PRELIMINARY GEOTECHNICAL REPORT
PROPOSED CAMPUS MODERNIZATION PROJECT

Elizabeth Learning Center
4811 Elizabeth Street
Cudahy, California

Prepared for:

LOS ANGELES UNIFIED SCHOOL DISTRICT
333 South Beaudry Avenue, 22nd Floor
Los Angeles, CA 90017

Prepared by:

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GDC Project No. LA-1321

May 17, 2017

Los Angeles Unified School District
333 South Beaudry Avenue, 22nd Floor
Los Angeles, CA 90017

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Attention: Mr. Peyman Soroosh Moghadam, P.E., S.E.
Supervising Structural Engineer

Subject: Preliminary Geotechnical Report
Proposed Campus Modernization Project
Elizabeth Learning Center
4811 Elizabeth Street, Cudahy. California

Dear Mr. Moghadam:

Group Delta Consultants, Inc. (GDC) is pleased to submit this preliminary geotechnical report for the proposed campus modernization project planned for the Elizabeth Learning Center at 4811 Elizabeth Street, Cudahy, California. Our scope of work was conducted in general accordance with our proposal dated January 31, 2017 (LAUSD PO #4500300662).

We appreciate this opportunity to provide geotechnical and geologic services for your project. If you have any questions pertaining to this report, or if we can be of further service, please do not hesitate to contact us.

Sincerely,

GROUP DELTA CONSULTANTS, INC.

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ELIZABETH LEARNING CENTER
4811 ELIZABETH STREET, CUDAHY, CALIFORNIA

1.0 INTRODUCTION

1.1 General

This Preliminary Geotechnical Report has been prepared for the proposed campus modernization project at the Elizabeth Learning Center in Cudahy, California. The purpose of the Preliminary Geotechnical Investigation is to identify the geotechnical conditions within the campus and provide preliminary, but complete, recommendations for planning the overall project. Depending on the final details and the locations of new buildings, additional explorations may be required to meet the Code requirements concerning the minimum number of explorations required per building.

The general location of the school campus is shown on the Site Vicinity Map in Figure 1. The campus, existing structures, and the locations of our current exploratory borings/CPT are shown in Figure 2.

1.2 Project Description

At the time of this submittal, the proposed campus modernization project is in a preliminary planning stage and a design team has not yet been selected. It is our understanding that the design will conceptually consist of the modernization of existing campus buildings and construction of new buildings up to 3-stories in height. Some of the newly constructed buildings may have subterranean parking. The specific details regarding the elements of the project, including architectural and structural plans and structural loads, are not yet available.

1.3 Scope of Work

Our scope of work included the following:

- Review available published geologic and geotechnical reports, and geologic publications and maps pertaining to the site and surrounding area.
- Conduct a geotechnical field investigation to investigate the subsurface conditions at the site, which consisted of drilling four (4) hollow stem auger (HSA) borings to depths of 31.5, 51.5 and 71.5 feet and advancing six (6) Cone Penetration Test (CPT) soundings to depths between 70.5 and 98.8 feet.
- Perform laboratory tests on selected soil samples from the geotechnical field investigation to define the subsurface profile and to evaluate the physical properties and engineering characteristics of the soils encountered.
- Provide evaluation and recommendations regarding the geologic and seismic hazards

affecting the proposed campus modernization in accordance with the 2016 California Building Code (CBC), ASCE 7-10, and CGS Note 48, including a site-specific ground motion analysis.

- Provide geotechnical recommendations for site grading, soil removal, earthwork, excavations and shoring, retaining walls, and foundation design.
- Provide pavement design recommendations for TI's ranging from 4 to 7.
- Evaluate the expansion potential and corrosivity of soils that will be in contact with buried concrete or metals.
- Prepare and submit eight (8) copies of this report, along with an electronic copy.

It should be noted that when the elements/buildings for the Campus Improvement Project are finalized, it may be necessary to drill additional borings to meet the California Building Code (CBC) requirement that there is at least 1 boring per 5,000 square feet of building footprint. In addition, CGS will not provide their final approval until this requirement is met, and a description of the project elements is provided.

2.0 GEOLOGY AND SEISMIC SETTING

2.1 Regional Geology

The site is located within the seismically active Los Angeles Basin area of southern California. The basin formed over 7 million years ago during transtensional tectonism between northwest and east-west trending fault systems (Wright 1991). Today, the basin is undergoing transpressional stress, bound by surrounding uplifting thrust blocks including the Whittier, Palos Verdes, and Santa Monica-Hollywood-Raymond fault systems. Internally, the basin is filled with sedimentation thousands of feet thick structurally influenced by thrusting fault blocks and strike slip faults dividing the basin into northwest trending valleys and ridges (Dolan, et al., 1995). The location to the site with respect to regional geology is presented in Figure 4 Regional Geologic Map.

2.2 Local Geology

The site is centrally located within the Los Angeles Basin on a broad alluvial fan gently sloping south. Structurally the fan is bound by the Santa Monica-Hollywood-Raymond fault system in the north, the Newport-Inglewood fault zone to the west and the Elsinore fault zone to the east. The alluvial fan deposits derived from erosional debris transported southward from the Santa Monica Mountains. The Los Angeles River flows south, directed through a concrete lined channel, located about 0.6 miles east of the site. Paleo meandering and flooding of the river has also contributed to the alluvial deposits underlying the site. The location of the site with respect to local geology is presented in the, Figure 2.

2.3 Seismic Setting

2.3.1 Seismic Faults

The site is located within the seismically active area of southern California and there is a potential for the site to experience strong ground shaking from local and regional faults. A fault that is considered to be seismically active is one that has ruptured in the last approximate 11,000 years (Holocene). A fault that is considered to be potentially active is one that has ruptured in the last approximate 130,000 years. Current regional seismic conditions summarized here-in are largely based on data provided by the USGS online fault and fold database, unless otherwise noted. The location of the site with respect to regional faults with the potential for future seismic activity is presented in Figure 5 Regional Fault and Seismicity Map.

Seismically active faults nearest to the site include the Puente Hills, Elysian Park, Newport-Inglewood, Whittier, and Raymond faults. The closest active fault to the site is the Puente Hills Blind Thrust fault. It is comprised of a series of stepping thrust belts, buried below the surface dipping to the northeast. One of the fault segments surface trace is projected about 0.25 miles south of the site. The Puente Hills fault projects at depth beneath the site and is considered capable of generating a magnitude (M) 6.9 earthquake. The site may be subject to hanging wall effects during and following a significant earthquake event. The surface projection of the Lower Elysian Park Blind Thrust Fault is located approximately 1 mile northeast of the site and is capable of generating a M6.7 earthquake. The Puente Hills and Elysian Park faults are considered sources for the Whittier Narrows M5.3 and 5.9 earthquakes in 1987.

Newport-Inglewood Fault Zone, which is about 6.1 miles west of the campus. The Newport-Inglewood Fault is a northwest trending strike-slip fault capable of generating a M7.2 earthquake with an estimated slip-rate of 1.0-5.0 mm/yr. It is associated with the 1933 M6.7 Earthquake which ruptured near Newport Beach. Segments along this fault zone are identified under the CGS AP Earthquake Fault Zone Act.

The Whittier fault zone is located about 8 miles east of the site. It is the northwest segment of faulting associated with the Elsinore fault zone which trends northwest over 100 miles in length across southern California and Baja. It is estimated to be a right lateral strike-slip fault capable of potential M6.9 earthquake. The Raymond Fault is located about 10.7 miles north of the site, trending east-west over 16 miles in length. It is estimated to be a left lateral fault segment of the Santa Monica-Hollywood-Raymond fault system and is considered to have a potential to generate a M6.7 earthquake.

The San Andreas Fault is the most significant seismically active fault in the region. It stretches over 800 miles across the state of California and represents the boundary of the North American Tectonic Plate and the Pacific Tectonic Plate. It is over 40 miles northeast of the site, and considered capable of M7.9 earthquakes with an estimated slip-rate of 12.8 mm/yr in the southern San Bernardino section. Historical earthquakes of M7.0 and greater have been recorded on the San Andreas Fault, including the estimated M7.9 Fort Tejon Earthquake in 1857.

2.3.2 Seismic History

Local historic earthquake search was performed with the USGS online earthquake search catalog, on May 9, 2017. The search included earthquakes of magnitude (M) 4.0 or greater within a 100-km radius of the site. Since 1932, 310 earthquakes have been recorded, of which, three are M6.0 and greater including the M6.7 Northridge Earthquake in 1994. Twenty-six M5.0 to M6.0 earthquakes were recorded including the Whittier Narrows M5.9 earthquake in 1987 about 8.8 miles northeast of the site. The closest earthquake of M4.0 or greater to the site is a M4.0 in 1933, located about 2.3 miles southeast of the site. No earthquake related damage has been reported on the campus.

While not within the search radius, earthquakes of M7.0 and greater have been recorded in southern California, including the 1952 White Wolf M7.5 Earthquake and 1992 Hector Mine M7.3 Earthquake. Figure 5 illustrates the location of regional mapped faults and earthquake epicenters recorded by USGS.

3.0 SITE CONDITIONS

3.1 Surface Conditions

The campus is bordered to the north by Clara Street, to the south by Elizabeth Street, and to the east and west by residential development, as shown in Figure 1. The campus is approximately 16 acres and is currently comprised of classroom and administration buildings, parking lots paved in asphalt concrete, physical education buildings, sport fields/courts, a playground area, and a lunch shelter area, as laid out in Figure 2. The campus topography is relatively level, as shown in Figures 3.1 and 3.2.

3.2 Subsurface Conditions

3.2.1 Geotechnical Field Investigation

GDC conducted a geotechnical field investigation to assess the subsurface conditions at the project site on April 14, 2017. The field investigation consisted of drilling four (4) hollow-stem auger borings (B-1, B-3, B-6, and B-8) to depths ranging from about 31.5 to 71.5 feet and advancing six (6) cone penetrations test (CPT) soundings (CPT-02, CPT-04, CPT-05, CPT-07, CPT-09, and CPT-10) to depths ranging from about 70.5 to 98.8 feet. Our exploration locations are shown in Figure 2.

The explorations were performed under the continuous technical supervision of our field engineer, who maintained detailed logs of the soils encountered, classified the materials, and assisted in obtaining soil samples. Relatively undisturbed samples were taken in the borings at about 2.5-foot depth intervals above 15 feet and 5-foot depth intervals thereafter. Standard Penetration Tests (SPT) and representative bulk samples were also taken. Additional details of the field exploration program, including copies of the boring and CPT logs, are presented in Appendix A.

3.2.2 Laboratory Testing Program

Laboratory tests were performed on selected soil samples collected during our field investigation. The purpose of the laboratory tests was to classify soil samples and evaluate their physical properties and engineering characteristics. Laboratory testing included the following:

- Moisture Content and Dry Unit Weight;
- Atterberg Limits;
- Percent Passing No. 200 Sieve;
- Corrosion (pH, Sulfate, Chloride, Minimum Resistivity);
- Expansion Index;
- R-Value.

All testing was done in general accordance with applicable ASTM specifications. Details of the laboratory testing program and test results are presented in Appendix B.

3.2.3 Previous Field and Laboratory Data

GDC reviewed the following two geotechnical reports by Leighton Consulting Inc., (LCI) for new construction at the project site:

- "Geotechnical Investigation for the Proposed New Core Facilities Project at the Elizabeth Learning Center in the City of Cudahy, California" dated September 1, 2006, which provided geotechnical recommendations for construction of a new multi-purpose building.
- "Geotechnical Investigation for the Proposed New Core Facilities Project at the Elizabeth Learning Center in the City of Cudahy, California" dated May 1, 2007, which provided geotechnical recommendations for kitchen expansion and a new multi-purpose room.

The field investigation and laboratory testing program by LCI is summarized in the following sections.

3.2.3.1 Previous Field Investigation

LCI conducted a field investigation on August 11, 2006 that consisted of drilling two (2) hollow-stem auger borings and advancing one (1) CPT sounding to depths of about 50 feet bgs. LCI conducted another field investigation on February 15, 2007 that consisted of drilling two (2) hollow-stem auger borings and advancing two (2) CPT soundings to depths of about 50 feet bgs. Explorations locations are shown in Figure 2. Copies of boring and CPT logs are included in Appendix A.

3.2.3.2 Previous Laboratory Program

LCI performed the following laboratory tests on select soil samples from the previous field investigations described in Section 3.2.3.1:

- Moisture Content and Dry Unit Weight;
- Direct Shear;
- Grain Size Analysis;
- Corrosion (pH, Sulfate, Chloride, Minimum Resistivity);
- R-Value.

Test results are included in Appendix B.

3.3 Subsurface Soil Conditions

Generalized geologic cross-sections showing the subsurface conditions encountered in the field explorations are shown in Figures 3.1 and 3.2. Uncertified fill was encountered overlying native alluvium to a depth of about 1.5 feet at borings B-1 and B-3, a depth of about 3 feet at boring B-6, and a depth of about 2.5 feet at boring B-8. The fill generally consisted of sandy lean clay (CL). Deeper fills may be encountered between borings.

The alluvium generally consisted of interbedded poorly-graded sand (SP) to silty sand (SP-SM, SM), silt (ML) and lean clay (CL). The profile in the upper 15 feet consisted of mostly loose to medium dense poorly-graded sand (SP) and silty sand (SP-SM, SM). The profile below 15 feet consisted mostly of interbedded medium dense to very dense poorly-graded sand (SP) and silty sand (SP-SM, SM) and stiff to very stiff lean clay (CL) and silt (ML).

3.4 Groundwater

Groundwater was not encountered in the four borings drilled in the recent field investigation; however, perched water at a depth of about 43 feet was encountered in borings B-1 and B-3. Additionally, pore pressure dissipation tests (PPDTs) to estimate hydrostatic pore water pressure were performed at CPT-02 and CPT-09. Estimated water levels from the PPDTs ranged from 48 feet at CPT-02 to 50 feet at CPT-09.

The Seismic Hazard Report for the South Gate 7.5' Quadrangles, CGS SHZ Report 27, includes a map of the historical highest shallow groundwater levels at the site. The groundwater contour map indicates the depth to "the historically highest shallow ground water in perched, semi-perched, and other water table settings" in the vicinity of the project site is about 8 to 10 feet below the ground surface.

The historic high groundwater level of about 8 feet was used for design.

4.0 GEOLOGICAL HAZARD EVALUATION AND SEISMIC DESIGN

The geologic hazards evaluation for this project addresses requirements of Title 24 of the California Code of Regulations and California Geological Survey Checklist for Review of Geologic/Seismic Reports for California Public Schools, Hospitals, and Essential Services Building (CGS Note 48). A ground motion hazard analysis for the site was also performed in accordance with the 2016 California Building Code/ASCE 7-10, presented below in Section 5.7 Seismic Ground Motion Values.

4.1 Surface Fault Ground Rupture

Ground surface rupture potential at the site was evaluated with review of current CGS Fault Activity Map of California (2010), USGS online Fault and Fold database, and Alquist-Priolo (AP) Special Study Fault Zone Maps in the area. An active fault is defined as a fault with evidence for movement within the Holocene (last 11,000 years). The CGS considers active faults to have a high potential for future earthquakes capable of ground surface rupture. No known active faults are mapped crossing the site or projecting towards the site. Therefore, the possibility of ground surface fault rupture at the site is considered low.

4.2 Liquefaction and Seismic Settlement

Liquefaction involves sudden loss in strength of a saturated, cohesionless soil caused by the build-up of pore water pressure during cyclic loading, such as that produced by an earthquake. This increase in pore water pressure can temporarily transform the soil into a fluid mass, resulting in differential settlements and ground deformations. Typically, liquefaction occurs in areas where there are loose soils and the depth to groundwater is less than 50 feet from the surface. Seismic shaking can also cause soil compaction and ground settlement without liquefaction occurring, including settlement of dry sands above the water table.

The site is located within the State Earthquake Induced Liquefaction Seismic Hazard Zone for the South Gate Quadrangle, (shown in Figure 6). The historical high groundwater is about 8 and 10 feet below the ground surface.

The upwards of 60 feet of loose to medium-dense sand and silty sand overlaying dense sand contains a number of loose layers of varying thicknesses that are potentially susceptible to liquefaction. Therefore, the potential for liquefaction, lateral spreading, and seismic compaction to occur at the site is considerable.

The liquefaction potential was analyzed for the peak ground acceleration (PGA_M) of 0.74 g, using the simplified procedures recommended by NCEER (Youd and Idriss, 1997, 2001). To compute a mean magnitude to be used in analyses, we have deaggregated the seismic hazard curve at peak ground acceleration (T=0.01) using computer program EZ-FRISK (v7.65). A calculated mean magnitude of 6.66 was used. The site is classified as Site Class D, corresponding to a “stiff soil” profile, based on boring data and shear wave velocity interpretations using CPT data.

We have estimated the limited liquefaction assessment using soil profile obtained from the CPT performed in the current investigation and the computer software CLiq. For estimating seismic ground settlements, we used the method proposed by Zhang et al. (2002). The analysis was performed with a design groundwater depth of 8 feet (historic high). The results of the settlement analyses are provided in Appendix D. The predicted total seismically-induced settlement based on the historic high groundwater level using the CPT data are listed in the table below.

Table 3: Liquefaction/Seismic Settlements using CPT Data

CPT No.	Estimated Seismically-Induced Settlement (in)	Differential Settlements = 0.5xEst. Seismically-Induced Settlement (in)
CPT-02	2.9	1.45
CPT-04	2.8	1.4
CPT-05	2.4	1.2
CPT-07	1.8	0.6
CPT-09	2.8	1.4
CPT-10	3.7	1.85

The seismically-induced settlement of the site, as shown in the above table, may exceed the typical tolerance for structures supported on conventional shallow foundations. Alternatively, the proposed structures may be supported on deep foundations or a mat foundation.

No surface manifestation of liquefaction in the form of sand boils and no loss of bearing capacity is anticipated.

4.3 Lateral Spreading

Lateral spreading is characterized primarily by lateral movement of surficial soil layers of gently to steeply sloping saturated soil deposits as a consequence of liquefaction of a subsurface granular deposit. The site is situated within a relatively level alluvial plain. The closest significant body of water is the Los Angeles River, located about 0.6 miles east of the site. Here the river is directed through a concrete lined channel. Groundwater level at the site is generally below the channel floor, and the channel slopes are unsaturated. The potential hazard for lateral spreading at the site is considered low.

4.4 Landslides and Slope Stability

The project site is situated within a broad alluvial plain. Surrounding lots are relatively level as shown in Figure 1. There are no significant slopes that can present a landslide hazard at or near the site. Elevation ranges about a foot across the site. Therefore, the potential hazard for landslides and slope instability is not an issue at the campus.

4.5 Flooding and Inundation

Flooding and inundation potential at the site were evaluated through review of maps provided by FEMA (2008) and Los Angeles County Safety Element (1990). FEMA maps indicates the site is located outside the 0.2% Annual Chance of Floodplain. The Los Angeles County Flood and Inundation Hazard Map indicates the site is within a potential flood and inundation zone. The flood and inundation zone is related to the Hansen Dam. The Hansen Dam has undergone seismic retrofitting since its original construction according to the Los Angeles Department of Water and Power. However, if the reservoir was breached, flood waters would travel downstream toward the site. Dams are routinely inspected and continually evaluated for safety in compliance with the Federal Guidelines for Dam Safety issued in 1979 and Engineering Regulation ER 1110-2-1156, Safety of Dams – Policy and Procedures. The Hansen Dam is under the jurisdiction of the U.S. Army Corps (Corps) and has a Dam Safety Action Class III (DSAC III) rating as of March 2009 based on a Screening Portfolio Risk Analysis (SPRA) completed in May 2008. A DSAC III rating is given to dams that have issues which are “conditionally unsafe” and where the dam is “significantly inadequate, or the combination of life, economic or environmental consequences with probability of failure is moderate to high”. However, the Hansen Dam is not under emergency status. It is presently under regular observation, maintenance, and local emergency management.

The City of Los Angeles Hazard Mitigation Plan (2011) indicates dam failure is a moderate risk hazard. The site is not located within an inundation zone defined within the Plan Figure 7 M-1 Dam Inundation Hazard Areas. While dam failure has the potential to be a significant hazard to the site, through continued observation and regular maintenance of the Hansen Dam, the potential for inundation hazard to occur at the site is considered low.

4.6 Tsunami and Seiche

Low-lying areas along California's coast are subject to potentially dangerous tsunamis. The site is located about 14 miles east from the Pacific Ocean/Los Angeles coast. The Elevation of the site is about 130 feet. Therefore, the potential for a Tsunami is not considered an issue for the site. Since there are no large bodies of water near the site, the potential for a seiche event is also not considered an issue.

4.7 Soil Expansion and Collapse

Boring and CPT data indicates the soil is not susceptible to potential collapse. Soil expansion potential was tested at boring B-1 at 31-31.5 feet depth and B-6 at 0-3 feet and 12.5-14 feet depth and found to be non-expansive. The results of the lab tests are presented in Appendix B.

4.8 Soil Corrosivity

A bulk soil sample was collected in boring B-6 at 0-3 feet depth and tested for corrosivity potential. The soil chloride content indicates the soil corrosivity to metals is negligible. However, the soil resistivity and sulfate content indicates the soil is corrosive to ferrous metals and cement. The results of the lab tests are presented in Appendix B.

4.9 Other Geologic Hazards and Considerations

Naturally occurring hazardous elements within the subsurface materials, such as asbestos, radon, and oil and methane gas were evaluated for the potential presence on or near the site. California Geological Survey Map Sheet 59, of known sites with naturally occurring asbestos does not indicate there is a potential for naturally occurring asbestos to be at the site and the hazard is considered to be low. California Geological Survey Special Radon Potential Zone Map indicates the site is within a low radon potential area. Review of the Division of Oil, Gas and Geothermal Resources Regional Wildcat Map indicates the site is outside field boundaries, productive boundaries, and drilling sites. One active well is located about 0.25 miles south of the site. Three other wells are located within 0.5 miles of the site and are either plugged or buried. No wells are located on the campus. Therefore, the occurrence of naturally occurring oil and or methane gases onsite is considered low.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 General

The total predicted dynamic settlement that could be traced to the ground surface is generally about 2 to 3 inches, but could be up to approximately 4 inches locally and may not be tolerable for structures supported on conventional shallow foundations. Therefore, it may not be suitable for structures supported on conventional shallow foundations without ground improvement. For areas where computed total static-plus-seismic settlement is less than 2 inches, such as the southwest portion of the campus, conventional shallow spread footing or mat foundation could be applicable, depending on the loading from the proposed structure. The option to use conventional shallow spread footing and/or a mat foundation should be confirmed by a more comprehensive geotechnical field investigation during design phase.

Alternatively, the proposed structures may be supported on piles. Both cast-in drilled hole (CIDH) piles and/or Auger Cast Displacement (ACD) piles may be used for support of the proposed structures.

Driven piles are relatively long, slender columns used to offer support and/or to resist forces. They are generally made of preformed material having a predetermined shape and size that can be physically inspected prior to and during installation. Driven piles are typically installed by impact hammering, vibrating or pushing the pile into the earth. If adjacent structures are sensitive to vibration, driven piles will not be a suitable option for the support the proposed structures.

Downdrag loads on pile foundations can be an important design consideration when earthquake-induced liquefaction is expected to cause ground settlements. For design of piles, the additional downdrag load should be considered and added to the service level (or allowable level) structural demand.

In conclusion, the following are possible foundation options for the conceptual design:

- Conventional spread footings/mat foundations for the western portion of the Campus;
- Conventional spread footings and/or mat foundation with ground improvement for the remaining portions of the campus;
- Deep foundation (CIDH or ACD piles).

More discussion about foundation recommendations is provided in Section 5.8.

5.2 Demolition

Prior to the start of grading, demolition will be required to remove existing improvements, which may include existing pavement, fences, etc. Any void created from the demolition should be properly backfilled to the limits determined by the project geotechnical engineer. Any soils loosened or disturbed during the demolition should also be removed.

5.3 Removals

Prior to the start of grading, the new building sites should be stripped of any vegetation and topsoil. The topsoil may be stockpiled and reused in landscaped areas. It should be anticipated that existing fill may be present anywhere on the site, and could be locally deep. Existing undocumented fill should be considered as unsuitable for use unless otherwise noted by the project Geotechnical Engineer, and should not be used to support foundations, pavement, and hardscape without removal and recompaction.

If the proposed structures are being supported on pile foundations, no removal will be required for foundation support. However, future distress of slab-on-grade could be expected resulting from either dynamic settlement or settlement within existing undocumented fill soils. Therefore, we recommend that the floor slab be structurally supported.

In pavement areas, the removal and recompaction of uncertified fill should extend to a minimum depth of 2 feet below pavement level. All removals should extend a minimum of 5 feet outside building and pavement areas, or a distance equal to the depth of excavation, whichever is greater. The actual limits for removals should be determined by the project geotechnical engineer during grading, based on the actual conditions encountered.

The civil engineer should identify the presence and location of all existing utilities in and near the work area. Precautions should be taken to remove, relocate or protect existing utilities, as appropriate.

5.4 Earthwork

All grading should conform to the requirements of the 2016 California Building Code and the general grading recommendations outlined below.

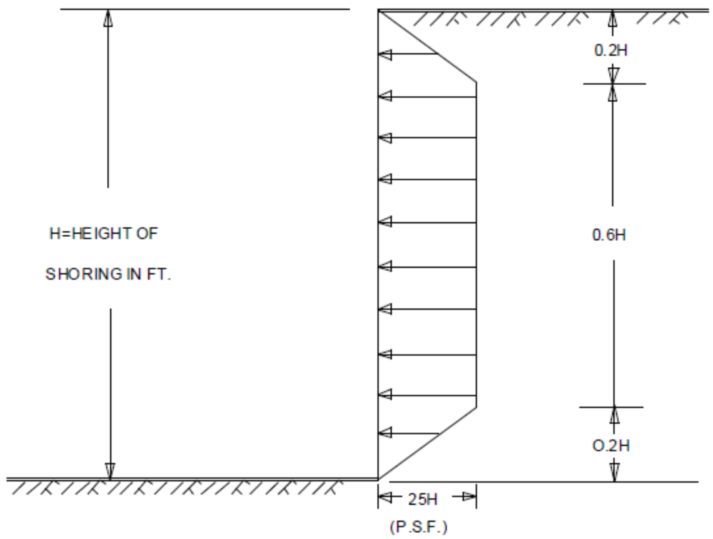
- The contractor is responsible for notifying the project geotechnical engineer of a pre-grading meeting prior to the start of construction and grading operations and anytime that the operations are resumed after an interruption.
- The geotechnical engineer should determine the limits for the removal and recompaction, based on the actual conditions encountered.
- Temporary excavations should be sloped at 1H:1V or flatter, or shoring should be used.
- The bottom of the excavation for removals should be observed and approved by the project geotechnical engineer. Any loose or yielding soils should be overexcavated and recompacted to the limits determined by the project geotechnical engineer.
- All structural fill should consist of generally sandy soils, and should be free of expansive clay, rock greater than 3 inches in maximum size, debris and other deleterious materials. All structural fill should be compacted to at least 95 percent of

the maximum dry density determined by ASTM D 1557. Fill placed in non-structural and landscape areas should be compacted to at least 90 percent.

- In general, the near surface soils encountered in our explorations were found to consist of silty sand, sand, sandy silt and silt, and should be acceptable for use in new compacted fill.
- If imported soils are used as structural fill, the soils shall be free of vegetation organic materials, expansive clay, debris, or rocks greater than 3 inches in any dimension, and shall be approved by the project Geotechnical Engineer. Imported soils shall have an expansion index of less than 30 and plasticity index of less than 15. All fill soils shall be approved by the project geotechnical engineer before use.
- All earthwork and grading should be performed under the observation of the project geotechnical engineer. Compaction testing of the fill soils shall be performed at the discretion of the project geotechnical engineer. At a minimum, testing should be performed for approximately every 2 feet in fill thickness or 500 cubic yards of fill placed, whichever is more restrictive. If specified compaction is not achieved, additional compactive effort, moisture conditioning, and/or removal and recompaction of the fill soils will be required.
- All materials used for asphalt, concrete and base shall conform to the current "Green Book," and shall be compacted to at least 95 percent relative compaction.
- If, in the opinion of the geotechnical engineer, contractor, or owner, an unsafe condition is created or encountered during grading, all work in the area shall be stopped until measures can be taken to mitigate the unsafe condition. An unsafe condition shall be considered any condition that creates a danger to workers, on-site structures, on-site construction, or any off-site properties or persons.

5.5 Temporary Excavation and Shoring

If shoring is required, either cantilever or braced shoring can be utilized. Cantilever shoring should be designed for an active earth pressure equivalent to a fluid weighing 30 pcf. Braced temporary shoring should be designed for a lateral earth pressure of 25H, applied as a trapezoidal pressure distribution as shown in the figure below. These lateral earth pressures assume level backfill and drained conditions and do not include surcharging from adjacent loads.



