNDERREAM	=	0.000	CU.YDS.

PREDICTED RESULTS

QS = ULTIMATE SIDE RESISTANCE;
QB = ULTIMATE BASE RESISTANCE;
WT = WEIGHT OF DRILLED SHAFT (FOR UPLIFT CAPACITY ONLY);

QU = TOTAL ULTIMATE RESISTANCE;
QBD = TOTAL ALLOWABLE LOAD USING A FACTOR OF SAFETY APPLIED TO THE ULTIMATE BASE RESISTANCE;
QDN = TOTAL ALLOWABLE LOAD USING FACTORS OF SAFETY APPLIED TO THE ULTIMATE SIDE RESISTANCE AND THE ULTIMATE BASE RESISTANCE.

LENGTH (FEET)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	QBD (TONS)	QDN (TONS)	QU/VOLUME (TONS/CU.YDS)
21.0	3.82	8.70	99.89	108.58	41.99	36.77	28.44
22.0	4.00	17.68	101.56	119.23	51.53	40.92	29.81
23.0	4.18	26.93	102.11	129.04	60.96	44.81	30.86
24.0	4.36	36.44	102.11	138.55	70.48	48.61	31.75
25.0	4.55	46.20	102.11	148.31	80.24	52.52	32.63
26.0	4.73	56.10	102.11	158.21	90.14	56.48	33.47
27.0	4.91	66.04	102.11	168.15	100.08	60.45	34.25
28.0	5.09	76.01	102.11	178.12	110.05	64.44	34.99
29.0	5.27	86.01	102.11	188.12	120.05	68.44	35.68
30.0	5.45	96.04	102.11	198.15	130.08	72.45	36.33
31.0	5.64	106.09	102.11	208.20	140.13	76.47	36.94
32.0	5.82	116.16	102.11	218.27	150.19	80.50	37.51
33.0	6.00	126.24	102.11	228.36	160.28	84.54	38.06
34.0	6.18	136.34	102.11	238.46	170.38	88.57	38.57
35.0	6.36	146.45	102.11	248.56	180.49	92.62	39.06
36.0	6.55	156.56	102.11	258.68	190.60	96.66	39.52
37.0	6.73	166.68	102.11	268.80	200.72	100.71	39.95
38.0	6.91	176.80	102.11	278.91	210.84	104.76	40.37
39.0	7.09	186.91	102.11	289.03	220.95	108.80	40.76
40.0	7.27	197.02	102.11	299.13	231.06	112.85	41.13
41.0	7.45	207.12	102.11	309.23	241.16	116.89	41.48
42.0	7.64	217.20	102.11	319.32	251.24	120.92	41.81
43.0	7.82	227.27	102.11	329.39	261.31	124.95	42.13
44.0	8.00	237.32	102.11	339.44	271.36	128.97	42.43

RESULT FROM TREND (AVERAGED) LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.2991E-01	0.1910E-04	0.1191E-02	0.1000E-04
0.1496E+00	0.9552E-04	0.5957E-02	0.5000E-04
0.2991E+00	0.1910E-03	0.1191E-01	0.1000E-03
0.1502E+02	0.9565E-02	0.5957E+00	0.5000E-02
0.2260E+02	0.1437E-01	0.8935E+00	0.7500E-02
0.3019E+02	0.1917E-01	0.1191E+01	0.1000E-01
0.7533E+02	0.4792E-01	0.2978E+01	0.2500E-01
0.1318E+03	0.9035E-01	0.5957E+01	0.5000E-01
0.1680E+03	0.1268E+00	0.8935E+01	0.7500E-01
0.1899E+03	0.1590E+00	0.1191E+02	0.1000E+00

0.2522E+03	0.3301E+00	0.2944E+02	0.2500E+00
0.2766E+03	0.5901E+00	0.5216E+02	0.5000E+00
0.2830E+03	0.7178E+00	0.5876E+02	0.6250E+00
0.2895E+03	0.8455E+00	0.6535E+02	0.7500E+00
0.3271E+03	0.1611E+01	0.1036E+03	0.1500E+01