Surcharge loads, such as vehicular traffic, heavy construction equipment, and stockpiled materials, should be kept away from the top of temporary excavations a horizontal distance at least equal to the depth of excavation. Surface drainage should be controlled and prevented from running down the slope face. Ponding water should not be allowed within the excavation. Construction equipment and foot traffic should be kept off excavation slopes to minimize disturbance/sloughing.

All excavation slopes and shoring systems should meet the minimum requirements of the Occupational Safety and Health (OSHA) Standards. Maintaining safe and stable slopes on excavations is the responsibility of the contractor and will depend on the nature of the soils and groundwater conditions encountered and his method of excavation. Excavations during construction should be carried out in such a manner that failure or ground movement will not occur. The contractor should perform any additional studies deemed necessary to supplement the information contained in this report for the purpose of planning and executing his excavation plan.

5.6 Retaining Walls

5.6.1 Earth Pressure

Cantilever walls that are free to move laterally at least ½ inch for each 10-feet in height, may be designed for an equivalent fluid pressure of 30 pcf for level ground. Walls 6 feet or greater in height should be designed for seismic loading. Seismic earth pressure may be computed from Mononobe-Okabe method using horizontal seismic coefficient. We have used a horizontal seismic coefficient equal half of design acceleration. PGA_M of 0.74g was used for the design acceleration. Hence, a horizontal seismic coefficient of 0.34g has been used in the computation. Therefore, a seismic increment of 10 pcf may be used for design.

5.6.2 Wall Backfill

We recommend that retaining walls be backfilled with non-expansive granular soils with a (Plasticity Index) PI less than 15 and percent passing No. 200 sieve of less than 15 percent. A 2-ft thick cap consisting of less pervious onsite materials should be used to minimize infiltration of surface water. The finished surface should be graded to drain away from the proposed structures. Heavy compaction equipment operating adjacent to retaining walls can cause excessively high lateral soil pressures to be exerted on the wall. Therefore, soils within 5 feet of the wall should either be compacted with hand operated equipment or designed to withstand compaction pressure from heavy equipment.

5.6.3 Drainage

Retaining walls should be constructed with a properly designed drainage system to prevent buildup of hydrostatic pressures behind the wall. This may consist of geocomposite drain board or 12 inches of clean crushed rock encapsulated in filter fabric, discharging to weep holes or drain pipes. Basement walls or walls with architectural facades or coverings should be properly waterproofed to minimize moisture transmission through the walls.

5.7 Seismic Ground Motion Values

5.7.1 Site-Specific Ground Motion Seismic Parameters

A site-specific acceleration response spectrum was constructed in accordance with ASCE 7-10 Chapter 21, as described in Appendix C. The summary of the Design Acceleration Parameters is the following:

$$S_{DS}= 1.14 \text{ and, } S_{D1}= 1.08$$

The site-specific design spectrum is summarized in the Table 1 and provided in Appendix C.

Table 1: Site-Specific Design Spectrum

Period (s)	Design Earthquake Sa (g)
0.01	0.54
0.05	0.72
0.06	0.78
0.08	0.90
0.1	1.02
0.125	1.06
0.15	1.06
0.2	1.14
0.25	1.18
0.3	1.20
0.4	1.19
0.5	1.23
0.75	1.11
1	0.98
1.5	0.71
2	0.54
2.5	0.42
3	0.35
3.5	0.29
4	0.26

5.7.2 Ground Motion Seismic Parameters per CBC 2016/ASCE 7-10 (Code Values)

Design ground motion parameters were also developed in accordance with CBC 2016 / ASCE7-10 for the proposed project. The site coordinates used in our seismic hazard analysis are: -118.18305 (Longitude) and 33.9635 (Latitude). The site is classified as Site Class D, corresponding to a “stiff soil” profile based on shear wave velocity interpretations using CPT data.

The seismic design parameters were calculated using the USGS Ground Motion Parameter Calculator (Version 5.1.0), are summarized in Table 2.

Table 2: Seismic Ground Motion Values

Latitude: 33.9635 Longitude: -118.18305	
Site Class	D
Seismic Design Category	D
Mapped MCE Spectral Response Acceleration at Short Period (S_s)	1.98g
Mapped MCE Spectral Response Acceleration at Period of 1 Second (S_1)	0.697g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Adjusted MCE Spectral Response Acceleration at Short Period (S_{MS})	1.98g
Adjusted MCE Spectral Response Acceleration at Period of 1 Second (S_{M1})	1.046g
Design Earthquake Spectral Response Acceleration at Short Period (S_{DS})	1.32g
Design Earthquake Spectral Response Acceleration at Period of 1 Second (S_{D1})	0.697g
Peak Ground Acceleration Adjusted for Site Class (PGA_M)	0.742g

5.8 Foundation Recommendations

5.8.1 General

As discussed in Section 5.1 of the report, the foundation options consist of the following items:

- Conventional spread footings/mat foundations for the western portion of the Campus;
- Conventional spread footings and/or mat foundation with ground improvement for the remaining portions of the campus;
- Deep foundation (CIDH or ACD piles).

More discussions are provided below.

5.8.2 Conventional Spread Footings on Compacted Fill Soils – Light Structures at West Portion of the Campus

Based on the liquefaction evaluation discussed in Section 4.2 of this report, total seismically-induced settlement near the western portion of the campus may be less than 2 inches. Therefore, a lightweight structure may be supported on conventional shallow spread footings in this area. However, this foundation option should be confirmed in a more comprehensive geotechnical investigation during design phase.

5.8.2.1 Bearing Value

Spread footings established on at least 3 feet thick of properly compacted fill soils and at least 2 feet below the lowest adjacent grade or floor level may be designed to impose a net dead-plus-live load pressure of 2,000 pounds per square foot. The excavations should be deepened as necessary to extend into satisfactory soils. A one-third increase can be used for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

5.8.2.2 Settlement

We estimate the total static-plus-seismic settlement will be on the order of 2 inches. The structure should be designed to accommodate differential settlement of 1 inch.

5.8.2.3 Lateral Resistance

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 can be used between the footings and the floor slab and the supporting soils. The passive resistance of natural soils or properly compacted fill soils can be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. A one-third increase in the passive value can be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

5.8.3 Mat Foundations on Compacted Fill Soils – West Portion of the Campus

5.8.3.1 Bearing Value

A mat foundation established on at least 3 feet thick of properly compacted fill soils and at least 2 feet below the lowest adjacent grade may be designed to impose a net dead-plus-live load pressure of 1,500 pounds per square foot. The excavations should be deepened as necessary to extend into satisfactory soils. A one-third increase can be used for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

5.8.3.2 Settlement

We estimate the total static-plus-seismic settlement will be on the order of 4 inches. The structure should be designed to accommodate differential settlement of 1 inch across the mat foundation.

5.8.3.3 Lateral Resistance

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 can be used between the mat foundation and the supporting soils. The passive resistance of natural soils or properly compacted fill soils can be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. A one-third increase in the passive value can be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

5.8.4 Ground Improvement – Campus Wide

For new structures not located within the western portion of the campus and supported on conventional spread footings or a mat foundation, we recommend that ground improvement be performed to mitigate the potential for liquefaction, liquefaction-induced settlement and seismically-induced settlement. If effective, soil improvement could limit seismically-induced settlement to less than 1 inch, with differential settlement of less than ½ inch. To achieve this improvement, we recommend that ground improvement be performed to a depth of 30 feet below the existing grade. Depending on the improvement type selected, the zone of improvement may also need to extend laterally beyond the edge of each structure. All utilities should be designed with flexible connections. All utilities should be designed with flexible connection capable of withstanding at least 6 inches of deformation at the point at which they encroach on a zone of improved soils.

5.8.4.1 Soil Mixing

Soil mixing involves introducing a cement-based slurry into the soil and mixing it, using single or multiple augers, to create a stable soil-cement mass. Soil-cement with unconfined compressive strengths ranging between 10 psi to 500 psi are possible depending on the soil type and binder content.

Soil mixing can also treat a wide variety of soil types and is safe to use adjacent to existing buildings without adverse effects, such as vibrations or soil heave. In addition, due to the fact that a large, relatively high-strength mass of soil is created, it would not be necessary to extend the area of improvement beyond the footprint of the building. However, because of the relative high-cost of this method, it could be combined with a densification method to provide a more economical overall design.

As with other soil improvement methods, the soil improvement contractor will design the mix proportions, depth, spacing, and size of the zone of treatment based on the target foundation design parameters and their design requirements.

5.8.4.2 Compaction Grouting

Compaction grouting is perhaps the most cost-effective method of mitigating liquefaction potential. The method involves the injection of a high-pressure, low-slump grout into the soil at depth. The resulting grout bulb displaces the densifies the surrounding soil.

The process of compaction grouting starts with the insertion of grout pipes to the design depth. The low-slump grout is then injected into the surrounding soil at a pre-determined pressure. The grout pipes are then withdrawn incrementally. Unlike replacement-type methods of ground improvement, the grout bulbs displace the surrounding soil and the zone of improvement is larger than the grout bulb itself. Therefore, a much larger mass of soil can be improved via this method than with other grouting methods, such as soil mixing and jet grouting.

One major limitation of this method is that the in-situ vertical stress must be sufficient to limit ground heave and induce lateral displacement and densification of the surrounding soil. This limitation defectively prevents the use of this method within 10 to 15 feet of the ground surface.

Compaction grouting can be effective in a variety of soil conditions and generally requires less installation time than other methods of soils improvement. If compaction grouting is selected, the soil improvement contractor will design the width and spacing of the compaction grout columns based on the target foundation design parameters and their design requirements.

For compaction grouting, the improved soil zone would need to extend beyond the edge of the structure a distance of at least ½ of the depth of improved soil.

5.8.4.3 Jet Grouting

Jet grouting can replace potentially liquefiable soils with cylinders of hardened soils, or soilcrete, by injecting a cement slurry at depth and mixing it with the surrounding soils. Soilcrete columns of more than 5 feet in diameter can be achieved in loose soils. Use of this method would be ideal in confined spaces or next to sensitive structures due to the lack of harmful vibrations, the limited space required, and the ability to maneuver safely around buried utilities. For these reasons, this method has been used in the past to underpin and rehabilitate existing structures. In addition, jet grouting is much faster than other methods of soil improvement. However, the cost is generally lower for jet grouting than for other forms of soil improvement.

Jet grouting uses high-pressure water to cut the soils, mix in the cement slurry, and lift the soil cuttings to the surface. Treatment of most soil types is achievable by controlling the rate of rotation and withdrawal of the nozzle. The soilcrete column can be interconnected with adjacent columns to create a high-strength soilcrete mass. Because of the relatively high-cost of this method, it could be combined with a densification method to provide a more economical overall design.

As with other soil improvement methods, the soil improvement contractor will design the mix proportions, depth, spacing, and size of the zone of treatment based on the target foundation design parameters and their design requirements.

For jet grouting, the improved soil zone would not need to extend outside of the limits of the structure.

5.8.5 Deep Foundation - Auger Cast Displacement (ACD) Piles

5.8.5.1 Axial Capacity

The ultimate downward capacities of 14-, 16-and 18-inch-diameter ACD pile as a function of penetration below bottom of pile cap are presented on Figures 7.1 through 7.3. The ultimate upward capacities are presented on Figure 8.1 through 8.3. The pile capacities shown on Figures 7.1 to 8.3 are for dead-plus-live load capacities; a one-third increase may be used for wind or seismic loads. The capacities presented are based on the strength of the soils; the compressive and tensile strengths of the pile sections should be checked to verify the structural capacity of the piles.

To compute allowable downward capacities, a factor of safety for soil bearing values should be 2 or shall not be less than the overstrength factor (Ω) of the structures supported, which is greater. For allowable upward capacities, a minimum factor of safety of 3 should be used unless the uplift is due to wind or seismic loading, which minimum factor of safety of 2 can be used.

Pile resistance impacted by liquefaction potential is not considered in the skin friction and end bearing. The location of the neutral plane, defined as a plane where there is no relative movement between the soils and the piles, was calculated based on the results of our liquefaction analyses. The portions of the piles above the neutral plane could experience downdrag load when earthquake-induced liquefaction is expected to cause ground settlements which occurs excess pore pressure due to liquefaction dissipated. This additional downdrag load should be added to the allowable structural demand.

The estimated downdrag load for each of 14-, 16- and 18-inch-diameter ACD pile are shown in the table below:

Pile Dimension	Downdrag Load (kips)
14-inch ACD Pile	115
16-inch ACD Pile	130
18-inch ACD Pile	145

5.8.5.2 Lateral Capacity

The lateral capacity of the recommended piles was evaluated using the computer program LPILE v2016 (Ensoft, 2016). The lateral capacities at 0.25 inches and 0.5 inches of pile head deflection, for both fixed head and free head conditions, and for single and grouped piles, are provided in the table below. To utilize a fixed head condition, the pile and pile cap connections must be able to translate laterally without rotation, and be designed for the fixed head moment.

TABLE. LATERAL PILE RESISTANCE - ACD PILES

Pile Type	Pile Head Condition	Pile Head Deflection (inch)	Single Pile			Grouped Pile		
			Max. Shear (kips)	Max. Moment (kip-ft)	Depth to Max. Moment (feet)	Max. Shear (kips)	Max. Moment (kip-ft)	Depth to Max. Moment (feet)
14-inch ACD	Free	0.25	8	26	5	6	21	6
		0.5	13	46	5	9	36	6
	Fixed	0.25	21	72	0	14	55	0
		0.5	33	123	0	22	94	0
16-inch ACD	Free	0.25	11	37	5	8	29	6
		0.5	17	64	6	12	49	7
	Fixed	0.25	26	99	0	18	76	0
		0.5	40	168	0	27	127	0
18-inch ACD	Free	0.25	14	51	6	10	38	7
		0.5	22	85	6	14	64	7
	Fixed	0.25	32	132	0	21	100	0
		0.5	49	219	0	32	165	0

For the pile group analyses, the lateral pile capacity was reduced based on an assumed spacing between piles. Assuming a 3 by 3 pile group, and a pile center-to-center spacing of 3D (where D is the pile diameter), a p-multiplier of 0.55 was used. If piles are spaced at a center to center spacing of 7D or greater, no reduction in lateral pile capacity is required (i.e., single pile system). It is recommended the project structural engineer verify the maximum moment capacity of the pile.

5.8.5.3 Pile Settlement

We estimate the settlement of pile foundations to be less than ½ inch. Details of pile foundations being contemplated should be provided to us so that additional settlement estimates can be made.

5.8.5.4 Pile Load Testing Program

We recommend that a static axial pile load testing program be completed prior to installation of production piles. The pile load testing criteria for ACD piles are summarized and discussed below:

- Number of static load tests required:

Total Production Piles	No. of Static Load Tests Required
<100	1
101-300	2
301-1000	3
1001-2000	4
2001-4000	5

- Minimum one (1) pile load test shall be performed per 30,000 square feet of building footprint;
- Gamma-Gamma Test and Low Strain Integrity Test shall be conducted on all test piles and reaction piles;
- Low Strain Integrity Test shall be performed on 10% of the production piles.

The testing program would be carried out as a separate mobilization by the pile contractor. GDC envisions that the testing program will require approximately 8 hours to perform each pile load test in the field plus an additional week of geotechnical analyses by GDC to provide the pile-length and allowable load recommendations.

In addition to testing each pile to the ASTM 1143-81 standards, a creep test is recommended at the allowable load. The creep test holds the allowable load for at least two hours to demonstrate displacement of the test pile slows to less than 0.005 inch per hour, which is half the rate recommended in ASTM 1143-81.

GDC should monitor the test and production-pile installations to verify that piles are installed in accordance with the geotechnical recommendations and have achieved a satisfactory penetration depth.

5.8.5.5 ACD Pile Installation Monitoring

The installation of ACD piles should be monitored by automated Pile Installation Recorder (PIR) equipment supplied by the pile installation contractor. During drilling, the PIR should record drill torque, depth and elapsed time (and drill rate). During placement of grout into the dilled shaft, the PIR should record the grout pressure, incremental grout volume pumped, volume ratio for each increment, and the elapsed time (and withdrawal rate).

Grout flow volume shall be measured and recorded by means of a magnetic flow meter for increments not exceeding 1-foot of pile length, as a means of verifying that grout volumes pumped are sufficient to fully replace the displaced soil. A grouting factor of safety of 1.05 shall be used to increase the volume of grout pumped into each 1-foot increment by 5% during withdrawal. In the event of interrupted or stopped grouting, or if the monitoring equipment detects a low grout volume for any depth increment, the displacement auger shall be re-advanced five (5) feet past the zone before continuing the grouting operation. A replacement pile shall be installed if excessive bleeding (accumulation of water or laitance at the top of the pile) is observed.

Grout mix and installation characteristics of the grout should also be monitored and grout strength should be verified by performing compression tests on samples taken.

5.8.6 Deep Foundation – Cast-in-Drilled-Hole (CIDH) Piles

5.8.6.1 Axial Capacity

The ultimate downward capacities of 24-, 30-and 36-inch-diameter CIDH pile as a function of penetration below bottom of pile cap are presented on Figures 9.1 through 9.3. The ultimate upward capacities are presented on Figure 10.1 through 10.3. The pile capacities shown on Figures 9.1 to 10.3 are for dead-plus-live load capacities; a one-third increase may be used for wind or seismic loads. The capacities presented are based on the strength of the soils; the compressive and tensile strengths of the pile sections should be checked to verify the structural capacity of the piles.

To compute allowable downward capacities, a factor of safety for soil bearing values should be 2 or shall not be less than the overstrength factor (Ω) of the structures supported, which is greater. For allowable upward capacities, a minimum factor of safety of 3 should be used unless the uplift is due to wind or seismic loading, which minimum factor of safety of 2 can be used.

Pile resistance impacted by liquefaction potential is not considered in the skin friction and end bearing. The location of the neutral plane, defined as a plane where there is no relative movement between the soils and the piles, was calculated based on the results of our liquefaction analyses. The portions of the piles above the neutral plane could experience downdrag load when earthquake-induced liquefaction is expected to cause ground settlements which occurs excess pore pressure due to liquefaction dissipated. This additional downdrag load should be added to the allowable structural demand.

The estimated downdrag load for each of 24-, 30- and 36-inch-diameter CIDH pile are shown in the table below:

Pile Dimension	Downdrag Load (kips)
24-inch CIDH Pile	190
30-inch CIDH Pile	240
36-inch CIDH Pile	288

5.8.6.2 Lateral Capacity

The lateral capacity of the recommended piles was evaluated using the computer program LPILE v2016 (Ensoft, 2016). The lateral capacities at 0.25 inches and 0.5 inches of pile head deflection, for both fixed head and free head conditions, and for single and grouped piles, are provided in the table below. To utilize a fixed head condition, the pile and pile cap connections must be able to translate laterally without rotation, and be designed for the fixed head moment.

TABLE. LATERAL PILE RESISTANCE - CIDH PILES

Pile Type	Pile Head Condition	Pile Head Deflection (inch)	Single Pile			Grouped Pile		
			Max. Shear (kips)	Max. Moment (kip-ft)	Depth to Max. Moment (feet)	Max. Shear (kips)	Max. Moment (kip-ft)	Depth to Max. Moment (feet)
14-inch ACD	Free	0.25	26	107	7	17	78	7
		0.5	37	170	7	25	123	8
	Fixed	0.25	54	269	0	35	201	0
		0.5	78	433	0	51	325	0
16-inch ACD	Free	0.25	38	177	7	25	131	9
		0.5	56	281	8	36	207	11
	Fixed	0.25	79	461	0	51	345	0
		0.5	113	737	0	74	554	0
18-inch ACD	Free	0.25	51	261	9	34	196	11
		0.5	76	425	10	50	318	13
	Fixed	0.25	106	710	0	69	532	0
		0.5	151	1133	0	99	850	0

For the pile group analyses, the lateral pile capacity was reduced based on an assumed spacing between piles. Assuming a 3 by 3 pile group, and a pile center-to-center spacing of 3D (where D is the pile diameter), a p-multiplier of 0.55 was used. If piles are spaced at a center to center spacing of 7D or greater, no reduction in lateral pile capacity is required (i.e., single pile system). It is recommended the project structural engineer verify the maximum moment capacity of the pile.

5.8.6.3 Pile Settlement

We estimate the settlement of pile foundations to be less than ½ inch. Details of pile foundations being contemplated should be provided to us so that additional settlement estimates can be made.

5.8.6.4 Pile Installation

Caving may be anticipated during drilling below groundwater. Special technique, such as casing or drilling mud, may be used to prevent caving.

Piles spaced less than five diameters on center should be drilled and filled alternately, with the concrete permitted to set at least 8 hours before drilling an adjacent hole. The pile installation should be completed the same day that the drilling is performed. A collar should be placed around the mouth of the shaft after drilling to prevent soils from entering the excavation, and the pile shafts should be covered until concrete is placed.

Concrete should be pumped from the bottom up through a rigid pipe extending to the bottom of the drilled excavation, with the pipe being slowly withdrawn as the concrete level rises. The discharge end of the pipe should be at least 5 feet below the surface of the concrete at all times during placement. The concrete pump pressure should be at least 200 pounds per square inch. The discharge pipe should be kept full of concrete during the entire placing operation and should not be removed from the concrete until all of the concrete is placed and fresh concrete appears at the top of the pile. The volume of concrete pumped into the hole should be recorded and compared to design volume.

Only competent drilling contractors with experience in the installation of drilled cast-in-place piles should be considered for the pile construction. The drilling of the pile excavations and the placing of the concrete should be observed continuously by personnel of our firm to verify that the desired diameter and depth of piles are achieved.

5.9 Site Drainage

The site should be graded to maintain positive drainage, so all runoff is properly collected and conveyed to proper disposal in approved storm drains or drainage devices. The area around foundations should be sloped at 2 percent to drain runoff away and prevent ponding of water.

5.10 Expansive Soil

Based on recent and previous laboratory testing, the near surface sandy and silty soils have a tested Expansion Index (EI) of 0, which indicates a very low expansion potential.

5.11 Soil Corrosivity

One representative sample of the near surface soils encountered was tested to evaluate corrosion characteristics. The results indicate the test sample had a pH of 9.08; a water-soluble sulfate content of 1.15%, and a soluble chloride content of less than 0.01%. The sulfate results indicate that sulfate exposure is classified as severe.

The tested sample was also found to have a minimum measured electrical resistivity of 2,046 Ohm-cm. The following correlation can generally be used between electrical resistivity and corrosion potential:

Elect. Resistivity (Ohm-cm)	Corrosion Potential
less than 1,000	Severe
1,000-2,000	Corrosive
2,000-10,000	Moderate
Greater than 10,000	Mild

On the basis of the laboratory testing, the test samples are classified as moderately corrosive to buried metals. Our testing was for screening purposes only. The need for further evaluation and testing and the development of alternatives for corrosion protection should be provided by a corrosion consultant.

5.12 Utility Installations

If new buried service lines will be installed, the bedding should be a minimum of 4 inches thick and should consist of clean sand, No. 4 concrete aggregate or gravel, and should have a sand equivalent of not less than 30. Concrete encasement is anticipated for electrical conduits. The pipe zone material, which extends to a level 12 inches above the pipe should consist of sand and should have a sand equivalent of no less than 30, and a maximum rock size of 1 inch. All imported materials should be approved by the project geotechnical engineer before being brought onsite.

Trench zone backfill extends from a level 12 inches above the pipe to finished subgrade. Trench zone material should have a maximum size of 2 inches and should contain no organics or other deleterious materials. Most of the near surface soils at the site can be used for trench zone backfill. All fill soils should be approved by the project geotechnical engineer. Soils proposed to be imported should be approved before being brought on site.

All bedding and backfill materials should be mechanically compacted to at least 90 percent relative compaction. Jetting or flooding of backfill should not be permitted.

To prevent water from draining under building slabs through bedding on trench backfill, it is recommended that a concrete "dam" be installed outside the point of entry. The dam should be about 12 inches in thickness and extend at least 1 foot outside the width of the trench.

5.13 Environmental Issues

Evaluation of environmental issues for this project and their impact on site development are outside our scope of our work and are the responsibility of the project environmental consultant.

5.14 Pavement Design

Near surface soils consist of primarily sandy soils. Based on a calculated R-value of 20, the following pavement sections are recommended for Traffic Index (TI) values of 4, 5, 6, and 7:

Table 6: Traffic Index and Section Thickness

Traffic Index (TI)	Section Thickness (inch) AC over AB
4	3" AC/5" AB
5	3.5" AC/6" AB
6	4" AC/8.5" AB
7	4.5 AC/ 11" AB

Traffic Index values of 4 to 5 are recommended for car parking and non-truck areas. Traffic index of 6 or higher may be used for truck areas or for the streets. A concrete pavement consisting of 6 inches of concrete over 6 inches of aggregate base is recommended to be used for trash enclosures and other areas that will be subjected to high wheel loads or abrasive wheel forces, i.e., where there is a tight turning radius. The pavement section for additional TI's can be provided, if requested. The upper 12 inches of subgrade supporting pavements should be compacted to at least 95 percent relative compaction (ASTM D1557).

6.0 POST INVESTIGATION SERVICES

We recommend that final project plans and specifications should be reviewed by GDC to confirm that the full intent of the recommendations presented in this report have been properly applied to the design. During construction, all earthwork should be observed and tested by GDC, including site preparation, excavations, placement of compacted fill and backfill, and installation of foundations, slabs and hardscape.

7.0 LIMITATIONS

The conclusions and recommendations contained in this report are professional opinions that were compiled by searching published data, and are intended for the use of the LAUSD for the proposed development at this site. The recommendations should not be extrapolated to areas not covered by this report, or used for other facilities without the review and approval of GDC. If this report, or portions of this report, is provided to contractors, or included in specifications, it should be understood that they are provided for information only. A design level geotechnical report is necessary prior to development of final plans.

Our investigation and evaluations were performed in accordance with generally accepted local and state standards using that degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report.

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FIGURES

REFERENCE: SOUTH GATE QUADRANGLE USGS 7.5 MINUTE SERIES

DATE: 4/25/2017	DRAWN BY: JMT	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	SITE VICINITY MAP	PROJECT NUMBER: LA-1321
REVIEW BY: -	APPROVED BY: -		ELIZABETH LEARNING CENTER	SCALE: AS SHOWN
PREPARED BY: -			4811 ELIZABETH ST., CUDAHY, CA	FIGURE NUMBER: 1

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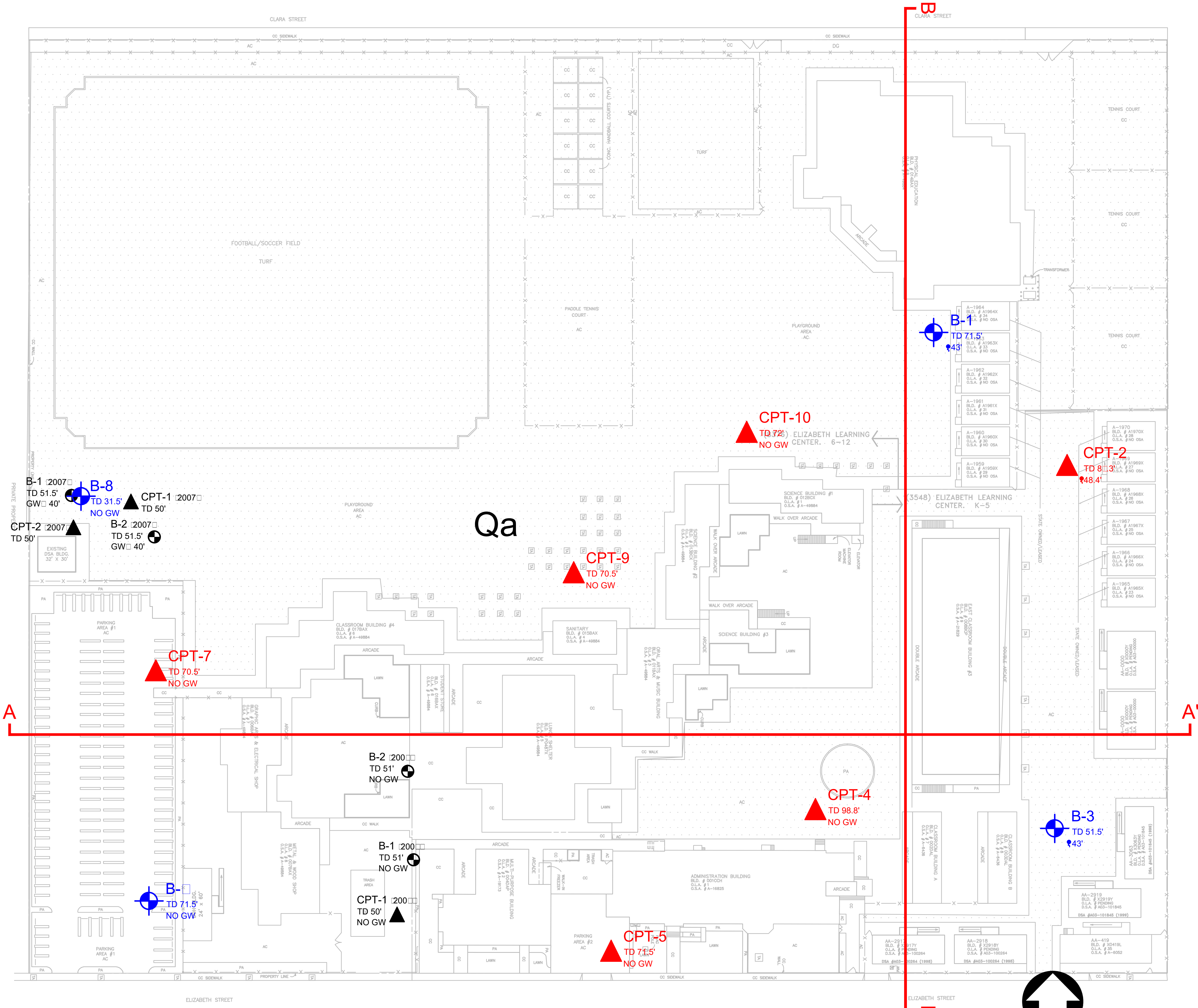

REFERENCE: LAUSD ELIZABETH LEARNING CENTER PLOT PLAN, 0[illegible]-19-01

LEGEND:

Qa	ALLUVIUM	▲	GDC CPT LOCATION AND NUMBER	B-1 (200_)	LEIGHTON BORING LOCATION AND NUMBER (REPORT DATE)
B-2	GDC BORING LOCATION AND NUMBER	A — A'	CROSS SECTION LOCATION	CPT-1 (200_)	LEIGHTON CPT LOCATION AND NUMBER (REPORT DATE)

0 80' 160'

DATE: 4/25/2017	DRAWN BY: JMT	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	EXPLORATION AND SITE GEOLOGY MAP ELIZABETH LEARNING CENTER 4811 ELIZABETH ST., CUDAHY, CA
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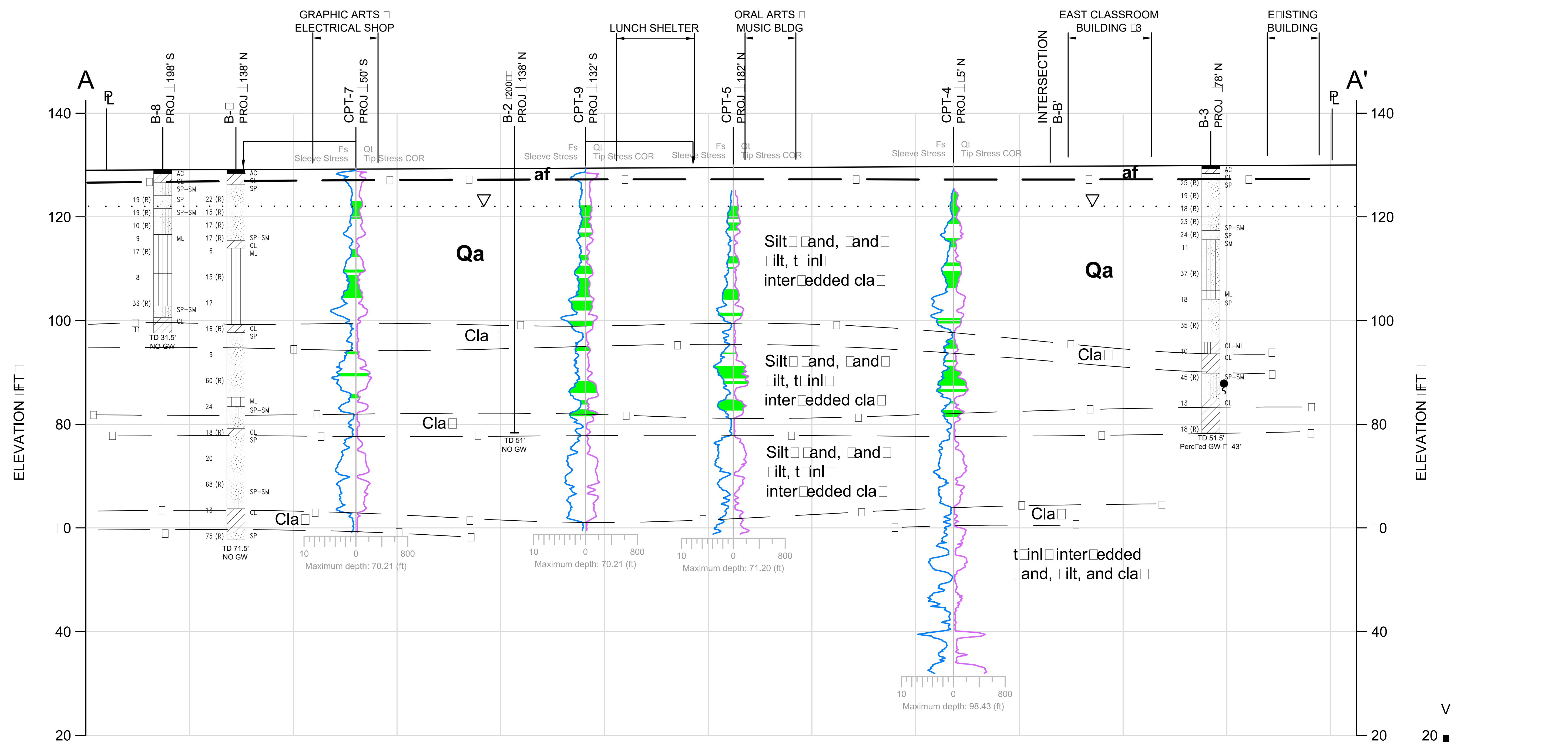


LEGEND:

HISTORICAL HIGH GROUNDWATER LEVEL

ZONES OF LIQUEFIABLE LAYERS

af ARTIFICIAL FILL

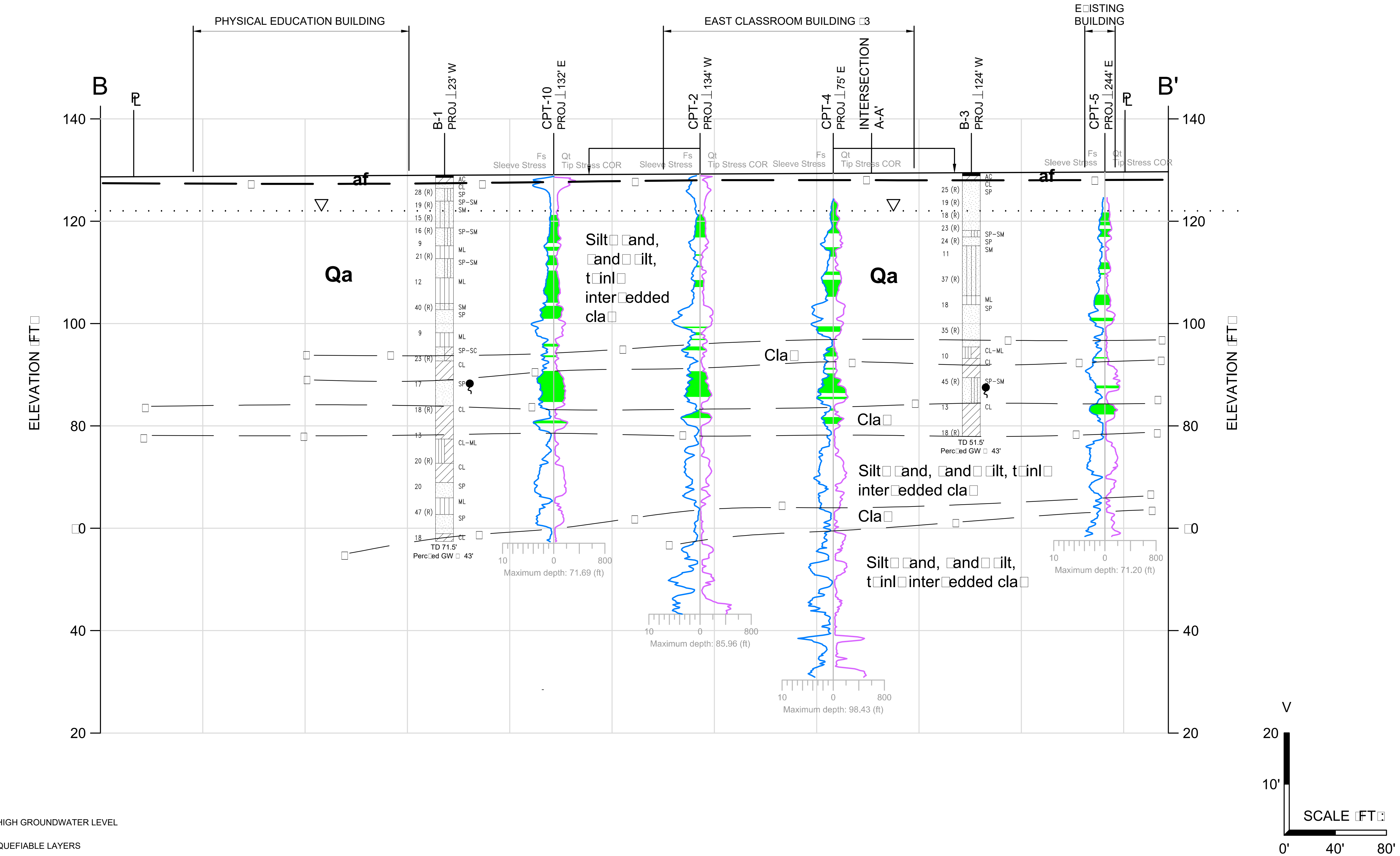
Qa ALLUVIUM THINNLY TO THIC□LY BEDDED SAND, SILT, AND CLAY□GENERALLY LATERALLY CONTINUOUS AND GRADATIONAL

APPRO□IMATE GEOLOGIC UNIT CONTACT, QUERIED WHERE UNCONFINED

APPRO□IMATE STRATIGRAPHIC CONTACT, QUERIED WHERE UNCONFINED

PERCHED GROUNDWATER ENCOUNTERED DURING DRILLING

DATE: 5/10/2017	DRAWN BY: JMT	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	CROSS SECTION A-A'
REVIEW BY: MS	APPROVED BY: ET		ELIZABETH LEARNING CENTER 4811 ELIZABETH ST., CUDAHY, CA
REVISED: -			PROJECT NUMBER: LA-1321 SCALE: AS SHOWN FIGURE NUMBER: 3.1



LEGEND:

- HISTORICAL HIGH GROUNDWATER LEVEL
- ZONES OF LIQUEFIABLE LAYERS
- **af** ARTIFICIAL FILL
- **Qa** ALLUVIUM THINNLY TO THICKLY BEDDED SAND, SILT, AND CLAY, GENERALLY LATERALLY CONTINUOUS AND GRADATIONAL
- APPROXIMATE GEOLOGIC UNIT CONTACT, QUERIED WHERE UNCONFINED
- APPROXIMATE STRATIGRAPHIC CONTACT, QUERIED WHERE UNCONFINED
- PERCHED GROUNDWATER ENCOUNTERED DURING DRILLING

DATE: 5/10/2017	DRAWN BY: JMT	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	CROSS SECTION B-B'
REVIEW BY: MS	APPROVED BY: ET		ELIZABETH LEARNING CENTER 4811 ELIZABETH ST., CUDAHY, CA
REVISED: -			

PROJECT NUMBER: LA-1321

SCALE: AS SHOWN

FIGURE NUMBER: 3.2

MAP UNITS

Late Holocene (Surficial Deposits)

- af **Artificial Fill** - deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills
- **Landslide Deposits** - may include debris flows and older landslides of various earth material and movement types; unconsolidated to moderately well-consolidated
- Qb **Beach Deposits** - unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand
- Qw **Alluvial Wash Deposits** - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand
- Qa **Alluvial Valley Deposits** - unconsolidated clay, silt, sand, and gravel recently deposited parallel to localized stream valleys and/or spread more regionally onto alluvial flats of larger river valleys; sandy sediment generally more dominant than gravelly sediment
- Ql **Lacustrine, Playa, and Estuarine (Paralic) Deposits** - mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites
- Qe **Eolian and Dune Deposits** - unconsolidated, generally well-sorted wind-blown sand; may occur as dune forms or sheet sand

Holocene to Late Pleistocene (Surficial Deposits)

- Qyf **Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
- Qya **Young Alluvial Valley Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
- Qyl **Young Lacustrine, Playa, and Estuarine (Paralic) Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected fine-grained sand, silt, mud and clay from lake, playa, and estuarine deposits of various types
- Qye **Young Eolian and Dune Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected wind-blown sands

Late to Middle Pleistocene (Surficial Deposits)

- Qof **Old Alluvial Fan Deposits** - slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
- Qoa **Old Alluvial Valley Deposits** - slightly to moderately consolidated, moderately dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
- Qol **Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types
- Qoe **Old Eolian and Dune Deposits** - slightly to moderately consolidated, moderately dissected wind-blown sands

Middle to Early Pleistocene (Surficial Deposits)

- Qvof **Very Old Alluvial Fan Deposits** - moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

Quaternary (Bedrock)

- Qss **Coarse-grained formations of Pleistocene age and younger** - primarily sandstone and conglomerate
- Qsh **Fine-grained formations of Pleistocene age and younger** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

Tertiary (Bedrock)

- Tss **Coarse-grained Tertiary age formations** - primarily sandstone and conglomerate
- Tsh **Fine-grained Tertiary age formations** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments
- Tv **Tertiary age formations of volcanic origin**

Mesozoic and Older (Bedrock)

- pKm **Cretaceous and pre-Cretaceous metamorphic formations of sedimentary and volcanic origin**

SYMBOL EXPLANATION

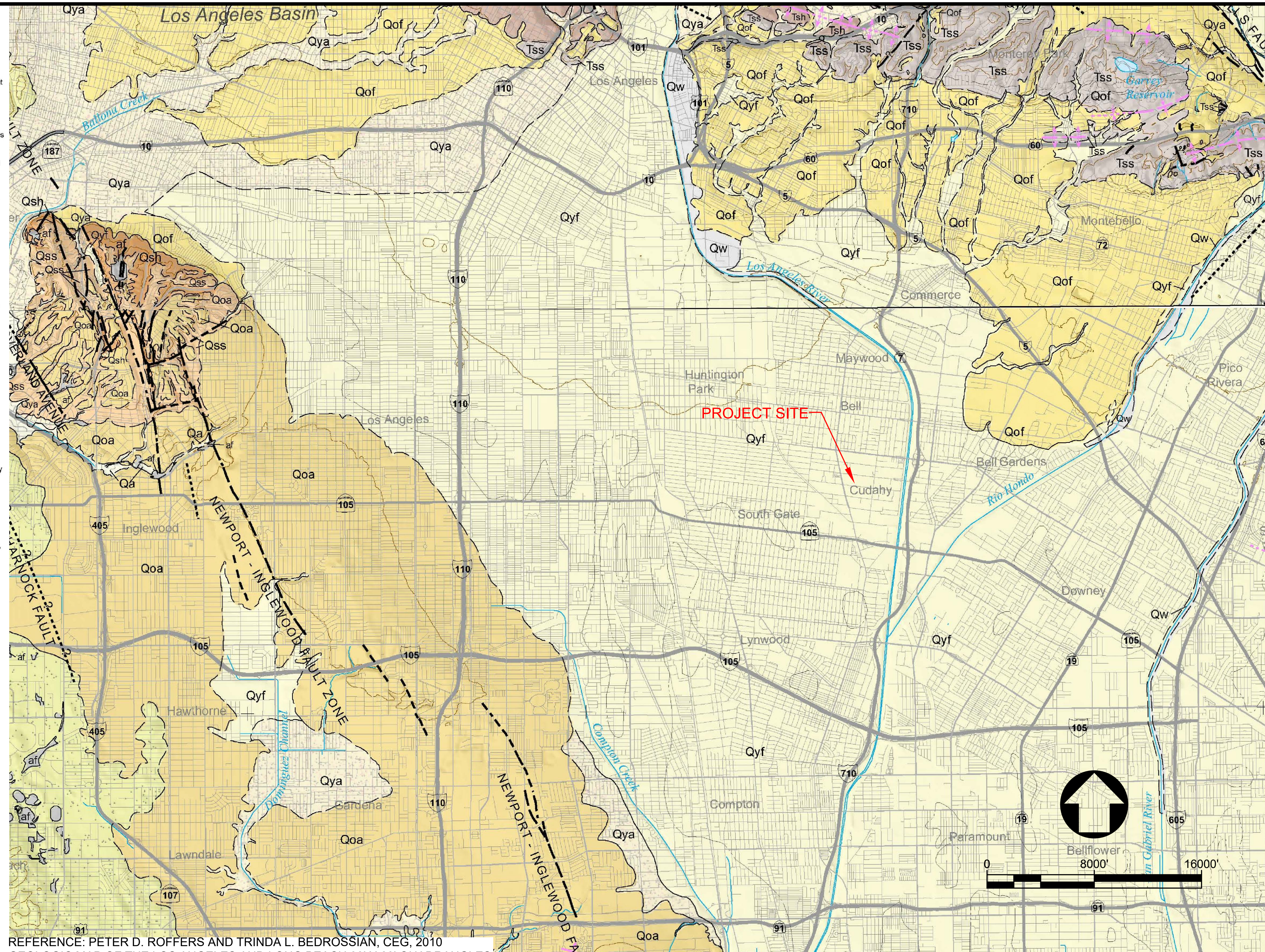
[For geologic line symbols: lines are solid where location is accurate, long-dashed where location is approximate, short-dashed where location is inferred, dotted where location is concealed. Queries added where identity or existence may be questionable.]

Contacts

- Contact
- Reference contact -- Used to delineate geologic units that were mapped as separate units on the original source map, but are consolidated on this map.
- Fault -- Includes strike-slip, normal, reverse, oblique, and unspecified slip

Folds -- Showing direction of plunge where appropriate

- Anticline
- Syncline
- Stream
- Wet area
- Road
- County boundary



REFERENCE: PETER D. ROFFERS AND TRINDA L. BEDROSSIAN, CEG, 2010
GEOLOGIC MAP OF THE LOS ANGELES AND LONG BEACH 30' X 60' QUADRANGLES

DATE: 4/25/2017	DRAWN BY: JMT	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	REGIONAL GEOLOGY MAP	PROJECT NUMBER: LA-1321
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REVISED: -			4811 ELIZABETH ST., CUDAHY, CA	FIGURE NUMBER: 4

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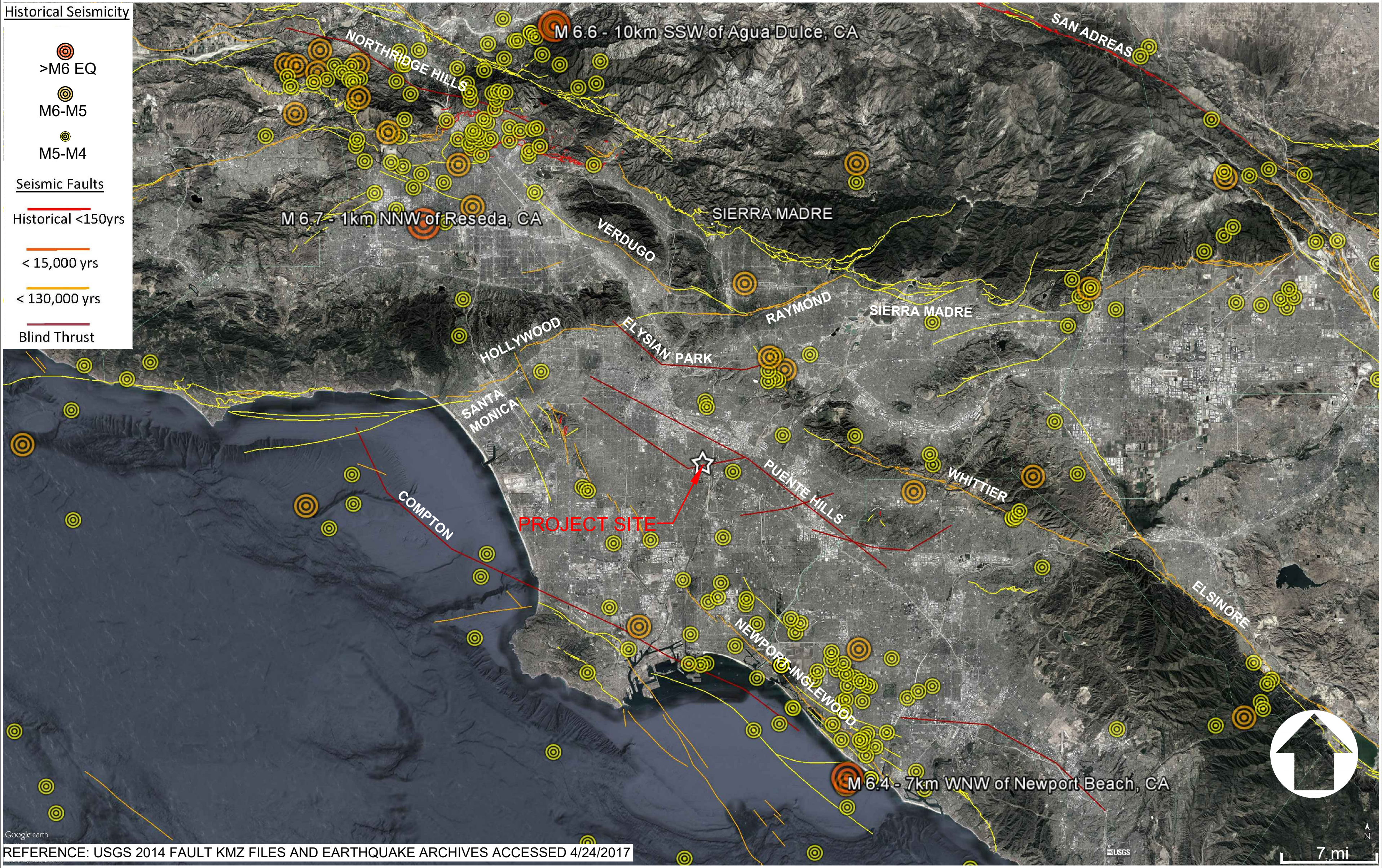

DATE: 4/25/2017	DRAWN BY: JMT	GROUP DELTA	GROUP DELTA CONSULTANTS, INC	REGIONAL SEISMICITY AND FAULT MAP	PROJECT NUMBER: LA-1321
REVIEW BY: -	APPROVED BY: -		370 Amapola Ave. Suite 212	ELIZABETH LEARNING CENTER	SCALE: AS SHOWN
PREPARED BY: -		DELTA	Torrance, CA. 90501	4811 ELIZABETH ST., CUDAHY, CA	FIGURE NUMBER: 5

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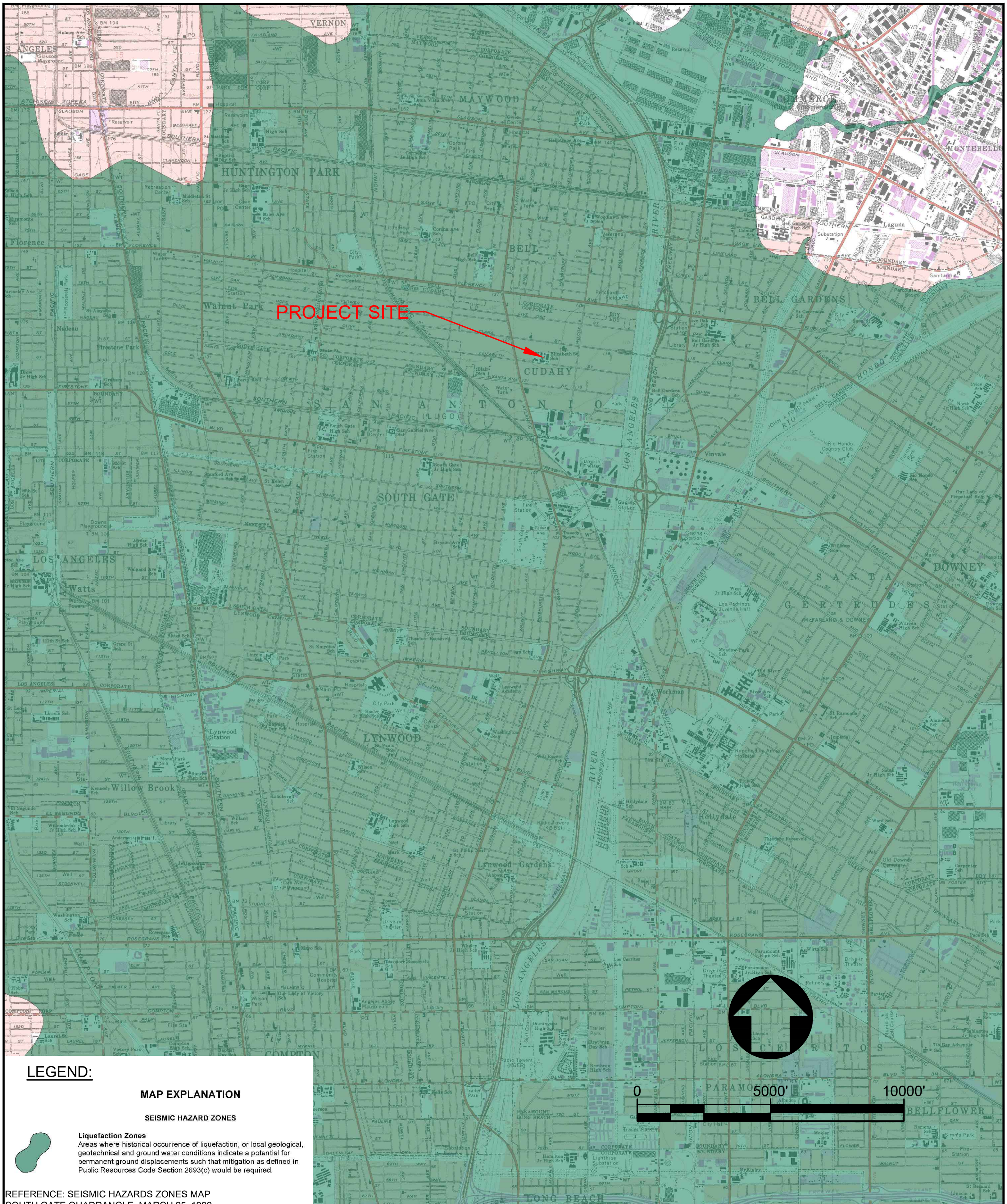

LEGEND:

MAP EXPLANATION

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

REFERENCE: SEISMIC HAZARDS ZONES MAP
SOUTH GATE QUADRANGLE, MARCH 25, 1999

DATE: 4/25/2017	DRAWN BY: JMT	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	EARTHQUAKE ZONES OF REQUIRED INVESTIGATION	PROJECT NUMBER: LA-1321
REVIEW: -	APPROVED BY: -			ELIZABETH LEARNING CENTER	SCALE: AS SHOWN
PREPARED BY: -				4811 ELIZABETH ST., CUDAHY, CA	FIGURE NUMBER: 6