RESULT FROM UPPER-BOUND LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.4156E-01	0.2259E-04	0.1702E-02	0.1000E-04
0.2078E+00	0.1129E-03	0.8510E-02	0.5000E-04
0.4156E+00	0.2259E-03	0.1702E-01	0.1000E-03
0.2096E+02	0.1134E-01	0.8510E+00	0.5000E-02
0.3156E+02	0.1704E-01	0.1276E+01	0.7500E-02
0.4212E+02	0.2273E-01	0.1702E+01	0.1000E-01
0.1044E+03	0.5665E-01	0.4255E+01	0.2500E-01
0.1766E+03	0.1041E+00	0.8510E+01	0.5000E-01
0.2154E+03	0.1417E+00	0.1276E+02	0.7500E-01
0.2336E+03	0.1729E+00	0.1702E+02	0.1000E+00
0.2750E+03	0.3384E+00	0.4119E+02	0.2500E+00
0.3037E+03	0.6003E+00	0.6995E+02	0.5000E+00
0.3076E+03	0.7269E+00	0.7378E+02	0.6250E+00
0.3114E+03	0.8535E+00	0.7761E+02	0.7500E+00
0.3441E+03	0.1617E+01	0.1103E+03	0.1500E+01

RESULT FROM LOWER-BOUND LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.1892E-01	0.1578E-04	0.6808E-03	0.1000E-04
0.9462E-01	0.7892E-04	0.3404E-02	0.5000E-04
0.1892E+00	0.1578E-03	0.6808E-02	0.1000E-03
0.9462E+01	0.7892E-02	0.3404E+00	0.5000E-02
0.1422E+02	0.1184E-01	0.5106E+00	0.7500E-02
0.1901E+02	0.1580E-01	0.6808E+00	0.1000E-01
0.4764E+02	0.3954E-01	0.1702E+01	0.2500E-01
0.8760E+02	0.7683E-01	0.3404E+01	0.5000E-01
0.1201E+03	0.1119E+00	0.5106E+01	0.7500E-01
0.1453E+03	0.1448E+00	0.6808E+01	0.1000E+00
0.2293E+03	0.3218E+00	0.1770E+02	0.2500E+00
0.2494E+03	0.5798E+00	0.3438E+02	0.5000E+00
0.2585E+03	0.7086E+00	0.4374E+02	0.6250E+00
0.2677E+03	0.8375E+00	0.5310E+02	0.7500E+00
0.3102E+03	0.1605E+01	0.9701E+02	0.1500E+01

DRILLED SHAFT INFORMATION

DIAMETER OF STEM	=	3.000	FT.
DIAMETER OF BASE	=	3.000	FT.
END OF STEM TO BASE	=	0.000	FT.
ANGLE OF BELL	=	0.000	DEG.
IGNORED TOP PORTION	=	20.000	FT.
IGNORED BOTTOM PORTION	=	0.000	FT.
AREA OF ONE PERCENT STEEL	=	10.180	SQ.IN.
ELASTIC MODULUS, Ec	=	0.360E+07	LB/SQ IN
VOLUME OF UNDERREAM	=	0.000	CU.YDS.

PREDICTED RESULTS

QS = ULTIMATE SIDE RESISTANCE;
QB = ULTIMATE BASE RESISTANCE;
WT = WEIGHT OF DRILLED SHAFT (FOR UPLIFT CAPACITY ONLY);
QU = TOTAL ULTIMATE RESISTANCE;
QBD = TOTAL ALLOWABLE LOAD USING A FACTOR OF SAFETY
APPLIED TO THE ULTIMATE BASE RESISTANCE;
QDN = TOTAL ALLOWABLE LOAD USING FACTORS OF SAFETY
APPLIED TO THE ULTIMATE SIDE RESISTANCE AND
THE ULTIMATE BASE RESISTANCE.

LENGTH (FEET)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	QBD (TONS)	QDN (TONS)	QU/VOLUME (TONS/CU.YDS)
21.0	5.50	10.44	124.02	134.46	51.78	45.52	24.45
22.0	5.76	21.21	128.92	150.14	64.19	51.46	26.06
23.0	6.02	32.31	133.83	166.14	76.92	57.53	27.59
24.0	6.28	43.72	138.73	182.45	89.97	63.73	29.03
25.0	6.55	55.44	142.59	198.03	102.97	69.70	30.25
26.0	6.81	67.32	145.26	212.58	115.74	75.35	31.23
27.0	7.07	79.25	146.60	225.85	128.11	80.56	31.95
28.0	7.33	91.21	147.05	238.26	140.23	85.50	32.50
29.0	7.59	103.21	147.05	250.26	152.23	90.30	32.96
30.0	7.86	115.24	147.05	262.29	164.26	95.11	33.39
31.0	8.12	127.30	147.05	274.35	176.32	99.94	33.80
32.0	8.38	139.39	147.05	286.43	188.40	104.77	34.19
33.0	8.64	151.49	147.05	298.54	200.51	109.61	34.55
34.0	8.90	163.61	147.05	310.65	212.62	114.46	34.90
35.0	9.16	175.74	147.05	322.78	224.75	119.31	35.22
36.0	9.43	187.88	147.05	334.92	236.89	124.17	35.53
37.0	9.69	200.02	147.05	347.06	249.03	129.02	35.82
38.0	9.95	212.16	147.05	359.20	261.17	133.88	36.10