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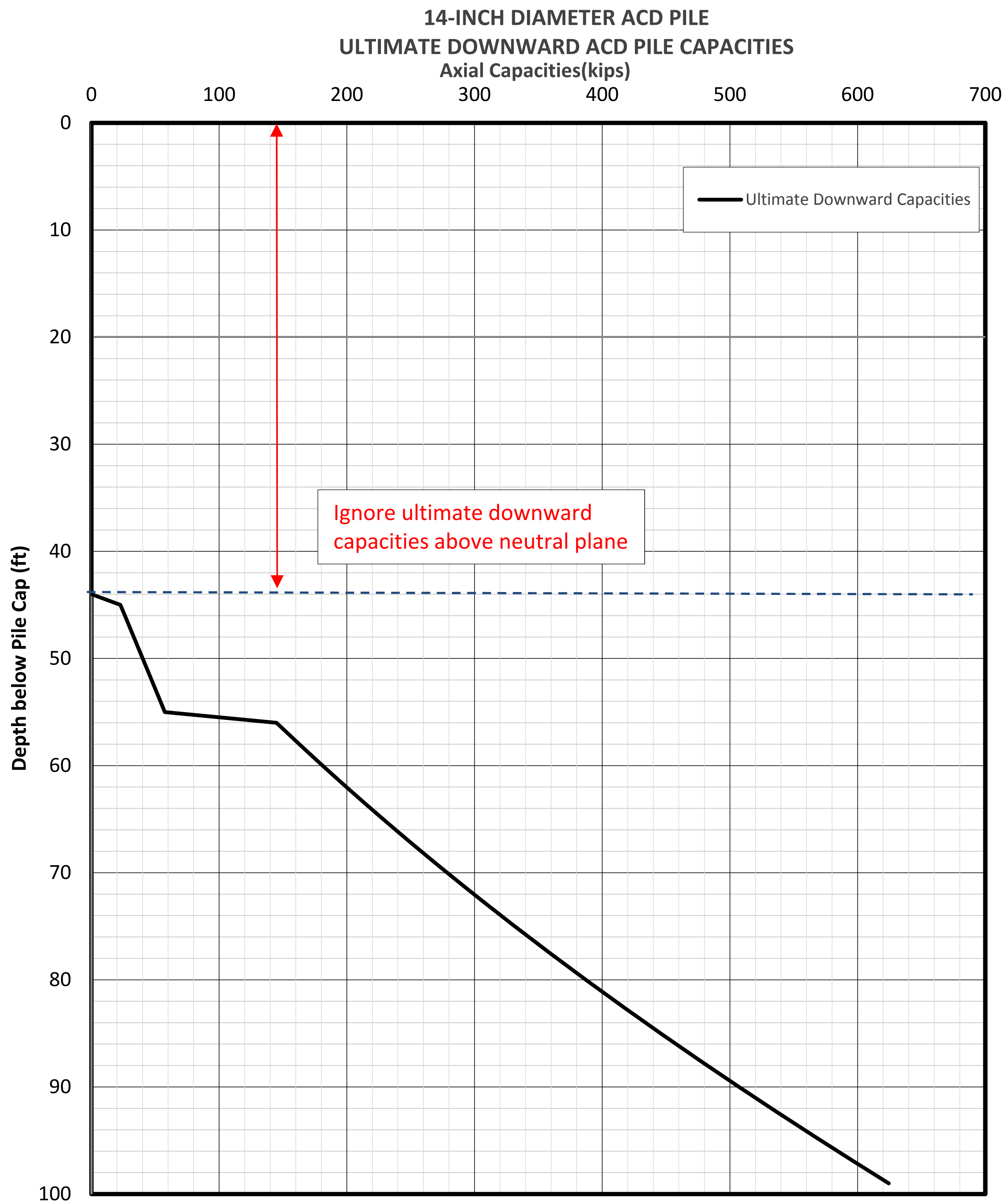
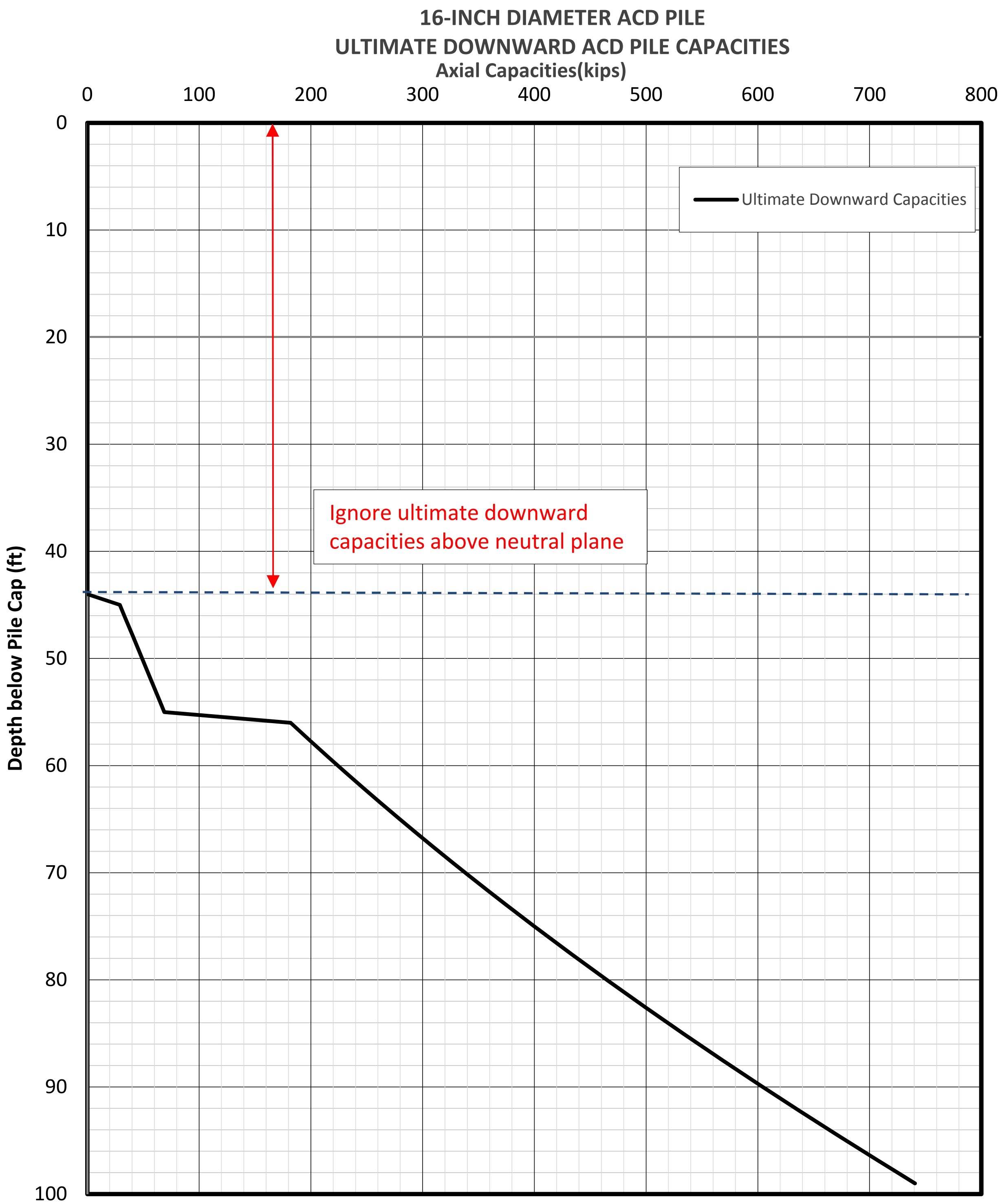


GROUP DELTA	Ultimate Downward Axial Capacity – 14-inch ACD Pile		Project Number: LA-1321 Date: 05/12/2017
	Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501	LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 7.1



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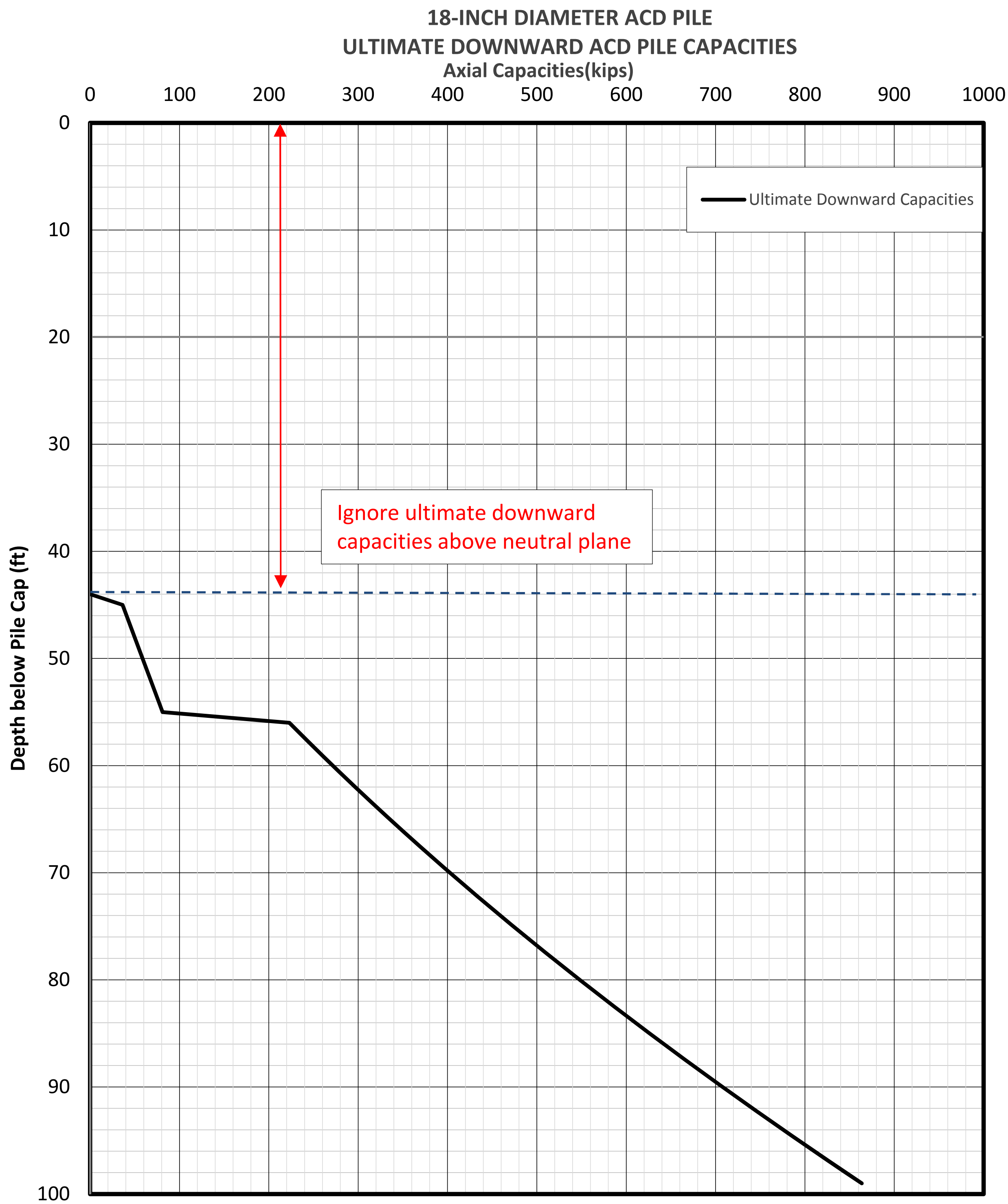
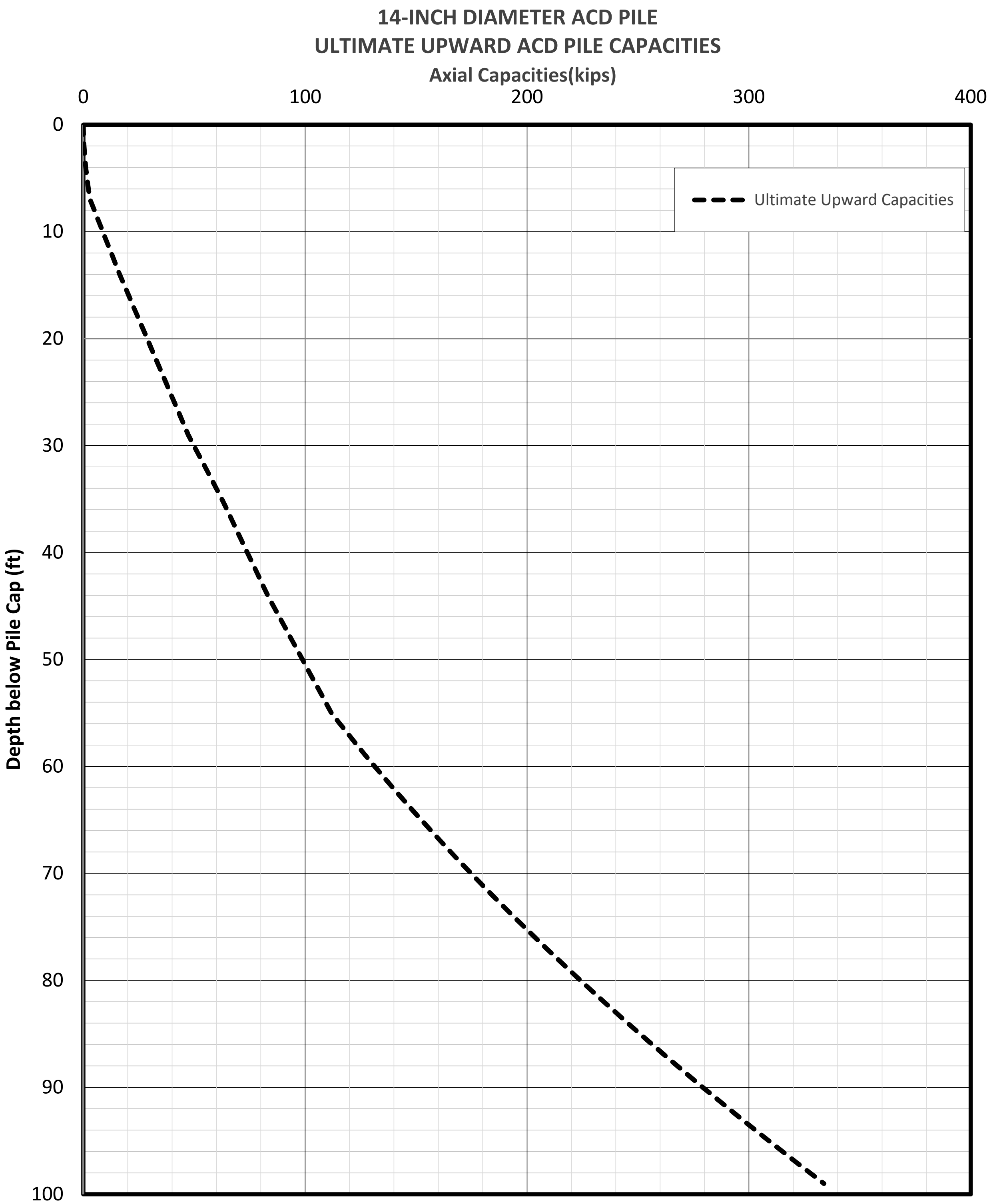
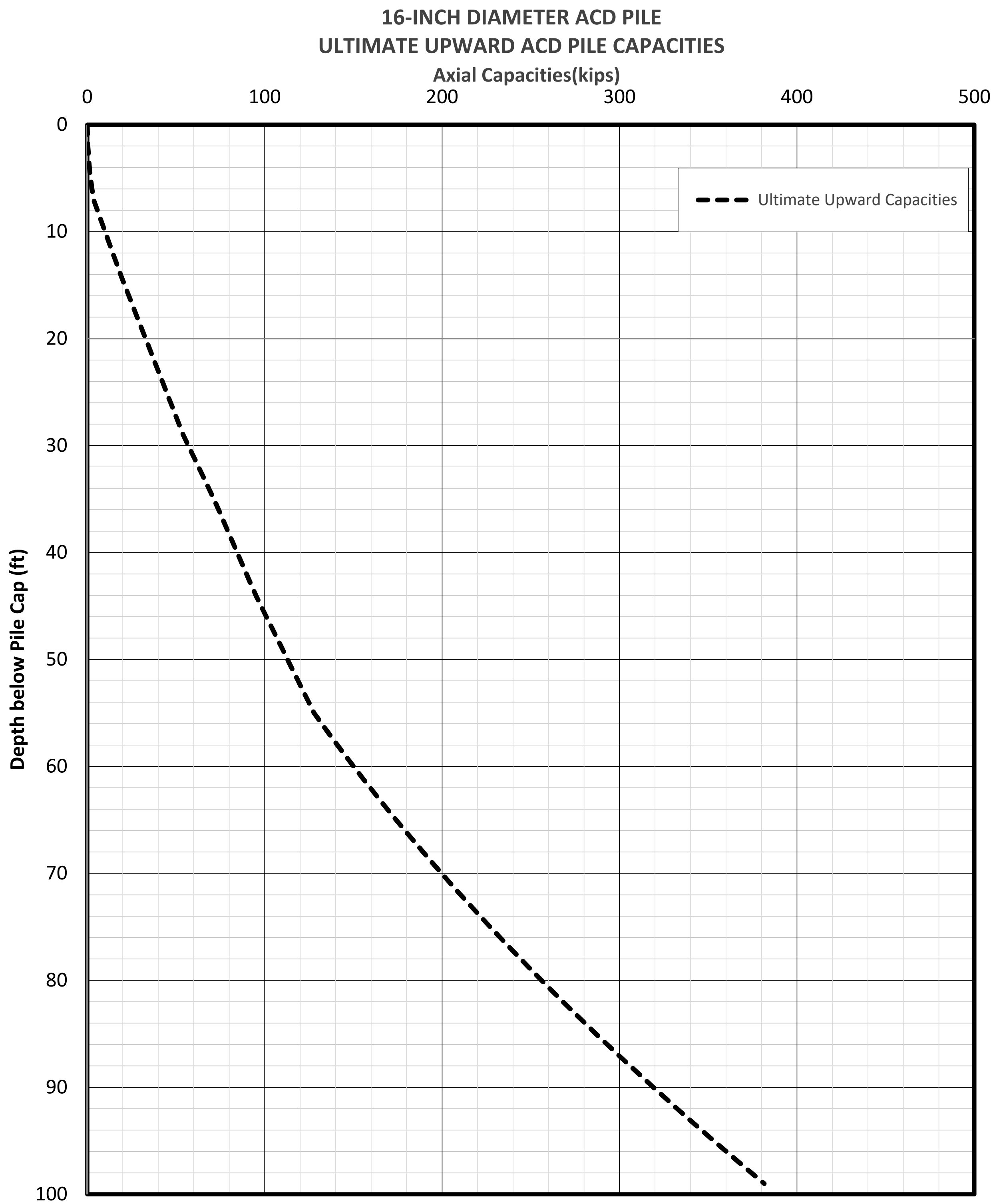
Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501	LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 7.2



GROUP DELTA	Ultimate Downward Axial Capacity – 18-inch ACD Pile	Project Number: LA-1321 Date: 05/12/2017
	Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501 LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 7.3



GROUP DELTA	Ultimate Upward Axial Capacity – 14-inch ACD Pile	Project Number: LA-1321 Date: 05/12/2017
Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501	LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 8.1



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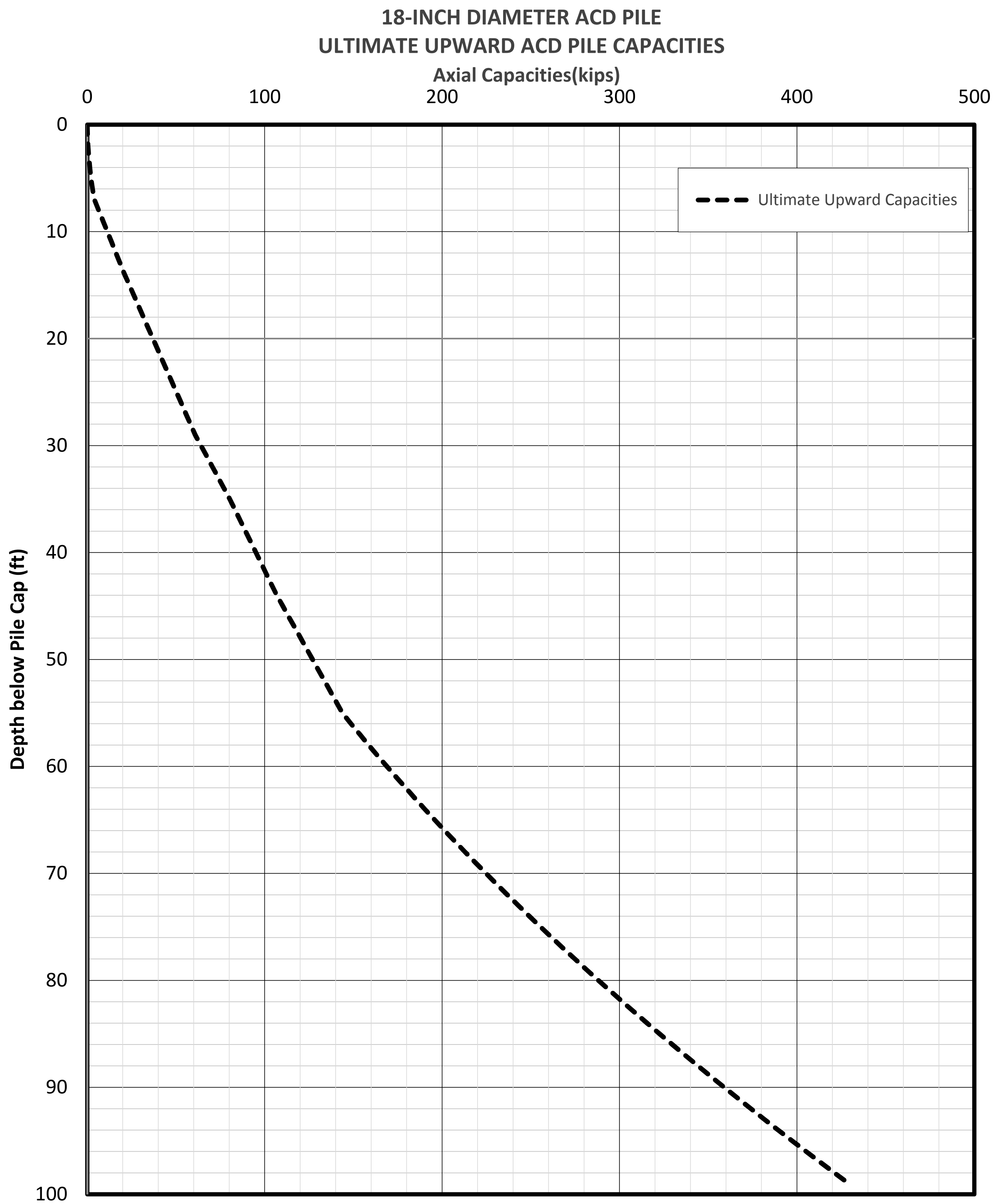
Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501	LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 8.2



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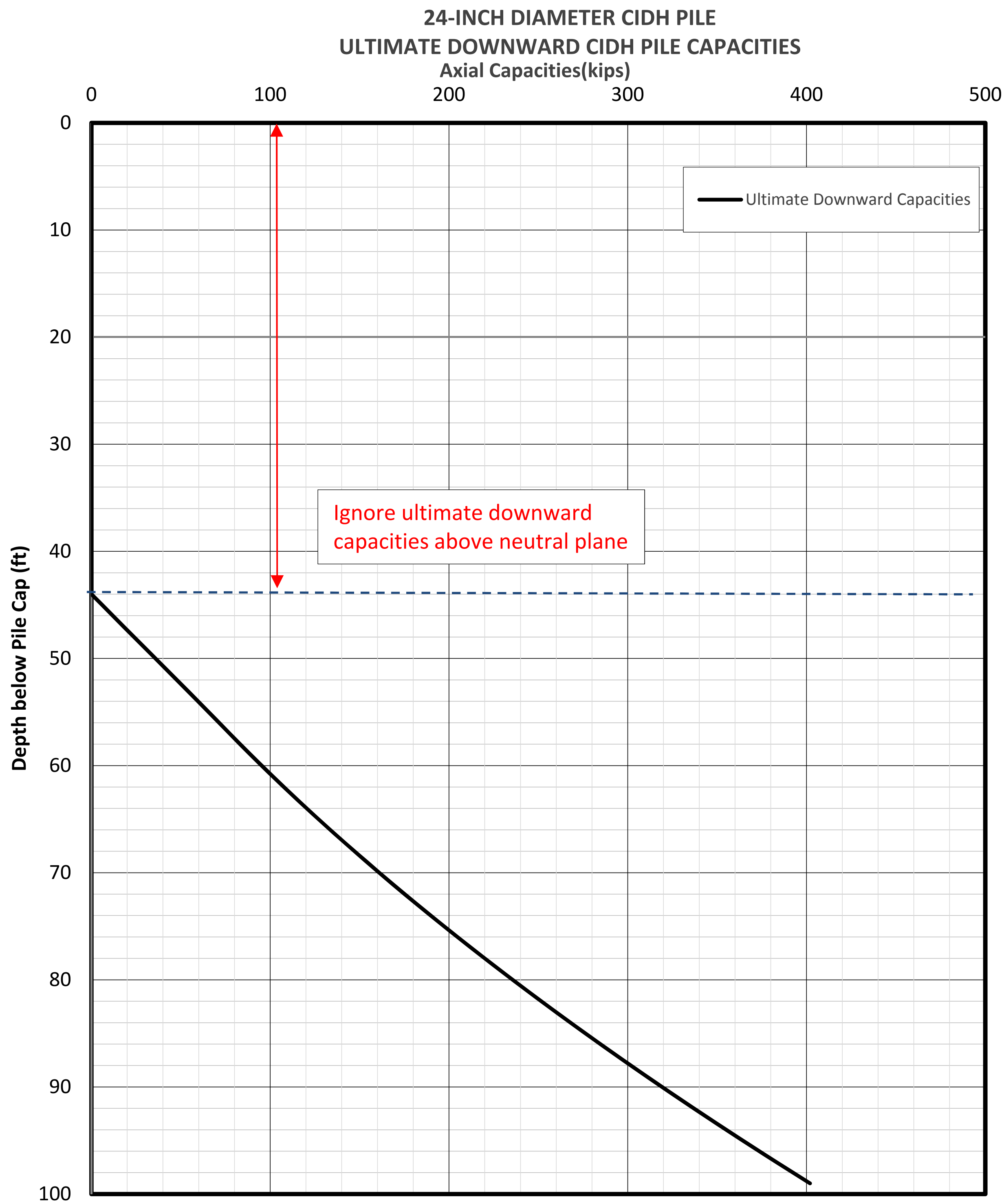
Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501	LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 8.3



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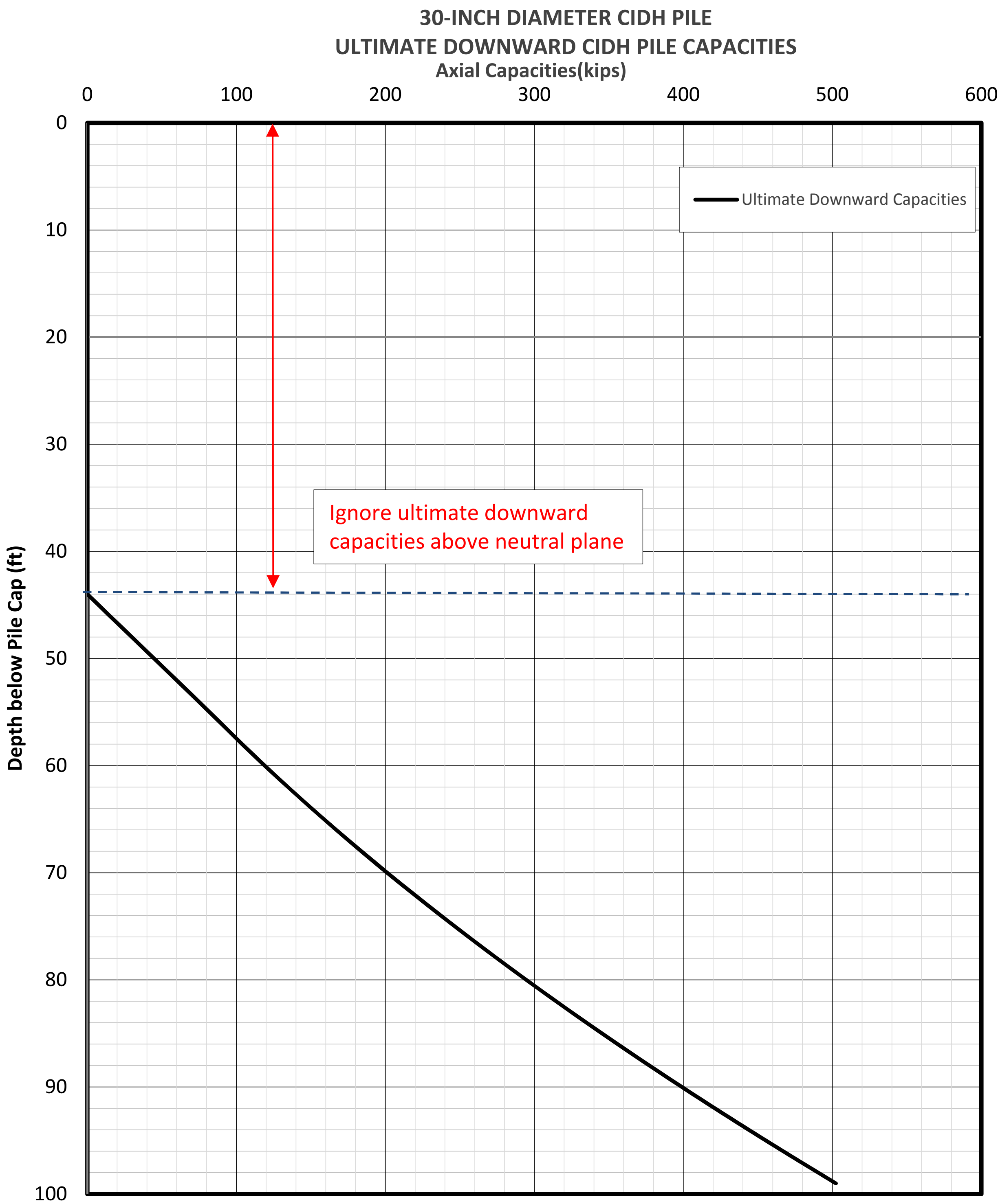

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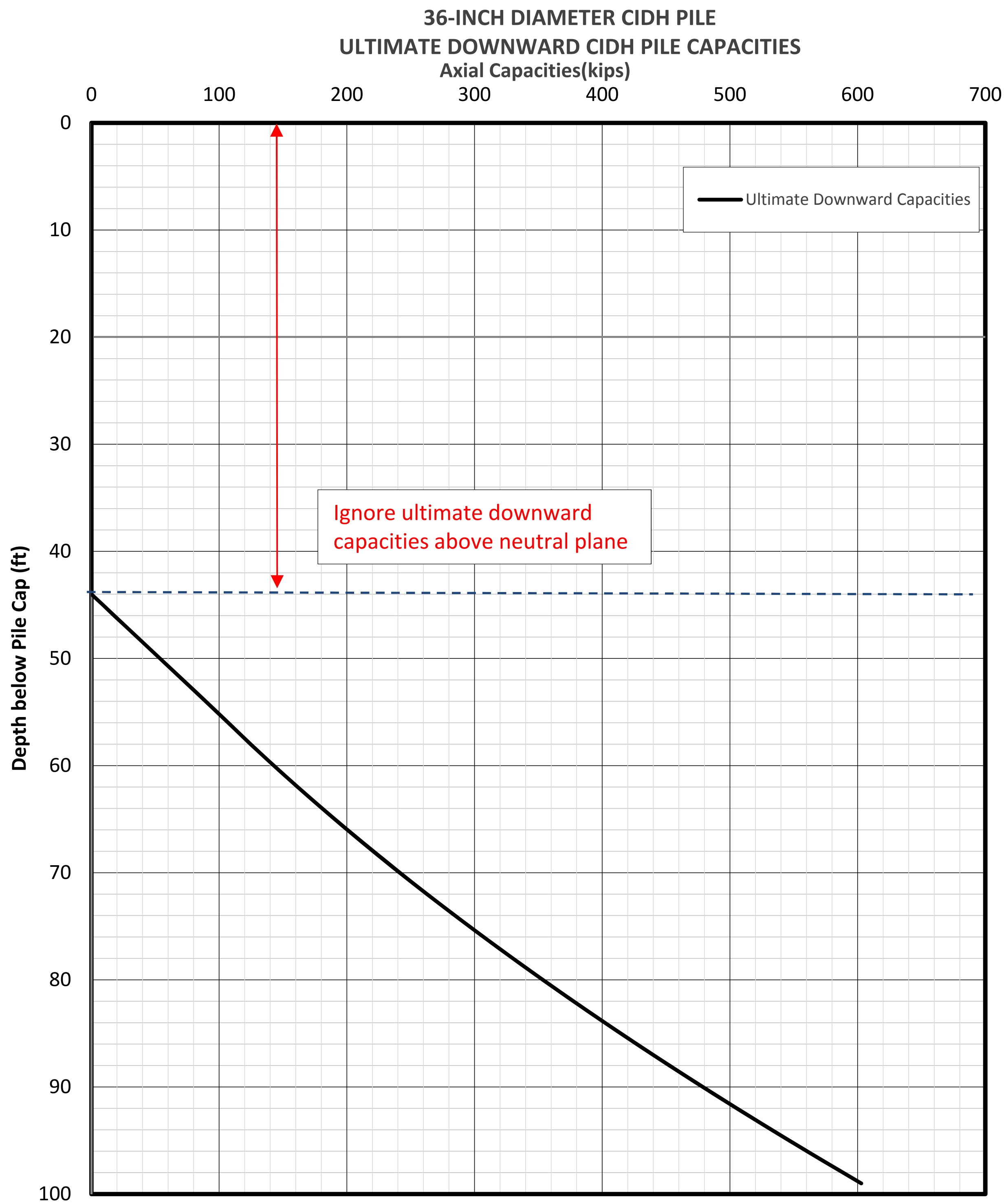
Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501	LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 9.2



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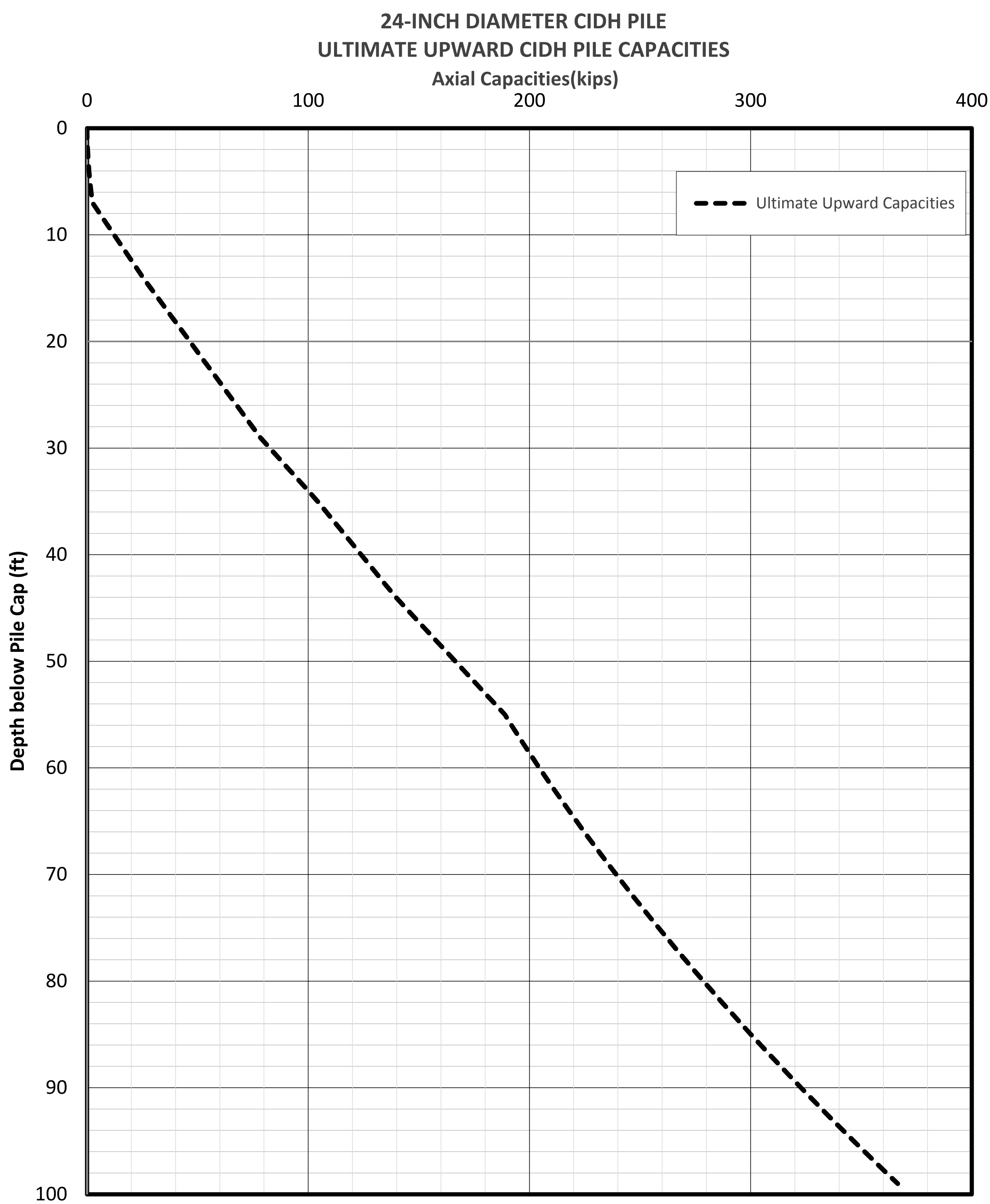
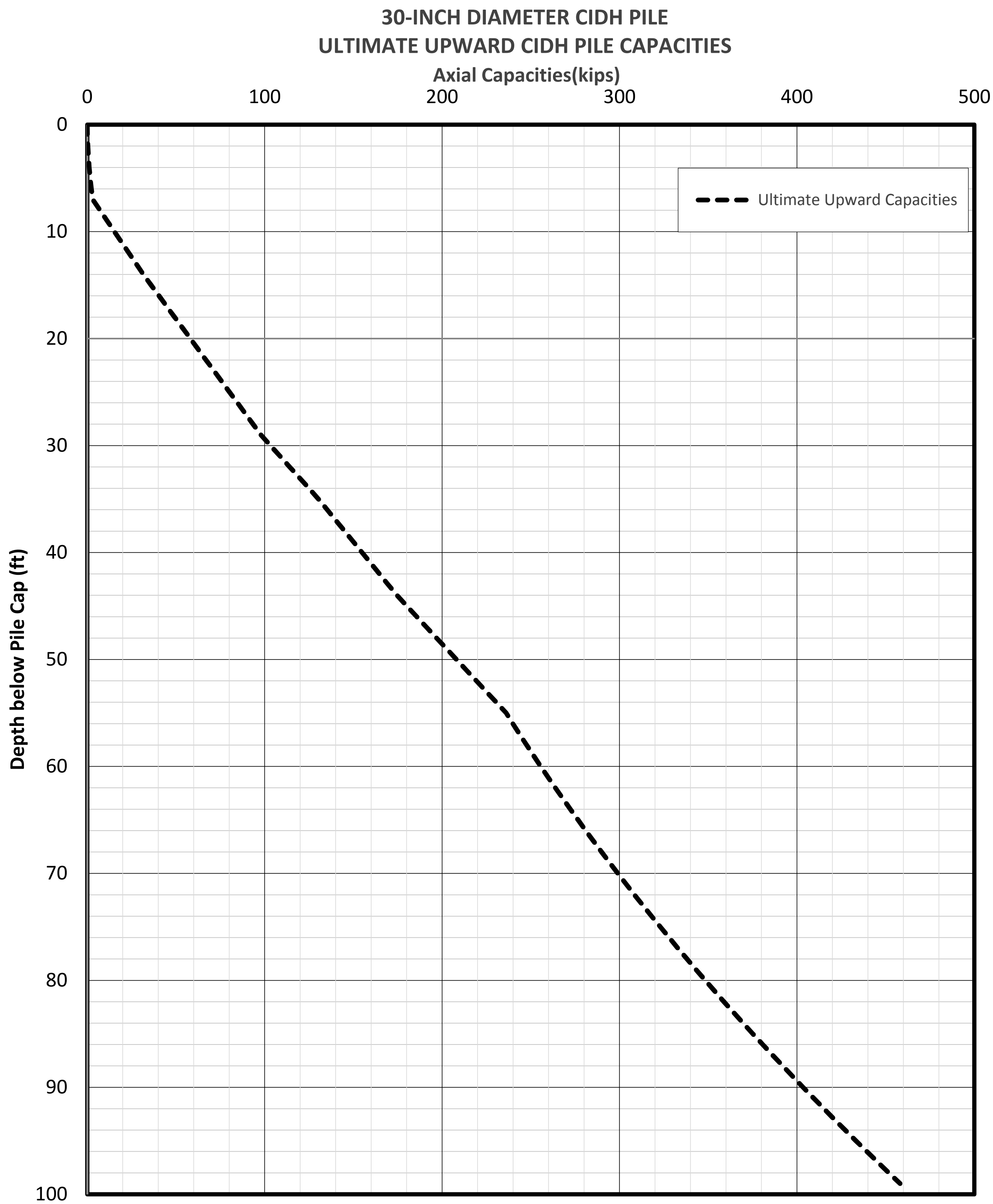


GROUP DELTA	Ultimate Upward Axial Capacity – 24-inch CIDH Pile		Project Number: LA-1321 Date: 05/12/2017
	Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501	LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 10.1



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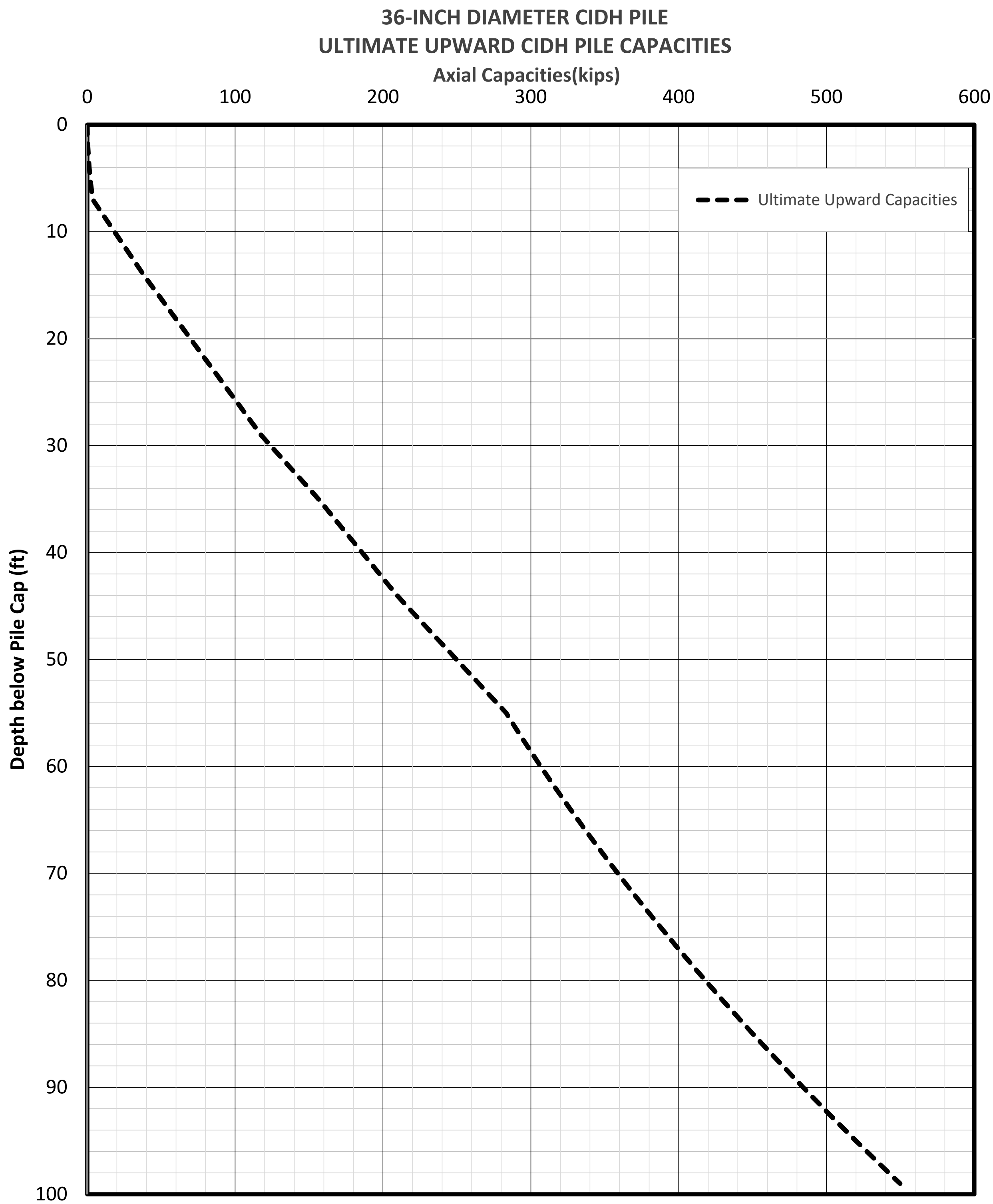
Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501	LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 10.2



GROUP DELTA	Ultimate Upward Axial Capacity – 36-inch CIDH Pile	Project Number: LA-1321 Date: 05/12/2017
	Group Delta Consultants, Inc. 370 Amapola Ave., Suite 212 Torrance, CA 90501 — LAUSD – Elizabeth Learning Center 4811 Elizabeth Street, Cudahy, CA	Figure 10.3

APPENDIX A – FIELD INVESTIGATION

APPENDIX A
FIELD EXPLORATION

A.1 Introduction

A geotechnical subsurface investigation was conducted for the preliminary planning stage at Elizabeth Learning Center in Cudahy, CA on April 14, 2017. Conceptual plans include modernization of existing campus buildings, construction of new buildings up to 3-stories; some new buildings plan to have underground parking. The investigation consisted of (4) hollow stem auger borings and (6) cone penetration tests (CPT). The exploration locations and numbers are shown in Figure 2 of the main report. Other previous investigations were conducted by Leighton Consulting, Inc. for a proposed kitchen expansion in 2006 and proposed multipurpose room in 2007. Applicable borings from the previous investigations are attached to the end of this appendix. The current and previous exploration locations are shown in Figure 2 of the main report. A summary table of the recent GDS investigation and previous investigations by Leighton are provided in Table A-1.

A.2 Soil Borings

Four hollow stem auger borings were advanced from the ground surface to a depth of 31.5 feet to 71.5 feet. The borings were drilled at an approximate ground elevation of (+129 to +130) to elevation (+98.5 to +57.5) feet. Subsurface materials were visually classified and recorded by a GDC field engineer in accordance with the Unified Soil Classification System (USCS).

Drive samples and bulk samples of the encountered materials were obtained from the borings and recorded on the boring logs. Drive samples were obtained with a Modified California Sampler lined with 1-inch high metal sample rings and a Standard Penetration Test (SPT) sampler. The Modified California Sampler has an outside diameter of 3-inches, and the inside diameter of the rings is 2.42-inches. The samples were retained in brass rings and placed in sealed plastic canisters to prevent moisture loss. Standard penetration tests (SPT) were conducted using a standard 2-inch outside diameter, 1.375-inch inside diameter, split-spoon sampler in accordance with ASTM D 1586. SPT samples were placed in sealable plastic bags to protect the natural moisture. The SPT and Modified California samplers were driven into the soil at the bottom of the borehole using a 140-pound hammer free falling 30 inches. The penetration resistance (or "blowcount") in blows per six inches of driving was recorded on the logs. Bulk samples were obtained by a shovel and placed into polyethylene bags.

A key for soil classification and a boring record legend are presented in Figures A-0a and A-0b, respectively. The boring logs are presented in Figures A-1a to A-4b. Applicable previous borings are attached to the end of this appendix.

A.3 Cone Penetration Tests (CPT)

Six cone penetration tests (CPT) were conducted at the site. The CPT's were advanced to depths ranging from about 70.5 feet to 98.8 feet below existing grade before reaching refusal in dense sand. The CPT's were drilled at an approximate ground elevation of (+129 to +130) feet to elevation (+59.5 to +31.2) feet. The CPTs were performed in general accordance with ASTM D3441, using a truck-mounted electric piezocone penetrometer.

CPTs are advanced from the ground surface with a truck-mounted hydraulic ram that pushes a steel rod with a conical tip and a cylindrical friction-sleeve into the ground. The conical tip has a 60-degree apex angle and a projected cross-sectional area of 1.55 square inches. The cylindrical friction sleeve has a surface area of 23.25 square inches. Both the tip and the sleeve have outside diameters of 1.4 inches.

As the rod is advanced, electronic instruments measure and record both the tip resistance and the frictional resistance on the sleeve. The tip and frictional resistance are then analyzed, using available correlations, to estimate soil classification, density, strength, and compressibility of the subsurface materials. Unlike soil borings, in which drive samples are typically taken at discrete intervals, the CPT provides a continuous record of soil properties with depth. Hence, the CPT can define the subsurface soil profile with much higher resolution than a soil boring, often detecting thin layers that are easily missed with conventional drilling and sampling. The CPT logs are presented in Figures A-5 to A-10. Applicable previous CPT logs are attached to the end of this appendix.

A.4 List of Attached Tables and Figures

The following table and figures are attached and complete this appendix:

TABLES

Table A-1
Summary of Recent and Previous GDC Field Explorations

Exploration No.	Date Performed	Ground Surface Elevation (feet, MSL)	Total Depth (ft)	Groundwater Depth (ft)	Exploration Type
B-1	4/14/2017	130	71.5	43 (perched)	Hollow Stem Auger
CPT-2	4/14/2017	129	86.3	Not Encountered	Cone Penetration Test
B-3	4/14/2017	130	51.5	43 (perched)	Hollow Stem Auger
CPT-4	4/14/2017	130	98.8	Not Encountered	Cone Penetration Test
CPT-5	4/14/2017	130	71.5	Not Encountered	Cone Penetration Test
B-6	4/14/2017	129	71.5	Not Encountered	Hollow Stem Auger
CPT-7	4/14/2017	129	70.5	Not Encountered	Hollow Stem Auger
B-8	4/14/2017	130	31.5	Not Encountered	Hollow Stem Auger
CPT-9	4/14/2017	130	70.5	Not Encountered	Cone Penetration Test
CPT-10	4/14/2017	128	72	Not Encountered	Cone Penetration Test
Leighton 2006 and 2007 Field Explorations					
B-1 (2006)	8/11/2006	129	51.5	Not Encountered	Hollow Stem Auger
B-2 (2006)	8/11/2006	130	51.5	Not Encountered	Hollow Stem Auger
CPT-1 (2006)	8/11/2006	130	50	Not Encountered	Cone Penetration Test
B-1 (2007)	2/15/2007	130	51.5	40	Hollow Stem Auger
B-2 (2007)	2/15/2007	130	51.5	40	Hollow Stem Auger
CPT-1 (2007)	2/15/2007	130	50	Not Encountered	Cone Penetration Test

FIGURES

KEY FOR SOIL CLASSIFICATION

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

PRIMARY DIVISIONS			***GROUP SYMBOL***	***SECONDARY DIVISIONS***
COARSE GRAINED SOILS (less than 50% fines passing the No. 200 Sieve)	GRAVEL (% GRAVEL > % SAND)	CLEAN GRAVEL (Less than 5% fines)	GW	Well-graded gravel, gravel with sand, little or no fines
			GP	Poorly-graded gravel, gravel with sand, little or no fines
		"DIRTY" GRAVEL (More than 12% fines)	GM	Silty gravel, silty gravel with sand, silty or non-plastic fines
			GC	Clayey gravel, clayey gravel with sand, clayey or plastic fines
	SAND (% SAND ≥ % GRAVEL)	CLEAN SAND (Less than 5% fines)	SW	Well-graded sand, sand with gravel, little or no fines
			SP	Poorly-graded sand, sand with gravel, little or no fines
		"DIRTY" SAND (More than 12% fines)	SM	Silty sand, silty sand with gravel, silty or non-plastic fines
			SC	Clayey sand, clayey sand with gravel, clayey or plastic fines
FINE GRAINED SOILS (50% or more fines passing the No. 200 Sieve)	SILTS AND CLAYS (Liquid Limit less than 50)		ML	Inorganic silt, sandy silt, gravelly silt, or clayey silt with low plasticity
			CL	Inorganic clay of low to medium plasticity, sandy clay, gravelly clay, silty clay, Lean Clay
			OL	Low to medium plasticity Silt or Clay with significant organic content (vegetative matter)
	SILTS AND CLAYS (Liquid Limit 50 or more)		MH	Inorganic elastic silt, sandy silt, gravelly silt, or clayey silt of medium to high plasticity
			CH	Inorganic clay of high plasticity, Fat Clay
			OH	Medium to high plasticity Silt or Clay with significant organic content (vegetative matter)
HIGHLY ORGANIC SOILS			PT	Peat or other highly organic soils

Note: Dual symbols are used for coarse grained soils with 5 to 12% fines (ex: SP-SM), and for soils with Atterberg Limits falling in the CL-ML band in the Plasticity Chart. Borderline classifications between groups may be indicated by two symbols separated by a slash (ex: CL/CH, SW/GW).

CONSISTENCY CLASSIFICATION

COARSE GRAINED SOILS		***FINE GRAINED SOILS***		
Blowcount SPT[1] (CAL)[2]	Consistency	Blowcount[3] SPT[1] (CAL)[2]	Consistency	Undrained Shear Strenth[3], S_u (ksf)
0-4 (0-6)	Very Loose	<2 (<3)	Very Soft	< 0.25
		2-4 (3-6)	Soft	0.25 -0.50
5-10 (7-15)	Loose	5-8 (7-12)	Firm	0.50 - 1.0
11-30 (16-45)	Med. Dense	9-15 (13-22)	Stiff	1.0 - 2
31-50 (46-75)	Dense	16-30 (23-45)	Very Stiff	2.0 - 4.0
>50 (>75)	Very Dense	>31 (>45)	Hard	>4.0

MOISTURE CLASSIFICATION

DRY - Absence of moisture, dusty, dry to the touch
MOIST- Damp but no visible water
WET- Visible free water, usually soil is below water table

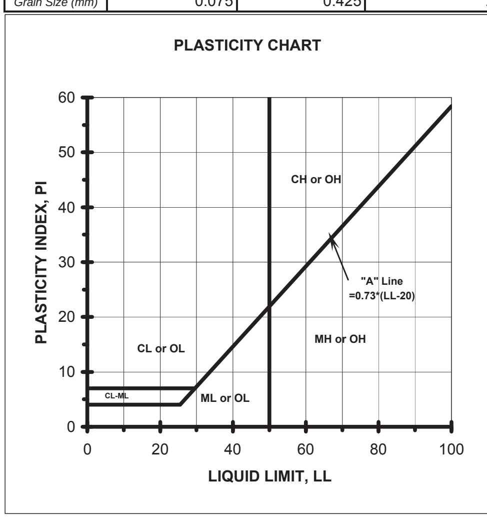
CONSISTENCY NOTES:

1. Number of blows of a 140-lb. hammer falling 30-inches to drive a 2-inch OD (1.375-inch ID) **SPT Sampler** [ASTM D-1585] the final 12-inches of driving
2. Number of blows of a 140-lb. hammer falling 30-inches to drive a 3-inch OD (2.42-inch ID) **California Ring Sampler** the final 12-inches of driving.
3. Undrained shear strength of cohesive soils predicted from field blowcounts is generally unreliable. Where possible, consistency should be based on S_u data from pocket penetrometer, torvane, or laboratory testing.

CLASSIFICATION CRITERIA BASED ON LABORATORY TESTS

Grain Size Classification

	CLAY AND SILT	SAND			GRAVEL		COBBLES		BOULDERS	
		Fine	Medium	Coarse	Fine	Coarse				
US Std Sieve	No. 200	No. 40	No. 10	No. 4	3/4"	3"		12"		
Grain Size (mm)	0.075	0.425	2	4.75	19.1	76.2		304.8		



Classification of earth materials shown on the logs is based on field inspection and should not be construed to imply laboratory analysis unless so stated.

Granular Soil Gradation Parameters

Coefficient of Uniformity: $C_u = D_{60} / D_{10}$

Coefficient of Curvature: $C_C = (D_{30})^2 / (D_{10} \times D_{60})$

D_{10}= 10% of the soil is finer than this diameter

D_{30}= 30% of the soil is finer than this diameter

D_{30}= 60% of the soil is finer than this diameter

Group Symbol	**Gradation or Plasticity Requirement**
SW	C_u>6 and C_c between 1 and 3
GW	C_u>4 and C_c between 1 and 3
GP or SP	Clean gravel or sand not meeting requirement for GW or SW
GM or SM	Plots below "A" Line on Plasticity Chart or PI < 4
GC or SC	Plots above "A" Line on Plasticity Chart and PI > 7

GROUP DELTA

GROUP SYMBOLS AND NAMES

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

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

FIELD AND LABORATORY TESTS

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

SAMPLER GRAPHIC SYMBOLS

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

DRILLING METHOD SYMBOLS

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

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (after drilling, date)

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

DEFINITIONS FOR CHANGE IN MATERIAL

Term	Definition	Symbol
Material Change	Change in material is observed in the sample or core, and the location of change can be accurately measured.	————
Estimated Material Change	Change in material cannot be accurately located because either the change is gradational or because of limitations in the drilling/sampling methods used.	- - - - - - - - -
Soil/Rock Boundary	Material changes from soil characteristics to rock characteristics.	

GROUP DELTA CONSULTANTS, INC. GEOTECHNICAL ENGINEERS AND GEOLOGISTS	FIGURE NUMBER A-0b
PROJECT NAME	PROJECT NUMBER

BORING RECORD LEGEND

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	1 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	71.5	130	▽ NE / *NM* DURING DRILLING; ▼ / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	130														2" Asphalt, 3" Base
		Bulk	Bulk-1												**FILL** SANDY CLAY (CL); dark brown; mostly fines; some medium SAND; low to medium plasticity.
															NATIVE Poorly-graded SAND (SP); light brown; mostly fine to medium SAND; nonplastic.
		ModCAL	R-2	9, 14, 14	28				6.6	98					Poorly-graded SAND with SILT (SP-SM); medium dense; brown; moist; mostly fine SAND; little medium SAND; trace coarse SAND; nonplastic; brick fragments; trace mica.
5	125	ModCAL	R-3	6, 9, 10	19				8.5	106					SILTY SAND (SM); medium dense; light brown; mostly fine to medium SAND; nonplastic.
		ModCAL	R-4	4, 6, 9	15										- Loose.
10	120	ModCAL	R-5	4, 6, 10	16				8.9	101		PA			- Increase in grain size. 83% SAND; 17% fines.
															Poorly-graded SAND with SILT (SP-SM); medium dense; light brown; moist; mostly medium SAND; few fines; nonplastic.
		SPT	S-6-2 / S-6-1	4, 4, 5	9				18.0						- Loose.
															SILT (ML); stiff; brown; moist; mostly fines; few fine SAND; nonplastic.
15	115	ModCAL	R-7	6, 10, 11	21										-Medium dense.
															Poorly-graded SAND with SILT (SP-SM); medium dense; brown; moist; mostly fine SAND; few fines; nonplastic; trace mica.
20	110	SPT	S-8	4, 6, 6	12				14.5						SANDY SILT (ML); medium dense; dark brown; moist; mostly fines; some fine to medium SAND; nonplastic to low plasticity.