39.0	10.21	224.29	147.05	371.34	273.31	138.73	36.36
40.0	10.47	236.42	147.05	383.47	285.44	143.58	36.61
41.0	10.74	248.54	147.05	395.59	297.56	148.43	36.85
42.0	11.00	260.64	147.05	407.69	309.66	153.27	37.07
43.0	11.26	272.73	147.05	419.77	321.74	158.11	37.28

RESULT FROM TREND (AVERAGED) LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.2758E-01	0.1583E-04	0.1430E-02	0.1000E-04
0.1379E+00	0.7913E-04	0.7148E-02	0.5000E-04
0.2758E+00	0.1583E-03	0.1430E-01	0.1000E-03
0.1379E+02	0.7914E-02	0.7148E+00	0.5000E-02
0.2072E+02	0.1188E-01	0.1072E+01	0.7500E-02
0.2771E+02	0.1585E-01	0.1430E+01	0.1000E-01
0.6942E+02	0.3965E-01	0.3574E+01	0.2500E-01
0.1269E+03	0.7690E-01	0.7148E+01	0.5000E-01
0.1719E+03	0.1116E+00	0.1072E+02	0.7500E-01
0.2015E+03	0.1431E+00	0.1430E+02	0.1000E+00
0.2823E+03	0.3117E+00	0.3545E+02	0.2500E+00
0.3212E+03	0.5721E+00	0.6674E+02	0.5000E+00
0.3305E+03	0.6998E+00	0.7670E+02	0.6250E+00
0.3477E+03	0.9797E+00	0.9411E+02	0.9000E+00
0.4020E+03	0.1895E+01	0.1493E+03	0.1800E+01

RESULT FROM UPPER-BOUND LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.3792E-01	0.1799E-04	0.2042E-02	0.1000E-04
0.1896E+00	0.8996E-04	0.1021E-01	0.5000E-04
0.3792E+00	0.1799E-03	0.2042E-01	0.1000E-03
0.1898E+02	0.9000E-02	0.1021E+01	0.5000E-02
0.2862E+02	0.1352E-01	0.1532E+01	0.7500E-02
0.3822E+02	0.1804E-01	0.2042E+01	0.1000E-01
0.9572E+02	0.4515E-01	0.5106E+01	0.2500E-01
0.1712E+03	0.8627E-01	0.1021E+02	0.5000E-01
0.2256E+03	0.1232E+00	0.1532E+02	0.7500E-01
0.2559E+03	0.1549E+00	0.2042E+02	0.1000E+00
0.3143E+03	0.3195E+00	0.4991E+02	0.2500E+00
0.3567E+03	0.5814E+00	0.9231E+02	0.5000E+00
0.3661E+03	0.7091E+00	0.1016E+03	0.6250E+00
0.3762E+03	0.9869E+00	0.1118E+03	0.9000E+00
0.4232E+03	0.1900E+01	0.1588E+03	0.1800E+01

RESULT FROM LOWER-BOUND LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.1762E-01	0.1373E-04	0.8169E-03	0.1000E-04
0.8812E-01	0.6865E-04	0.4085E-02	0.5000E-04
0.1762E+00	0.1373E-03	0.8169E-02	0.1000E-03
0.8812E+01	0.6865E-02	0.4085E+00	0.5000E-02
0.1322E+02	0.1030E-01	0.6127E+00	0.7500E-02
0.1764E+02	0.1373E-01	0.8169E+00	0.1000E-01
0.4425E+02	0.3436E-01	0.2042E+01	0.2500E-01
0.8362E+02	0.6772E-01	0.4085E+01	0.5000E-01
0.1182E+03	0.1001E+00	0.6127E+01	0.7500E-01
0.1466E+03	0.1312E+00	0.8169E+01	0.1000E+00
0.2499E+03	0.3038E+00	0.2099E+02	0.2500E+00
0.2857E+03	0.5629E+00	0.4117E+02	0.5000E+00
0.2949E+03	0.6905E+00	0.5175E+02	0.6250E+00
0.3191E+03	0.9724E+00	0.7646E+02	0.9000E+00
0.3808E+03	0.1890E+01	0.1397E+03	0.1800E+01