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	2 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/ELEV. GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	71.5	130	▽ NE / *NM* DURING DRILLING; ▼ / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			R-9	9 15 25	40										SILTY SAND (SM); medium dense; brown; moist; mostly fine to medium SAND; nonplastic.
															Poorly-graded SAND (SP); medium dense; moist; yellow brown; mostly medium SAND; nonplastic. - Light brown; oxidized; mostly fine to medium SAND.
30	100		S-10-2 S-10-1	5 4 5	9							PI			SILT (ML); stiff to very stiff; moist; gray; medium plasticity. PP=1.5 to 2.5 tsf
35	95		R-11	10 11 12	23										Poorly-graded SAND with CLAY (SP-SC); medium dense; gray; moist; mostly medium SAND; few fines; nonplastic.
															Lean CLAY with SAND (CL); stiff; gray; moist; little fine SAND; low to medium plasticity. PP=1.5 tsf
40	90		S-12	6 8 9	17										Poorly-graded SAND (SP); medium dense; gray; wet; mostly medium SAND; nonplastic.
45	85		R-13	4 6 12	18				32.8	89					Lean CLAY (CL); stiff; moist; dark gray; medium to high plasticity. PP=1.5 to 1.75 tsf

GDC_LOG_BORING_2013 LA1321 B-1,B-3,B-6,B-8.GPJ GDC2013.GDT 5/12/17

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	B-1

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	3 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/ELEV. GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	71.5	130	▽ NE / *NM* DURING DRILLING; ▼ / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			S-14	3 5 8	13										- trace fine SAND; trace mica.
55	75		R-15	7 10 10	20										SILTY CLAY (CL-ML); very stiff; dary gray; moist; low plasticity. PP=2.75 tsf
															Lean CLAY (CL); stiff; dark gray; moist; medium to high plasticity. PP=1.5 tsf
60	70		S-16	7 9 11	20										Poorly-graded SAND (SP); medium dense; dark gray; moist; mostly medium SAND; nonplastic.
65	65		R-17	14 18 29	47										SILT with SAND (ML); dense to very stiff; dark gray; moist; mostly fines; few fine SAND; trace mica. PP=2.0 to 2.25 tsf
															Poorly-graded SAND (SP); dense; dark gray; moist; mostly medium SAND; nonplastic.
70	60		S-18	6 8 10	18										Lean CLAY (CL); stiff; dark gray; moist; medium to high plasticity.
															Boring terminated at 71.5 feet. Perched water encountered at 42'-10". Groundwater not encountered. Backfill with soil cuttings and tamped, asphalt patched.

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370 Amapola Ave, Suite 212
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FIGURE

A-1 c

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	B-3

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	1 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	51.5	130	NE / *NM* DURING DRILLING; / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															2" Asphalt, 5" Base
			Bulk-1						5.2						**FILL** SANDY CLAY (CL); dark brown; moist; mostly fines; some medium SAND; low to medium Plasticity.
			R-2	10 12 13	25										**NATIVE** Poorly-graded SAND (SP); medium dense; light brown; moist; mostly medium SAND; nonplastic.
5	125		R-3	5 9 10	19										- Increase in grain size.
			R-4	9 8 10	18				8.2	93					
10	120		R-5	5 10 13	23										Poorly-graded SAND with SILT (SP-SM); dense; orange, light brown; moist; mostly medium SAND; few fines; nonplastic; trace mica.
			R-6	5 10 14	24										Poorly-graded SAND (SP); medium dense; light brown; moist; mostly medium SAND; nonplastic.
15	115		S-7	5 5 6	11							PA			SILTY SAND (SM); medium dense; orange brown; moist; mostly medium SAND; some fine SAND; some fines; nonplastic. 42% SAND; 58% fines.
20	110		R-8	9 16 21	37				13.3	118					
															SILT with SAND (ML); medium dense to very stiff; moist; brown; mostly fines; few fine SAND; nonplastic to low

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	B-3

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	2 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	51.5	130	∇ NE / *NM* DURING DRILLING; ▼ / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															plasticity.
			S-9-2 S-9-1	5 8 10	18				20.2 8.9						Poorly-graded SAND (SP); medium dense; light brown; moist; mostly fine to medium SAND; nonplastic.
30	100		R-10	9 15 20	35										- Light brown; oxidized; mostly fine SAND.
															SILTY CLAY (CL-ML); very stiff; light brown; trace to few fine SAND; low plasticity. PP=1.75 tsf
35	95		S-11-2 S-11-1	3 5 5	10										Lean CLAY (CL); very stiff; dark gray; medium plasticity. PP=1.5 to 1.75 tsf
40	90		R-12	10 19 26	45				18.6	104					Poorly-graded SAND with SILT (SP-SM); medium dense; light brown, gray; moist; mostly medium SAND; few fines; nonplastic.
45	85		S-13	2 5 8	13										Lean CLAY (CL); stiff to very stiff; dark gray; trace SILT; low to medium plasticity. PP=1.0 to 2.5 tsf

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FIGURE A-2 b

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	**B-3**

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	3 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	51.5	130	▽ NE / *NM* DURING DRILLING ▼ / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			R-14	4 6 12	18										- High plasticity. PP=1.5 tsf
															Boring terminated at 51.5 feet. Perched water encountered at 42'-10". Groundwater not encountered. Backfill with soil cuttings and tamped, asphalt patched.
55	75														
60	70														
65	65														
70	60														

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

FIGURE

A-2 c

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	**B-6**

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	1 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	71.5	129	⊻ NE / *NM* DURING DRILLING; ▼ / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															3" Asphalt, 7" Base
			Bulk-1						7.4			EI CR			**FILL** SANDY CLAY (CL); dark brown; moist; mostly fines; some medium SAND; low to medium plasticity.
	125		Bulk-2												**NATIVE** Poorly-graded SAND (SP); light brown; moist; mostly fine SAND; nonplastic.
5			R-2	7 9 13	22										- Medium dense.
	120		R-3	5 6 9	15				6.0	101					- Increase in grain size.
10			R-4	5 7 10	17										
	115		R-5	5 6 11	17				19.9		0:0	PI			Poorly-graded SAND with SILT (SP-SM); medium dense; brown; moist; mostly fine to medium SAND; few fines; trace clay; nonplastic.
															Lean CLAY (CL); stiff; brown; medium plastic
15			S-6	2 2 4	6										SILT (ML); stiff; brown; trace to few fine SAND; low plasticity.
	110														
20			R-7	5 6 9	15				23.4	103					- Olive brown.
	105														

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	**B-6**

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	2 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	71.5	129	⊻ NE / *NM* DURING DRILLING; ▼ / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															- Gray; less SAND, finer grain size.
		SPT	S-8	4, 5, 7	12										
	100														
30															SANDY CLAY (CL); stiff; gray; mostly fines; some fine SAND; medium plasticity.
		ModCAL	R-9	5, 7, 9	16										
															Poorly-graded SAND (SP); gray; moist; mostly fine SAND; nonplastic.
	95														
35															- Loose; mostly medium SAND.
		SPT	S-10	6, 3, 6	9										- Thin interbedd of SANDY SILT (ML); gray; some fine SAND; nonplastic to low plasticity. PP=1.75 to 2.0 tsf
	90														
40															- Gray, light brown; mostly medium SAND.
		ModCAL	R-11	17, 26, 34	60										- Mostly fine to medium SAND.
	85														SANDY SILT (ML); medium dense to very stiff; gray; wet; some fine SAND; nonplastic to low plasticity. PP=2.25 tsf
45															
		SPT	S-12-2, S-12-1	5, 10, 14	24										Poorly-graded SAND with SILT (SP-SM); medium dense; gray; wet; mostly fine SAND; few medium SAND; few fines; nonplastic.
	80														

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	B-6

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	3 of 3

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/ELEV. GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	71.5	129	∇ NE / NM DURING DRILLING; ▼ / NM AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		ModCAL	R-13	5 7 11	18				20.5	108					SANDY CLAY (CL); stiff; gray; mostly fines; some fine SAND; low to medium plasticity.
	75														
55		SPT	S-14	6 9 11	20										Poorly-graded SAND (SP); medium dense; gray; wet; mostly medium SAND; trace coarse SAND; nonplastic.
	70														
60		ModCAL	R-15	13 30 38	68										- Mostly medium SAND.
															Poorly-graded SAND with SILT (SP-SM); medium dense; gray; mostly medium SAND; few fines; nonplastic.
	65														
65		SPT	S-16-2 S-16-1	6 5 8	13										Lean CLAY (CL); stiff; gray; medium to high plasticity; trace mica. PP=1.5 tsf
	60														
70		ModCAL	R-17	15 30 45	75				19.6	109					Poorly-graded SAND (SP); very dense; gray; wet; mostly medium SAND; trace mica; nonplastic.
															Boring terminated at 71.5 feet. Groundwater not encountered. Backfill with soil cuttings and tamped, asphalt patched.
	55														

GROUP DELTA CONSULTANTS
370 Amapola Ave, Suite 212
Torrance, CA 90501

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

FIGURE A-3 c

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	**B-8**

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	1 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	31.5	130	⊻ NE / *NM* DURING DRILLING; ▼ / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	130														3" Asphalt, 7" Base
															FILL SANDY CLAY (CL); dark brown; moist; some medium SAND; low to medium plasticity.
															Poorly-graded SAND with SILT (SP-SM); yellow, light brown; moist; mostly fine SAND; few fines; nonplastic.
		Bulk	Bulk-1						7.7						
5	125														Poorly-graded SAND (SP); medium dense; light brown; moist; mostly medium SAND; nonplastic.
		ModCAL	R-2	7 / 8 / 11	19										
															Poorly-graded SAND with SILT (SP-SM) with thin interbeds of Lean CLAY (CL). Poorly-graded SAND with SILT (SP-SM); medium dense; brown; moist; mostly medium SAND; few fines; nonplastic. Lean CLAY (CL); brown; low plasticity.
		ModCAL	R-3	5 / 8 / 11	19										
10	120														
		ModCAL	R-4	5 / 5 / 5	10				6.1	99					
															SANDY SILT (ML); loose; brown; moist; some fine SAND; trace to few medium SAND; low plasticity; trace mica.
		SPT	S-5	4 / 4 / 5	9										
15	115														
		ModCAL	R-6	5 / 6 / 11	17				15.8	111					
20	110														SANDY SILT (ML); stiff to very stiff; brown; moist; mostly fines; few fine SAND; low plasticity; trace mica. 52% fines; 48% SAND.
		SPT	S-7	3 / 4 / 4	8							PA			

GROUP DELTA CONSULTANTS
370 Amapola Ave, Suite 212
Torrance, CA 90501

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
LAUSD Elizabeth Learning Center	LA-1321	**B-8**

SITE LOCATION	START	FINISH	SHEET NO.
4811 Elizabeth Street, Cudahy, CA	4/14/2017	4/14/2017	2 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 85	Hollow Stem Auger	KM	ET

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.		8	31.5	130	⊻ NE / *NM* DURING DRILLING; ▼ / *NM* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															- Very Stiff; oxidized; trace fine SAND.
			R-8	7 9 24	33										
															Poorly-graded SAND with SILT (SP-SM); medium dense; brown; moist; mostly fine SAND; few fines; nonplastic; trace mica.
															SANDY CLAY (CL); stiff to very stiff; brown; moist; mostly fines; some fine SAND; low to medium plasticity; trace mica.
30	100														
			S-9	2 5 6	11				23.2						
															Boring terminated at 31.5 feet. Groundwater not encountered. Backfill with soil cuttings and tamped, asphalt patched.
35	95														
40	90														
45	85														

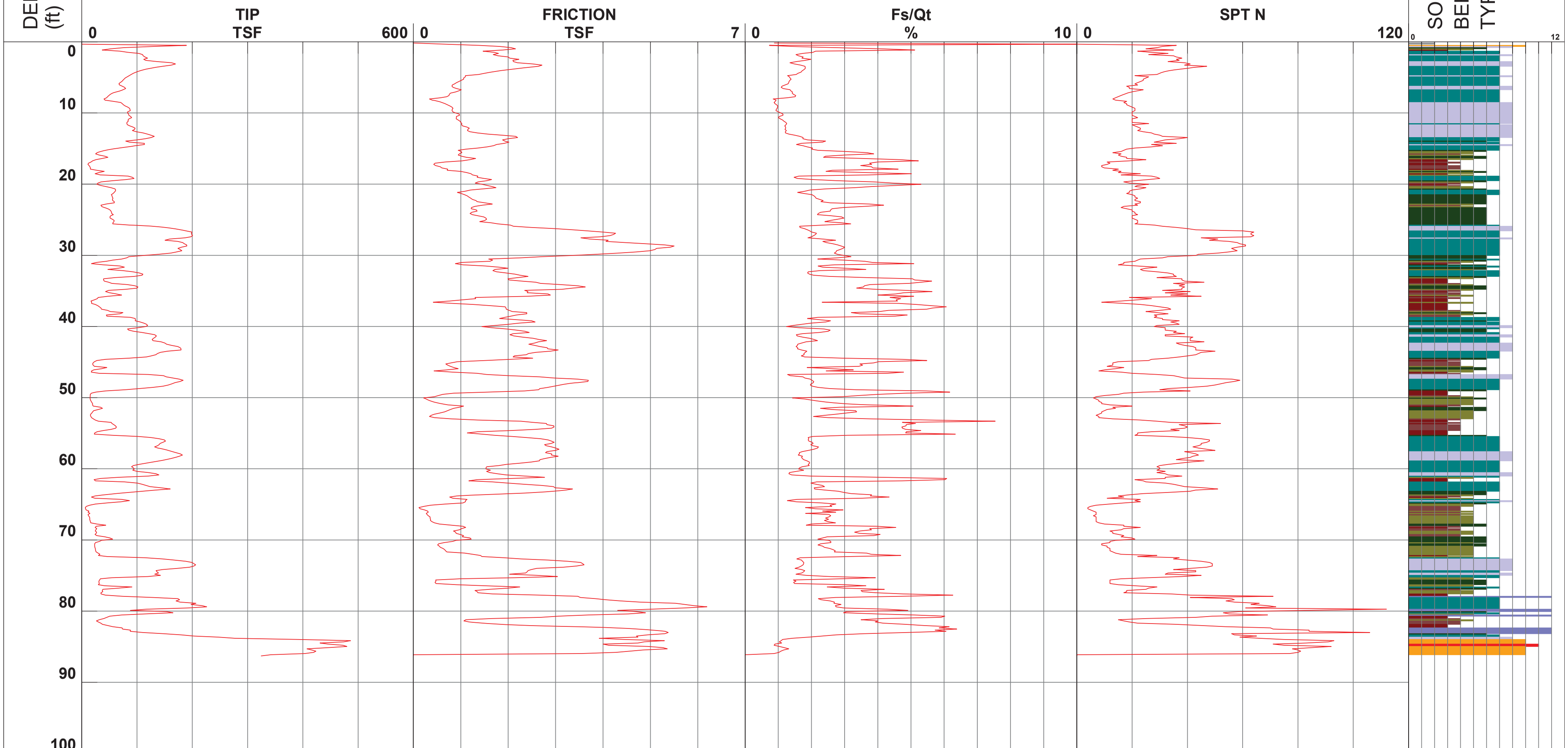
GDC_LOG_BORING_2013 LA1321 B-1,B-3,B-6,B-8.GPJ GDC2013.GDT 5/9/17

Group Delta

Project	Elizabeth Learning Center	Operator	RC JM	Filename	SDF(722).cpt
Job Number	LA1321	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	4/14/2017 11:19:55 AM	Maximum Depth	86.29 ft
EST GW Depth During Test		48.40 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

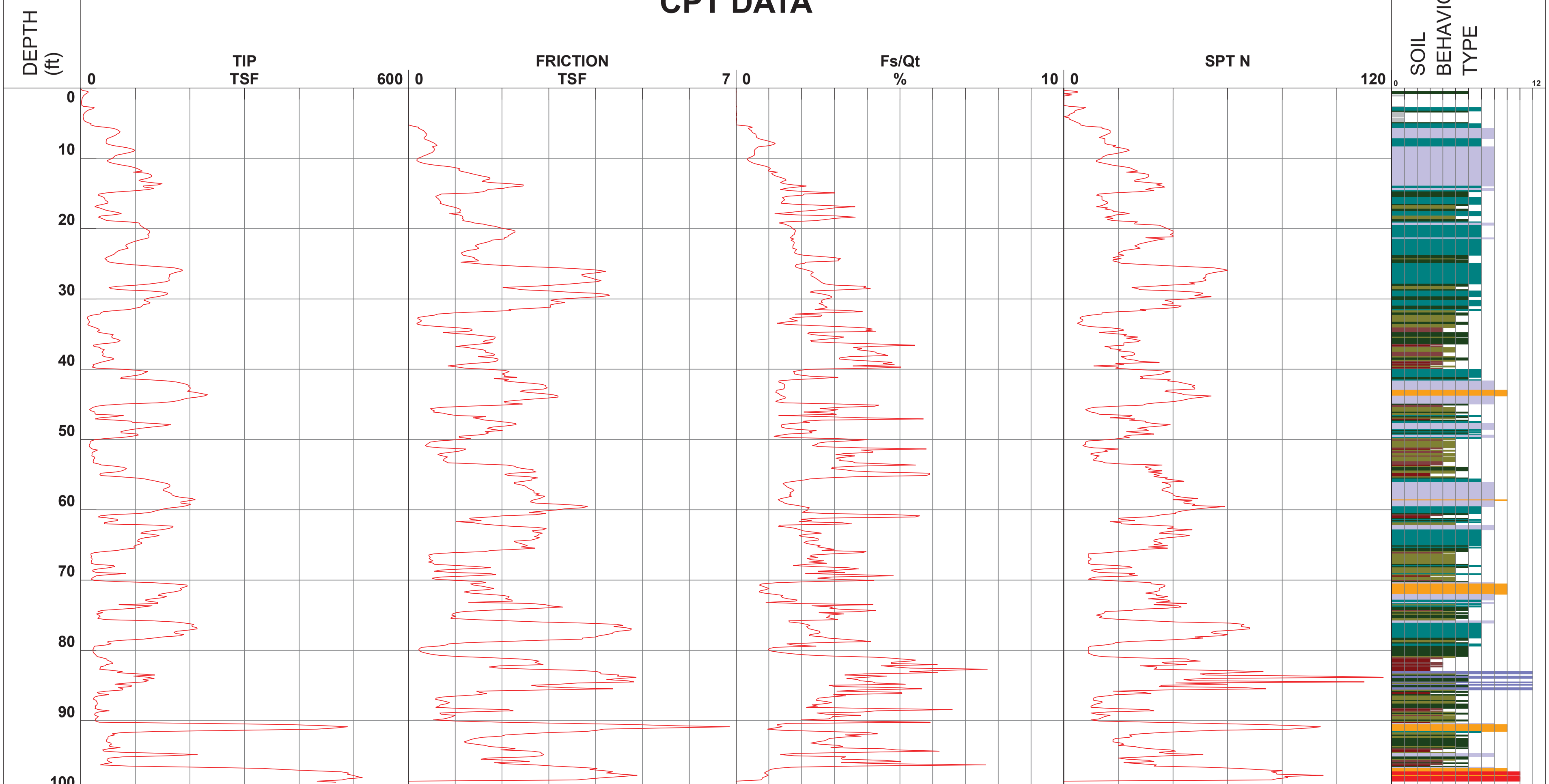
Figure A-5

Group Delta

Project	Elizabeth Learning Center	Operator	RC JM	Filename	SDF(723).cpt
Job Number	LA1321	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	4/14/2017 12:55:50 PM	Maximum Depth	98.75 ft
EST GW Depth During Test		50.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

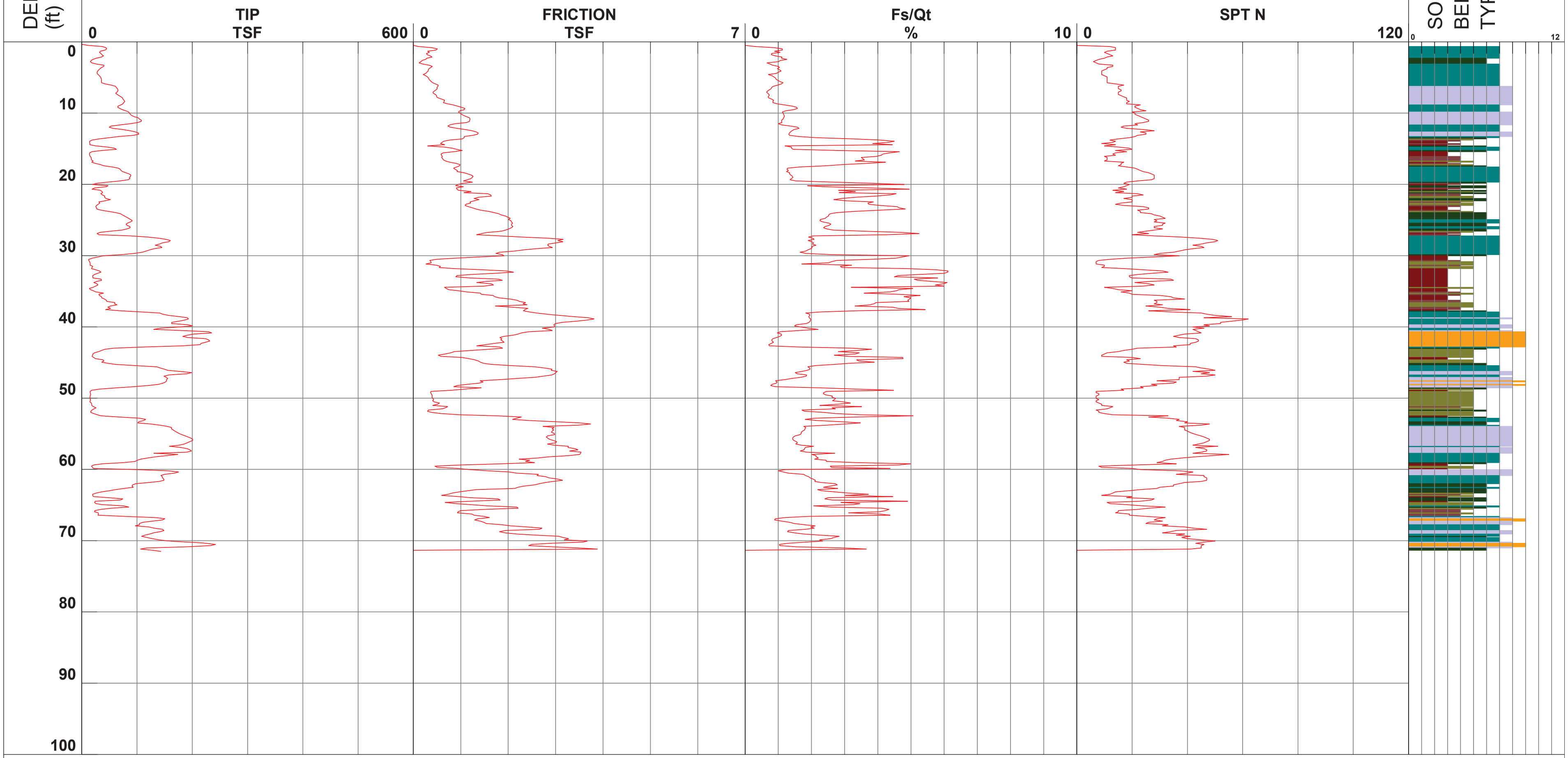
Figure A-6

Group Delta

Project	Elizabeth Learning Center	Operator	RC JM	Filename	SDF(724).cpt
Job Number	LA1321	Cone Number	DDG1281	GPS	
Hole Number	CPT-05	Date and Time	4/14/2017 2:46:15 PM	Maximum Depth	71.52 ft
EST GW Depth During Test		50.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

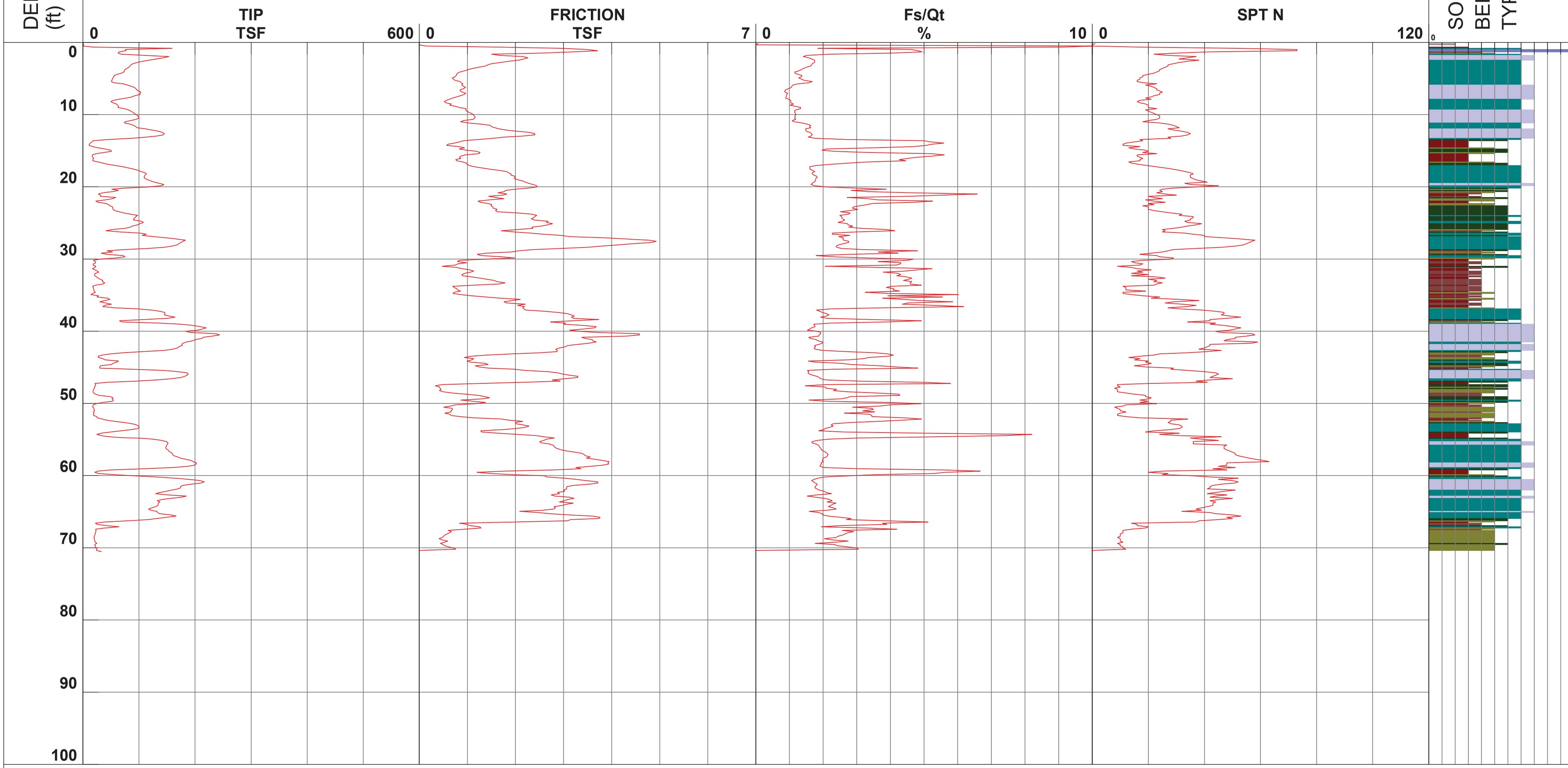
Figure A-7

Group Delta

Project	Elizabeth Learning Center	Operator	RC-JM	Filename	SDF(719).cpt
Job Number	LA1321	Cone Number	DDG1281	GPS	
Hole Number	CPT-07	Date and Time	4/14/2017 7:35:00 AM	Maximum Depth	70.54 ft
EST GW Depth During Test		50.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

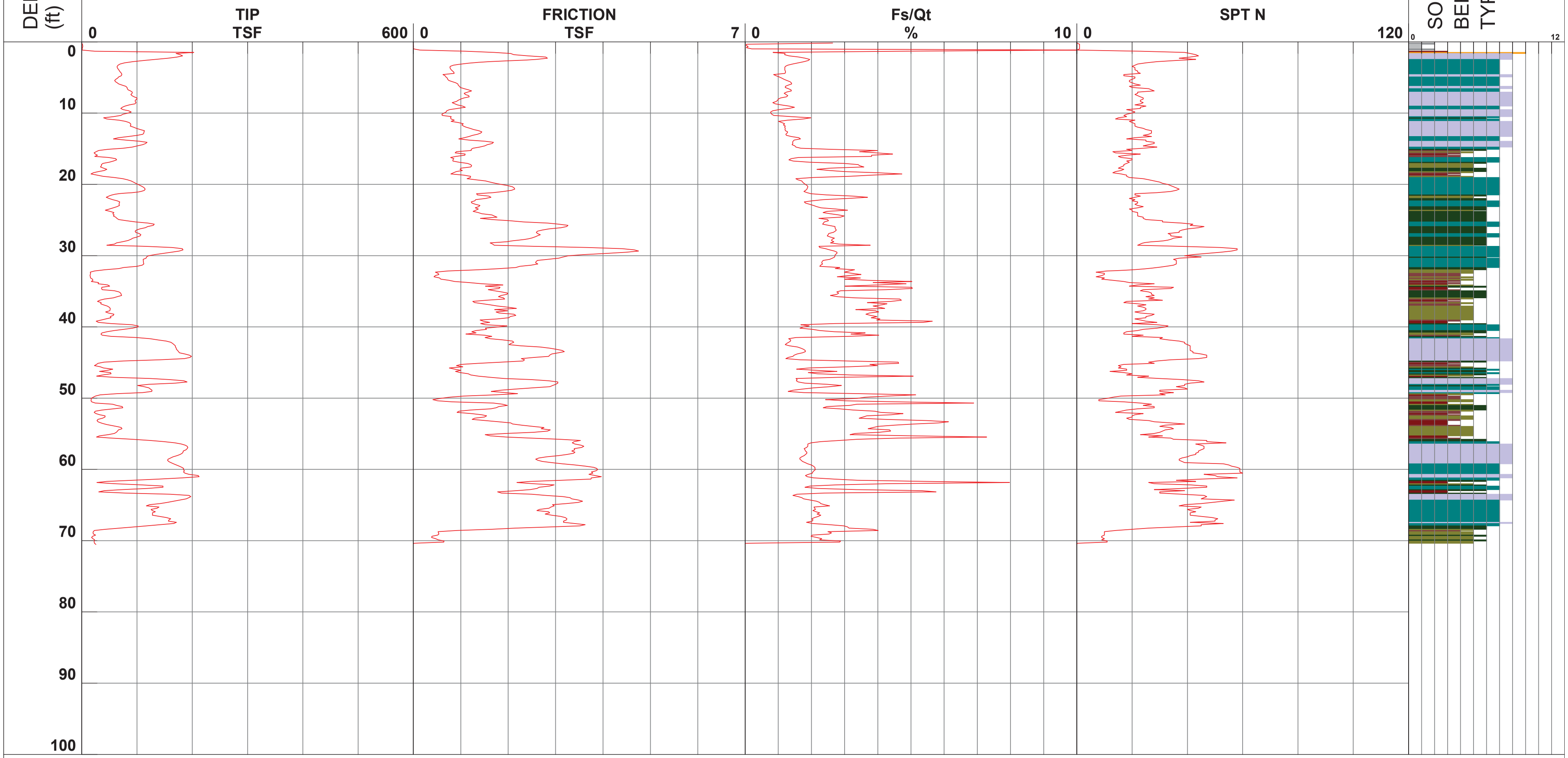
Figure A-8

Group Delta

Project	Elizabeth Learning Center	Operator	RC JM	Filename	SDF(720).cpt
Job Number	LA1321	Cone Number	DDG1281	GPS	
Hole Number	CPT-09	Date and Time	4/14/2017 8:36:07 AM	Maximum Depth	70.54 ft
EST GW Depth During Test		50.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

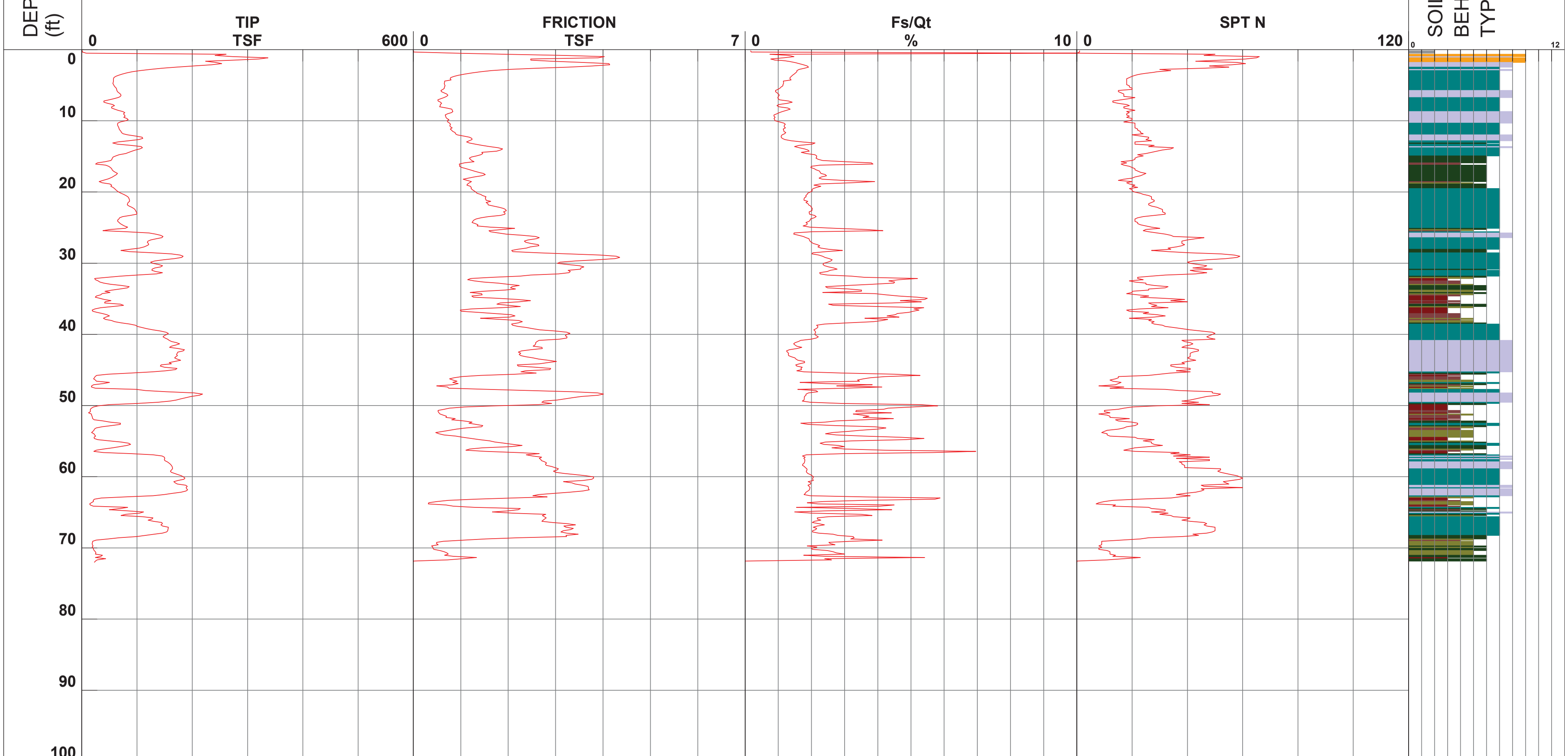
Figure A-9

Group Delta

Project	Elizabeth Learning Center	Operator	RC JM	Filename	SDF(721).cpt
Job Number	LA1321	Cone Number	DDG1281	GPS	
Hole Number	CPT-10	Date and Time	4/14/2017 9:51:31 AM	Maximum Depth	72.01 ft
EST GW Depth During Test		50.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Figure A-10

ATTACHMENTS

Leighton 2006 Investigation

GEOTECHNICAL BORING LOG B-1

Date	8-11-06	**Sheet**	1 of 2
Project	Elizabeth Learning Center	**Project No.**	601506-001
Drilling Co.	Martini Drilling Corporation	**Type of Rig**	CME 75
Hole Diameter	8 inches	**Drive Weight**	140lb Autohammer
Drop	30"		
Elevation Top of Hole	±129'	**Location**	See Geotechnical Map

Logged By JKG
Sampled By JKG

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0								@0': 3 inches asphalt over 4 inches of base.	
									Alluvium (Qal)	
				BAG-1				SP	@0.7': SAND, brown, moist, fine grained.	CR
				R-1	5 7 9			SP	@2.5': SAND, medium dense, olive brown, moist, fine grained.	
	5			R-2	3 5 8	97.5	24.9	SP	@5': Same as above, loose.	DS
				R-3	4 6 10	97.0	7.2	SP	@7.5': Same as above, medium dense.	
	10			S-1	2 3 3			SP	@10': Same as above, loose.	
	15			R-4	6 11 14	116.9	14.5	SP	@15': Same as above, medium dense.	
				S-2	2 3 5			CL-SC	@16.5': Sandy CLAY to Clayey SAND, medium stiff to loose, olive gray, moist, low plasticity, fine grained sand.	
	20			S-3	3 2 3			SP	@20': SAND, loose, olive gray, moist, fine grained, interbedded with CLAY, gray, moist, low plasticity.	
	25			R-5	4 5 10	108.6	17.5	SP	@25': SAND, medium dense, olive gray, moist, fine grained.	
				S-4	4 5 4			SP	@26.5': Same as above, loose.	
	30									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON CONSULTING, INC.

GEOTECHNICAL BORING LOG B-1

Date	8-11-06	**Sheet**	2 of 2
Project	Elizabeth Learning Center	**Project No.**	601506-001
Drilling Co.	Martini Drilling Corporation	**Type of Rig**	CME 75
Hole Diameter	8 inches	**Drive Weight**	140lb Autohammer — **Drop** 30"
Elevation Top of Hole	±129'	**Location**	See Geotechnical Map

Logged By JKG
Sampled By JKG

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30			S-5	6 6 3			ML	@30': Clayey SILT, stiff, olive gray, moist, low plasticity.	SA
	35			R-6	6 7 10	106.7	19.9	ML	@35': Same as above.	
	40			S-6	7 13 17			SP	@40': SAND, dense, gray, very moist, fine to medium grained.	
	45			R-7	6 19 15	100.6	26.3	SC	@45': Clayey SAND, medium dense, blue-gray, fine grained.	
	50			S-7	3 3 4			CL	@50': Sandy CLAY, medium stiff, blue-gray, very moist, fine grained, low plasticity.	
									Total depth of boring: 51.5 feet. No free groundwater encountered during drilling. Hole backfilled with soil cuttings.	
	55									
	60									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON CONSULTING, INC.

GEOTECHNICAL BORING LOG B-2

Date 8-11-06 | Sheet 1 of 2
Project Elizabeth Learning Center | Project No. 601506-001
Drilling Co. Martini Drilling Corporation | Type of Rig CME 75
Hole Diameter 8 inches | Drive Weight 140lb Autohammer | Drop 30"
Elevation Top of Hole ±130' | Location See Geotechnical Map

Logged By JKG
Sampled By JKG

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0								@0': 2 inches asphalt over 3 inches base. **Alluvium (Qal)**	
				BAG-1				SP	@0.5': SAND, brown, moist, fine grained, some silt.	RV
				R-1	10 14 16			SP	@2.5': SAND, medium dense, light olive brown, moist, fine grained.	
	5			R-2	4 7 10	100.1	23.7	SP	@5': Same as above.	DS
				R-3	5 9 10	95.9	3.9	SP	@7.5': Same as above, light tan, fine to medium grained.	
	10			R-4	5 11 15	107.9	6.9	SP	@10': Same as above, light olive brown, fine grained.	
	15			S-1	2 3 4			SC	@15': Clayey SAND, loose, olive, moist, fine grained, low plasticity clay.	
	20			R-5	10 10 11	105.2	10.4	SC	@20': Same as above, medium dense.	
				S-2	1 2 3			ML	@21.5': Sandy SILT, medium stiff, olive, moist, low plasticity, fine grained.	
	25			S-3	1 3 6			ML	@25': Same as above.	SA
	30									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- SA SIEVE ANALYSIS
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

LEIGHTON CONSULTING, INC.

GEOTECHNICAL BORING LOG B-2

Date 8-11-06 **Sheet** 2 of 2
Project Elizabeth Learning Center **Project No.** 601506-001
Drilling Co. Martini Drilling Corporation **Type of Rig** CME 75
Hole Diameter 8 inches **Drive Weight** 140lb Autohammer **Drop** 30"
Elevation Top of Hole ±130' **Location** See Geotechnical Map

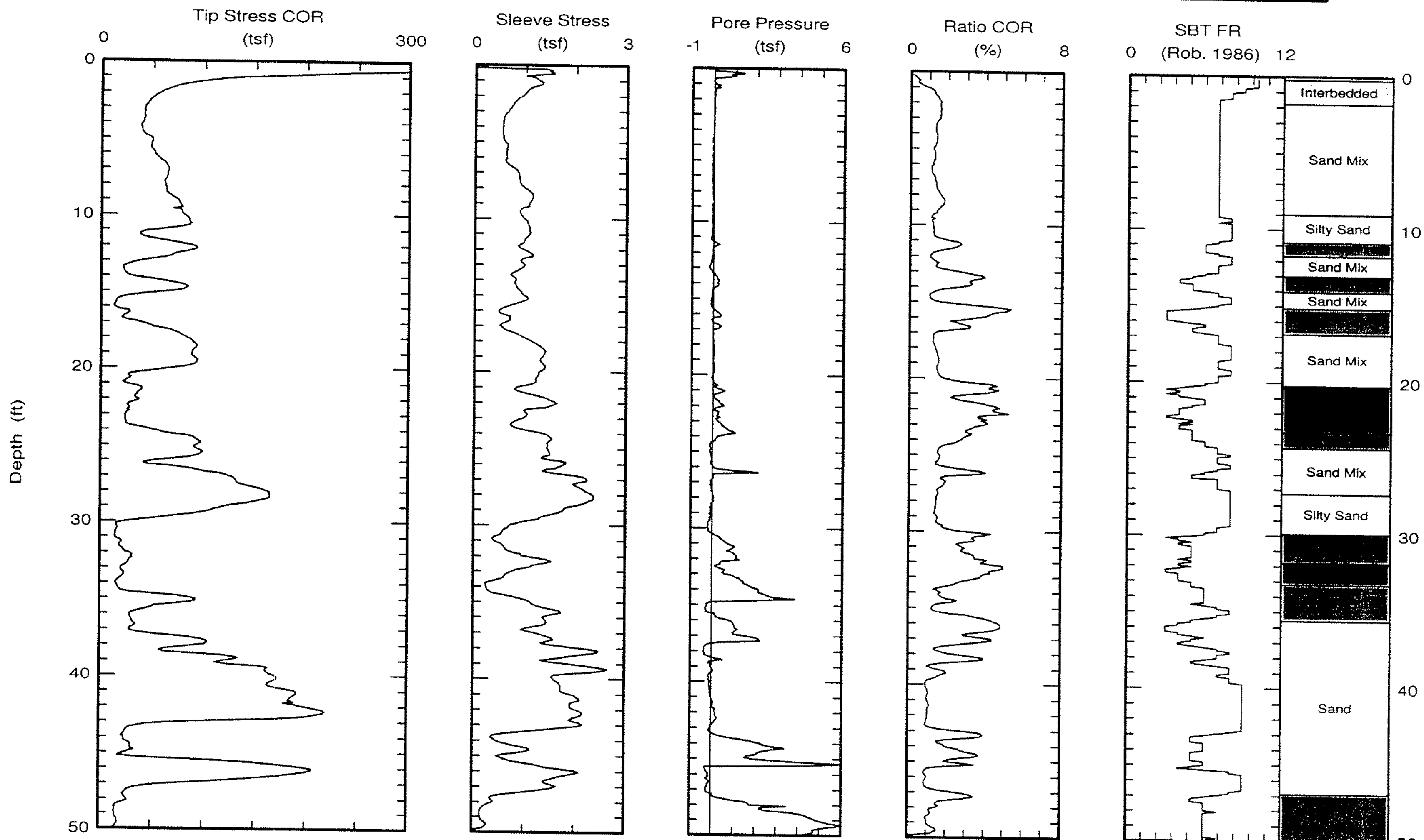
Logged By JKG
Sampled By JKG

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30			R-6	11 17 17	88.4	24.6	SC	@30': Clayey SAND to Silty SAND, medium dense, olive, moist, fine grained.	
				S-4	2 3 4			ML	@31.5': Clayey SILT to Sandy SILT, medium stiff, olive gray, very moist, low plasticity, fine grained sand.	
	35			S-5	2 4 4			ML	@35': Same as above, very moist.	
	40			R-7	6 12 23	109.2	19.8	SP	@40': SAND, medium dense, olive gray, very moist, fine to medium grained.	
				S-6	6 12 17			SP	@41.5': Same as above.	
	45			S-5	2 4 5			CL	@45': Sandy CLAY, medium stiff, olive gray, wet, low plasticity, fine grained sand.	
	50			R-8	7 7 8	98.3	25.7	CL	@50': Same as above.	
									Total depth of boring: 51.5 feet. No free groundwater encountered during drilling. Hole backfilled with soil cuttings.	
	55									
	60									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

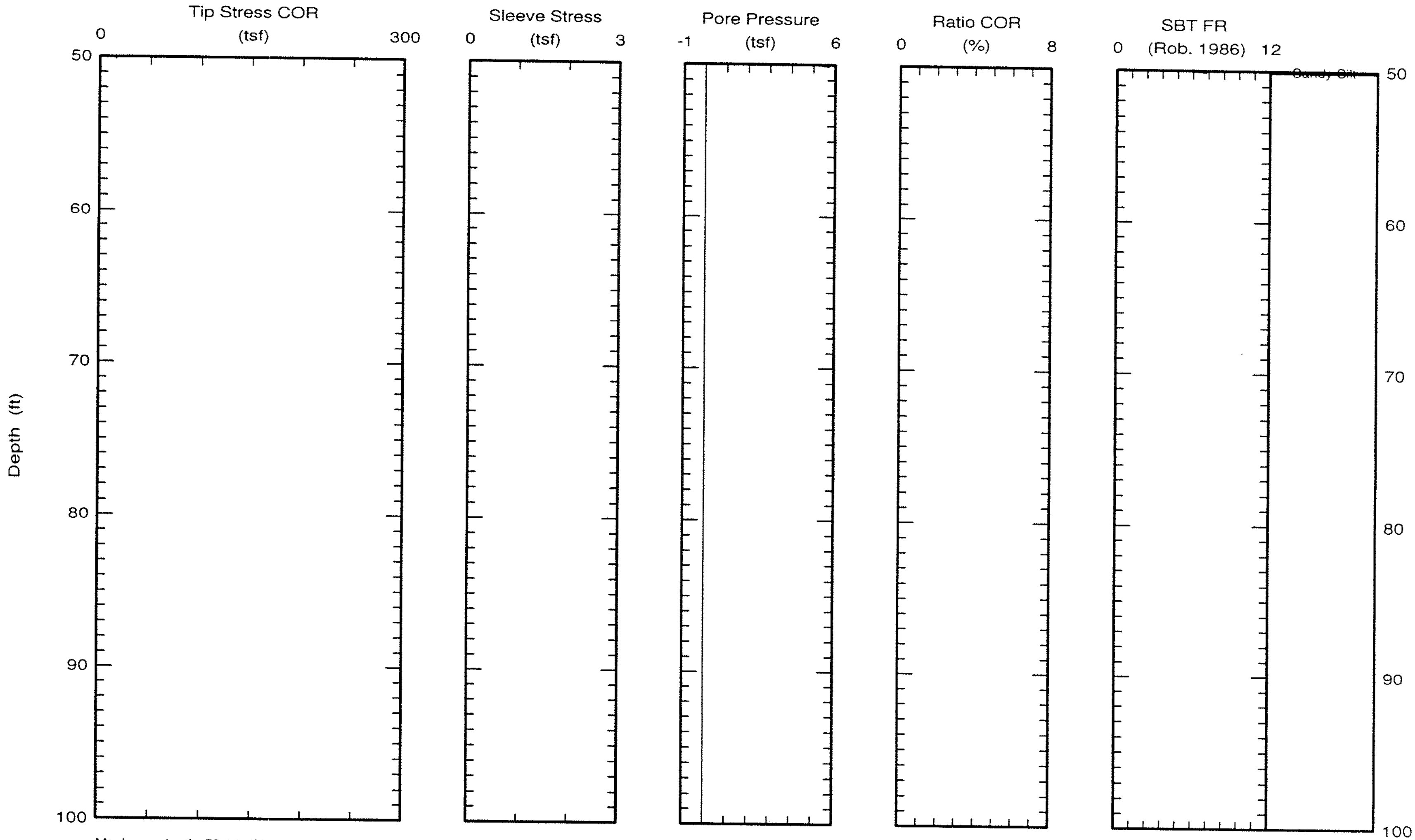
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 11/Aug/2006 Test ID: CPT-1 Project: HuntingtonPark
	Client: Leighton Consulting Job Site: Elizabeth Learning Center	



Maximum depth: 50.14 (ft)
Page 1 of 2

Test ID: CPT-1
File: Z11G0601C.ECP

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 11/Aug/2006 Test ID: CPT-1 Project: HuntingtonPark
	Client: Leighton Consulting Job Site: Elizabeth Learning Center	



Maximum depth: 50.14 (ft)

Test ID: CPT-1
File: Z11G0601C.ECP

Leighton 2007 Investigation

GEOTECHNICAL BORING LOG B-1

Date 2-15-07 | Sheet 1 of 2
Project Elizabeth Learning Center | Project No. 601506-002
Drilling Co. Martini Drilling Co. | Type of Rig CME-75
Hole Diameter 8 inches | Drive Weight 140 lbs Autohammer | Drop 30"
Elevation Top of Hole 130' | Location See Geotechnical Map

Logged By ACS
Sampled By SR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
130	0								@ Surface: 3 inches of asphalt over Silty SAND, brown, moist. **Alluvium (Qya)**	CR, RV
				Bag-1				SP/SM	@ 2': SAND to Silty SAND, brown, moist, fine grained.	
125	5			R-1	3 5 7	97.5	16.2	SM/SP	@ 5': Top- Silty SAND with gravel, brown, loose, moist, fine to coarse sand, 1/2" gravel. Bottom- SAND, brown, loose, moist, fine grained.	DS
				R-2	3 5 7	105.6	27.3	SM/ML	@7.5': Silty SAND to Sandy SILT, brown, loose, moist, fine grained sand.	DS
120	10			S-1	2 3 2			SM	@10': Silty SAND, brown, loose, moist, fine grained sand.	
115	15			R-3	3 5 6	103.8	22.8	SM	@15': Silty SAND, brown, loose, moist, fine grained sand.	
110	20			S-2	1 3 5			SM	@20': Silty SAND, brown, loose, moist, fine grained sand.	
105	25			R-4	5 9 11	95.5	27.5	SM/ML	@25':Top- Silty SAND, , brown, medium dense, moist, fine grained sand. Bottom- Sandy SILT, brown, stiff, moist, fine grained sand.	
100	30									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON CONSULTING, INC.

GEOTECHNICAL BORING LOG B-1

Date 2-15-07 Sheet 2 of 2
Project Elizabeth Learning Center Project No. 601506-002
Drilling Co. Martini Drilling Co. Type of Rig CME-75
Hole Diameter 8 inches Drive Weight 140 lbs Autohammer Drop 30"
Elevation Top of Hole 130' Location See Geotechnical Map

Logged By ACS
Sampled By SR

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
100	30			S-3	1 2 2			ML	@ 30': Sandy SILT, grayish brown, soft, moist, fine grained sand.	SA
95	35			R-5	6 11 14	109.3	18.0	SM	@ 35': Silty SAND, gray, medium dense, moist to wet, fine to medium grained sand.	
90	40			S-4	6 12 18			SP	@40': SAND, gray, medium dense to dense, wet, fine to medium grained sand.	
85	45			R-6	8 20 15	104.1	23.1	SP	@45': SAND, gray, medium dense, wet, fine to medium grained sand.	
80	50			S-5	2 3 5			SP	@50': SAND, gray, wet, loose, fine to medium grained sand.	
									Total depth of boring: 51.5 feet. Groundwater was encountered @ 40 feet during drilling. Boring backfilled with soil cutting and patched with cold- mix asphalt concrete.	
75	55									
70	60									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON CONSULTING, INC.

GEOTECHNICAL BORING LOG B-2

Date 2-15-07 **Sheet** 1 **of** 2

Project Elizabeth Learning Center **Project No.** 601506-002

Drilling Co. Martini Drilling Co. **Type of Rig** CME-75

Hole Diameter 8 inches **Drive Weight** 140 lbs Autohammer **Drop** 30"

Elevation Top of Hole 130' **Location** See Geotechnical Map

Logged By ACS

Sampled By SR/ACS

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
130	0								@ Surface: 3 inches of asphalt over Silty SAND, brown, moist. <u>Alluvium (Qya)</u>	
				Bag-1				SP	@ 2': SAND with silt, gray, moist, fine grained sand.	
125	5			R-1	3 7 11	97.6	24.1	SM	@ 5': Silty SAND, brown, medium dense, moist, fine grained sand.	DS
				R-2	4 6 10	103.6	3.2	SP-SM	@ 7.5': Silty SAND to SAND, brown, loose to medium dense, moist, fine-medium grained sand.	
120	10			R-3	5 7 8	101.9	5.0	SP/SM	@ 10': Silty SAND to SAND, brown, loose, moist, fine-medium grained sand.	
115	15			S-1	1 2 3			ML	@ 15': Sandy SILT, brown, medium stiff, moist, fine grained.	
110	20			R-4	7 8 8	99.9	17.3	ML/SM	@ 20': Sandy SILT to Silty SAND, brown, loose, moist, fine to medium grained sand.	
105	25			S-2	3 4 5			ML	@ 25': Sandy SILT, brown, stiff, moist, fine grained.	
100	30									

SAMPLE TYPES:

- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:

- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- SA SIEVE ANALYSIS
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

LEIGHTON CONSULTING, INC.

GEOTECHNICAL BORING LOG B-2

Date: 2-15-07 | Sheet 2 of 2
Project: Elizabeth Learning Center | Project No.: 601506-002
Drilling Co.: Martini Drilling Co. | Type of Rig: CME-75
Hole Diameter: 8 inches | Drive Weight: 140 lbs Autohammer | Drop: 30"
Elevation Top of Hole: 130' | Location: See Geotechnical Map

Logged By: ACS
Sampled By: SR/ACS

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
100	30			R-5	3 4 7	97.2	28.7	CL-ML	@ 30': Sandy SILT to Silty CLAY, brown, medium stiff, moist, fine grained.	
95	35			S-3	1 3 5			SM	@ 35': Silty SAND, grayish brown, loose, moist to wet, fine grained sand.	
90 ▽	40			R-6	6 15 26	107.1	19.4	SP	@ 40': SAND, olive gray, medium dense, wet, fine to medium grained sand.	
85	45			S-4	3 7 10			SP	@ 45': SAND, , olive gray, medium dense, wet, fine grained sand.	
80	50			R-7	4 7 10	101.7	24.9	CL-ML	@ 50': Sandy SILT to Clayey SILT, olive gray, stiff, moist, fine grained.	
									Total depth of boring: 51.5 feet. Groundwater was encountered @ 40 feet during drilling. Boring backfilled with soil cuttings and patched with cold-mix asphalt concrete.	
75	55									
70	60									

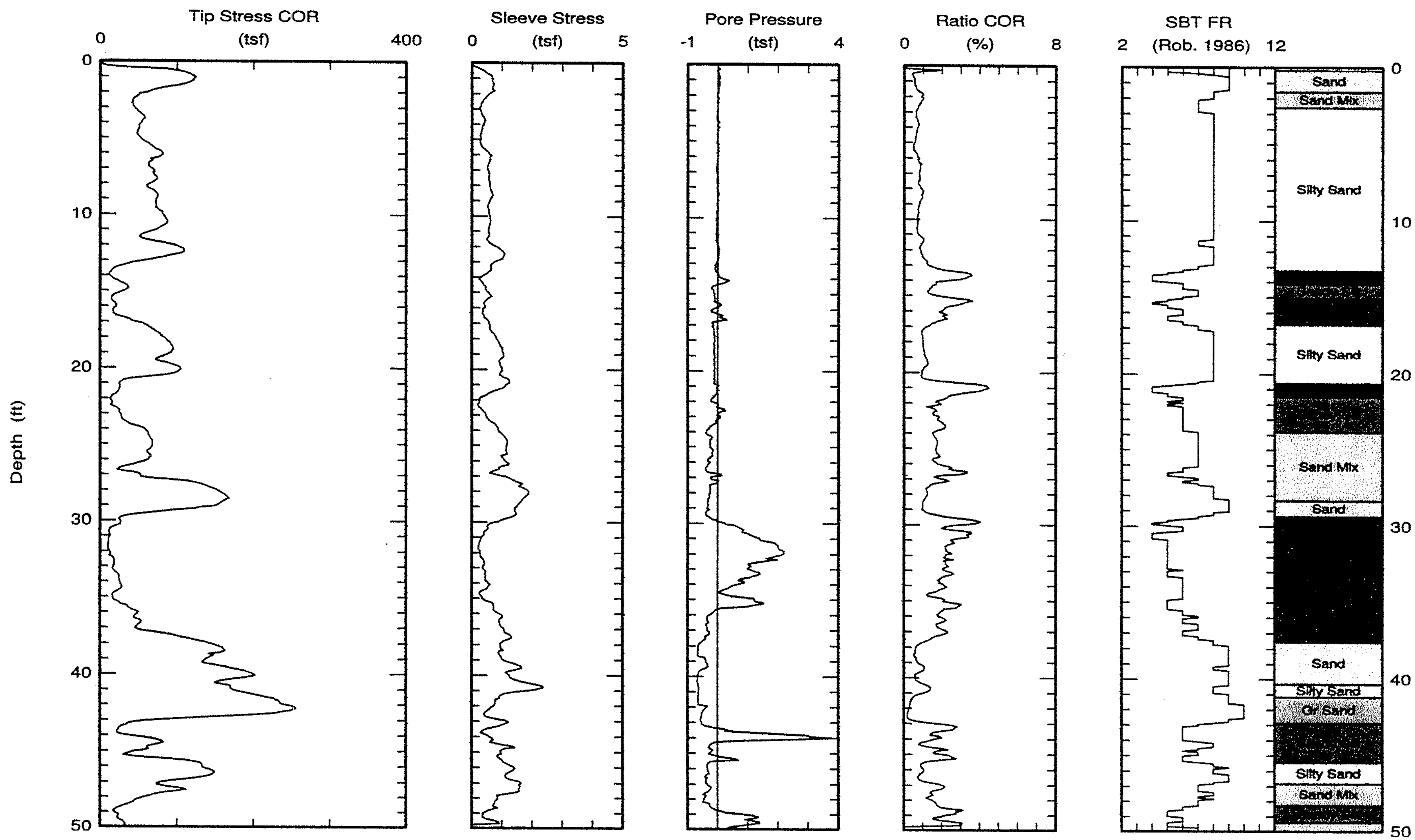
SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- SA SIEVE ANALYSIS
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

LEIGHTON CONSULTING, INC.

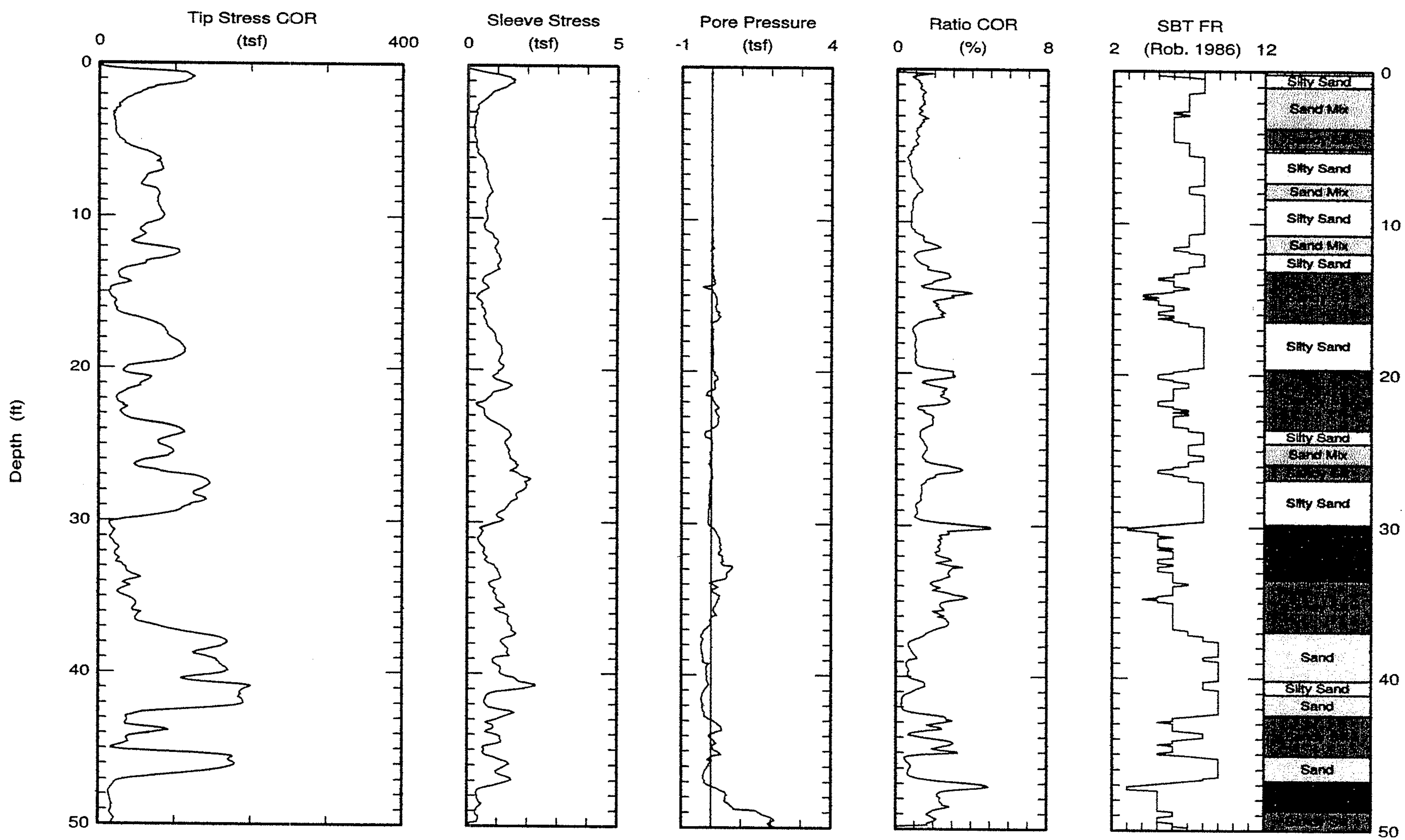
Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

CPT Data 30 ton rig	Date: 15/Feb/2007 Test ID: CPT-1 Project: Cudahy
Customer: Leighton Consulting Job Site: Elizabeth Learning Center	



Maximum depth: 50.12 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 15/Feb/2007 Test ID: CPT-2 Project: Cudahy
	Customer: Leighton Consulting Job Site: Elizabeth Learning Center	



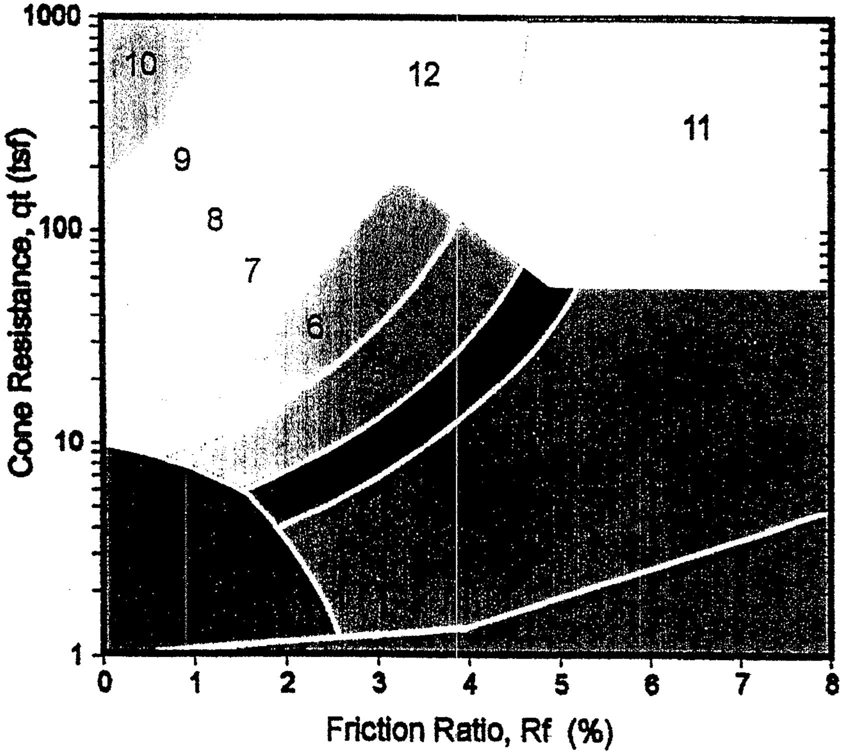
Maximum depth: 50.09 (ft)

Page 1 of 2

KEHOE TESTING & ENGINEERING

CPT Classification Chart

(after Robertson and Campanella, 1988)



Zone	q_t / N	Soil Behavior Type	UCSCS
1	2	sensitive fine grained	OL-OH
2	1	organic material	Pt-OH
3	1	clay	CH
4	1.5	silty clay to clay	CL-CH
5	2	clayey silt to silty clay	ML-CL
6	2.5	sandy silt to clayey silt	MH-ML
7	3	silty sand to sandy silt	SM-ML
8	4	sand to silty sand	SP-SM
9	5	sand	SP
10	6	gravelly sand to sand	SW-SP
11	1	very stiff fine grained *	CL-MH
12	2	sand to clayey sand *	SP-SC

* overconsolidated or cemented

APPENDIX B – LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

B.1 Introduction

The laboratory testing was performed using appropriate American Society for Testing and Materials (ASTM) and Caltrans Test Methods (CTM).

Modified California drive samples, Standard Penetration Test (SPT) drive samples, and bulk samples collected during the field investigation were carefully sealed in the field to prevent moisture loss. The samples of earth materials were then transported to the laboratory for further examination and testing. Tests were performed on selected samples as an aid in classifying the earth materials and to evaluate their physical properties and engineering characteristics. Laboratory testing for this investigation included:

- Soil Classification: USCS (ASTM D 2487) and Visual Manual (ASTM D 2488);
- Moisture content (ASTM D 2216) and Dry Unit Weight (ASTM D 2937);
- Atterberg Limits (ASTM D 4318);
- Grain Size Distribution (ASTM D 422) & % Passing #200 Sieve (ASTM D 1140);
- Pocket Penetrometer;
- Expansion Index (D 4829);
- R-Value (ASTM D2844, CTM 301)
- Soil Corrosivity:
  - pH (CTM 643);
  - Water-Soluble Sulfate (ASTM D 516, CTM 417);
  - Water-Soluble Chloride(Ion-Specific Probe, CTM 422);
  - Minimum Electrical Resistivity (CTM 643);

Applicable lab results from previous Leighton 2006 and 2007 investigations are attached at the end of this appendix. Brief descriptions of the laboratory testing program and test results are presented below.

B.2 Moisture Content and Dry Unit Weight

The natural moisture content of selected SPT and California ring samples and dry unit weight of California ring samples were determined in general accordance with ASTM D 2216 and ASTM D2937. Results of these tests are presented on the boring logs in Appendix A.

B.3 Atterberg Limits

Soil plasticity was evaluated by measuring the Atterberg limits. This test includes Liquid Limit (LL) and Plastic Limit (PL) tests to determine the Plasticity Index (PI) in accordance with ASTM D4318.

Results of these tests are illustrated in the plasticity chart shown in Figures B-1.1 to B-1.2 and on the boring logs of Appendix A.

B.4 Grain Size Distribution and Percent Passing No. 200 Sieve

Determination of fines verses coarser soil particles was performed by the percent #200 Sieve test. Representative samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. The percentage of fines (soil passing No. 200 sieve) was determined in accordance with ASTM D1140. The washed fraction retained on the No. 200 sieve was then screened on a No. 4 sieve, and the fraction retained on No. 4 was weighed to determine the percentage of gravel. The results of percent passing No. 200 sieve is presented in the boring logs in Appendix A.

B.5 Pocket Penetrometer

Compressive soil strength of cohesive samples were measured using a pocket penetrometer. The measured values (in tsf) are presented in the boring logs of Appendix A.

B.6 Expansion Index

The expansion potential of the site soils was estimated using the Expansion Index Test in accordance with ASTM D 4829. The results of this test are listed in Table B-1.

B.7 R-Value

An R-Value test was performed to measure the potential strength of the upper soils on site to use as potential subgrade. The results of this test are shown in Figures B-2.1.

B.8 Soil Corrosivity

Tests were performed in order to determine corrosion potential of site soils on concrete and ferrous metals. Corrosivity testing included minimum electrical resistivity and soil pH (Caltrans method 643), water soluble chlorides (Orion 170A+ Ion Probe or Caltrans Test Method 422), and water-soluble sulfates (ASTM D 516). The test results are summarized in Table B-2.

B.9 List of Attached Tables and Figures

The following tables and figures are attached and complete this appendix:

Table B-1	Summary of Expansion Index
Table B-2	Summary of Soil Corrosivity
Figures B-1.1 to B-1.2	Atterberg Limits Test Results
Figures B-2.1	R-Value Test Results
Attachment	Previous Leighton lab results

TABLES

Table B-1
Summary of Expansion Index

Boring No.	Depth (ft)	Expansion Index
Bulk-1	0-3	0

Table B-2
Summary of Soil Corrosivity

Boring No.	Depth (ft)	pH	Sulfate Content (%)	Chloride Content (%)	Minimum Resistivity (ohm-cm)
Bulk-1	0-3	9.08	1.15	<0.01	2,046

FIGURES

ATTERBERG LIMITS

ASTM D-4318 / AASTHO T-89 / CTM 204

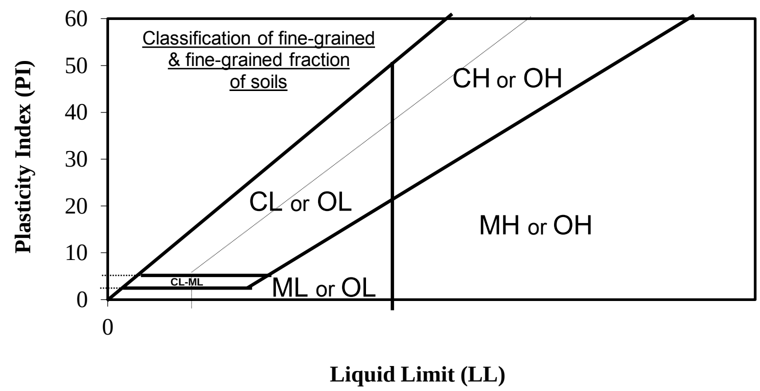
Project Name:	Elizabeth LC	Tested By :	E.D.	Date:	05/01/17
Project No. :	LA1321	Data Input By:	E.D.	Date:	05/02/17
Boring No.:	B-6	Checked By:	D.R.	Date:	05/02/17
Sample No. :	R-5	Depth (ft.) :	12.5-14'		
Initial Moisture:		Container No.:	AL-1		
Description.:	Olive Gray Silty Sand (SM)	*NON PLASTIC*			

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]						
Container No.						
Wet Wt. of Soil + Cont. (gm.)						
Dry Wt. of Soil + Cont. (gm.)						
Wt. of Container (gm.)						
Moisture Content (%) [Wn]						

LIQUID LIMIT	
PLASTIC LIMIT	
PLASTICITY INDEX	**NP**

PI at "A" - Line = 0.73(LL-20) =

One - Point Liquid Limit Calculation $LL=Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint Wet Preparation
- [X] Dry Preparation
  Multipoint Dry Preparation
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test

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ATTERBERG LIMITS

ASTM D-4318 / AASTHO T-89 / CTM 204

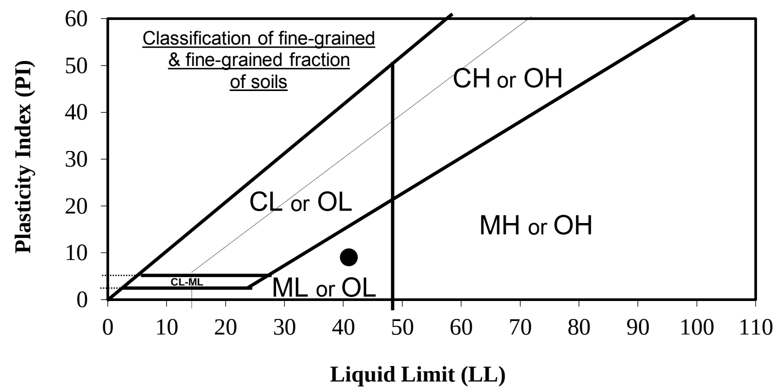
Project Name:	Elizabeth LC	Tested By :	E.D.	Date:	05/01/17
Project No. :	LA1321	Data Input By:	E.D.	Date:	05/02/17
Boring No.:	B-1	Checked By:	D.R.	Date:	05/02/17
Sample No. :	S-10-1	Depth (ft.) :	31-31.5'		
Initial Moisture:		Container No.:	AL-2		
Description.:	Olive Gray Silt (ML)				

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			32	24	16	
Container No.	A	B	C	D	E	
Wet Wt. of Soil + Cont. (gm.)	25.79	26.14	24.93	24.05	25.32	
Dry Wt. of Soil + Cont. (gm.)	23.27	23.51	22.21	21.53	22.21	
Wt. of Container (gm.)	15.26	15.17	15.26	15.41	15.04	
Moisture Content (%) [Wn]	31.46	31.53	39.14	41.18	43.38	

LIQUID LIMIT	**41**
PLASTIC LIMIT	**32**
PLASTICITY INDEX	**9**

PI at "A" - Line = 0.73(LL-20) = **15.3**

One - Point Liquid Limit Calculation $LL=W_n(N/25)^{0.121}$



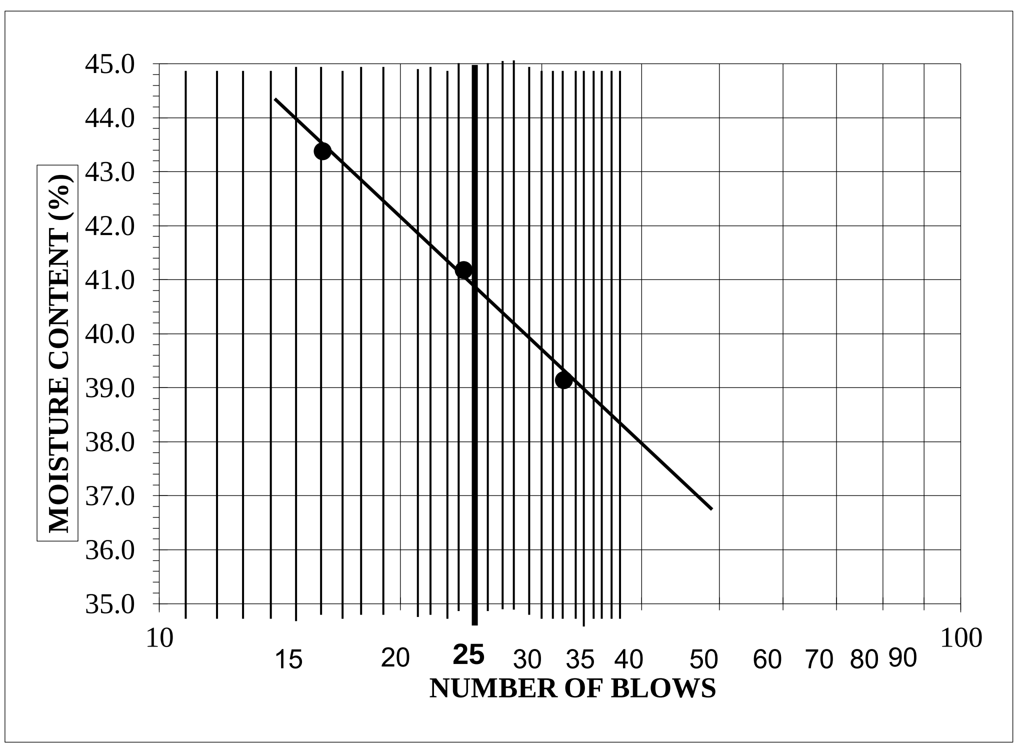
PROCEDURES USED

☐ Wet Preparation
Multipoint Wet Preparation

☒ Dry Preparation
Multipoint Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



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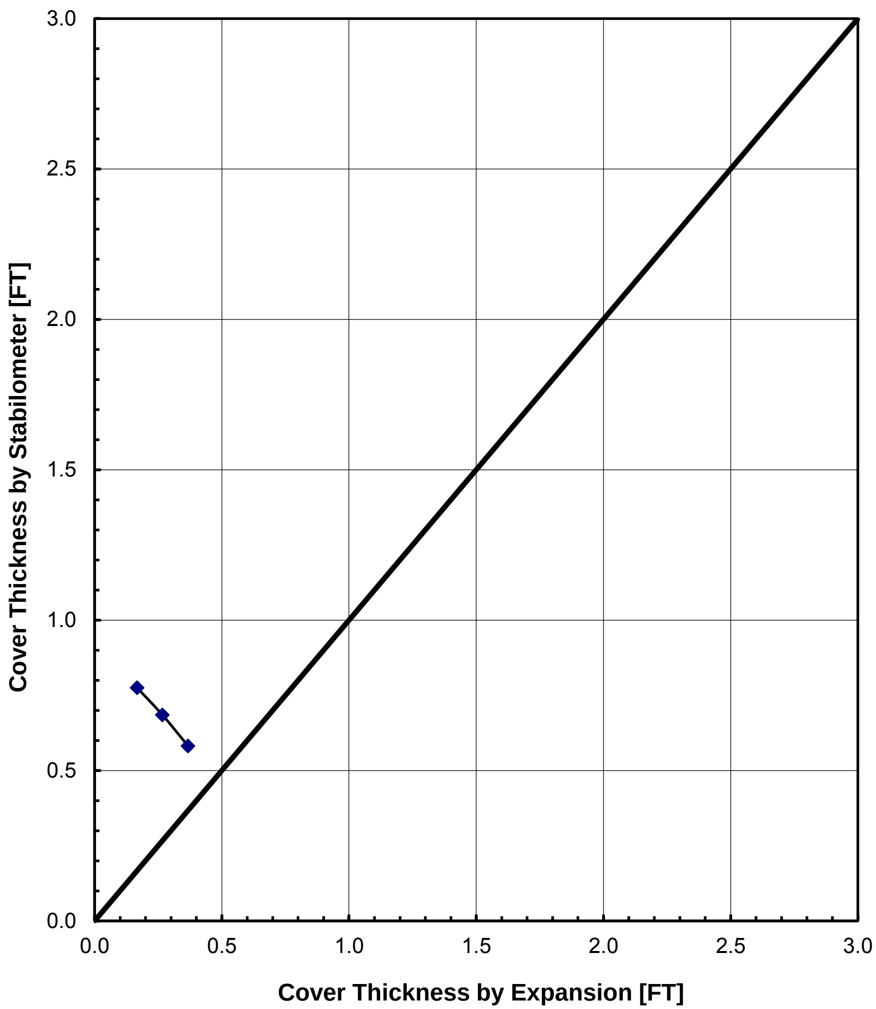
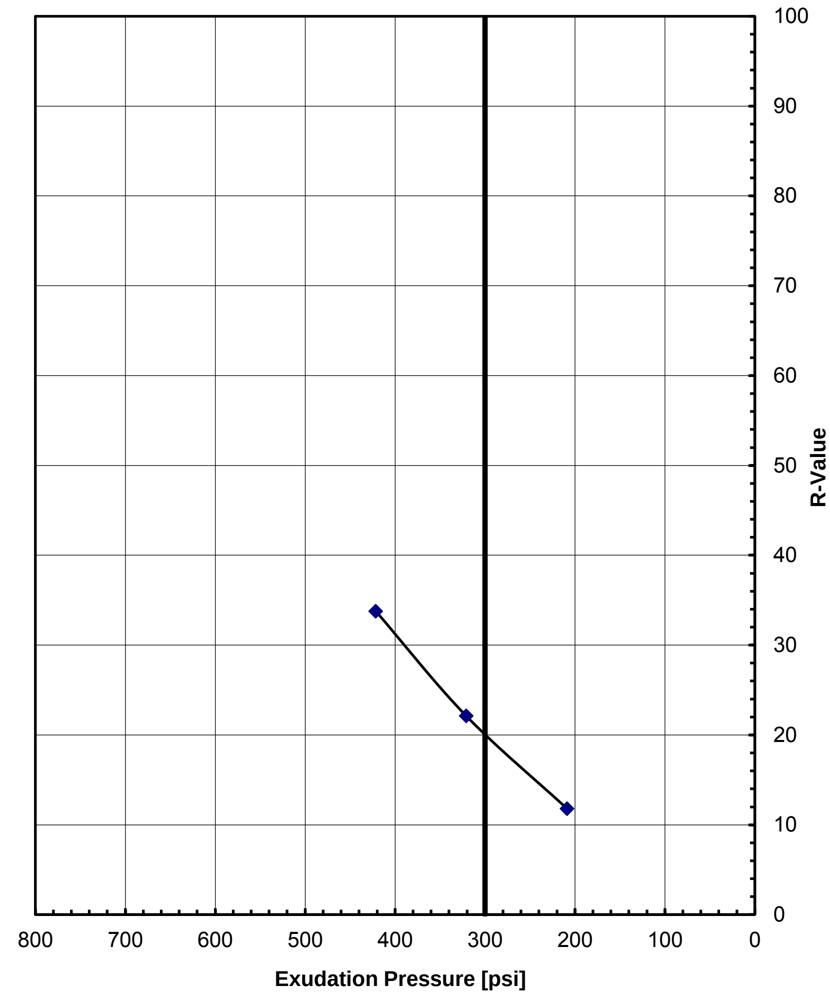
Sample: SO.4541, B-6 Bulk O @ 0-3'

R-Value at Equilibrium: 20





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COVER AND EXUDATION CHARTS

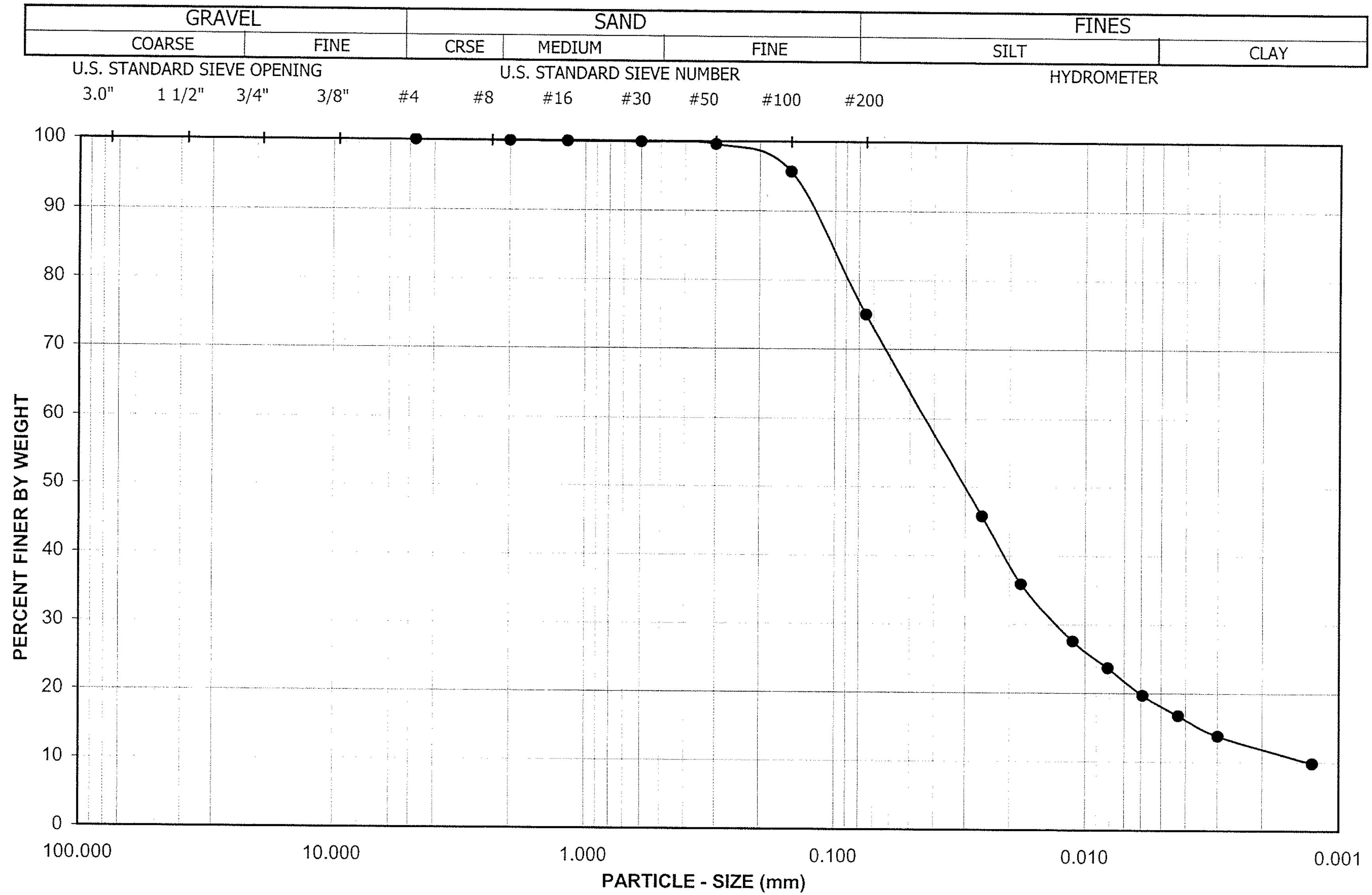
Project No. LA-1321

Project Name: Elizabeth LC

FIGURE B-2.1

ATTACHMENT

Leighton 2006 Investigation



Project Name: Elizabeth LC

Project No.: 601506-001

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

Exploration No.: B-1 Sample No.: S-5

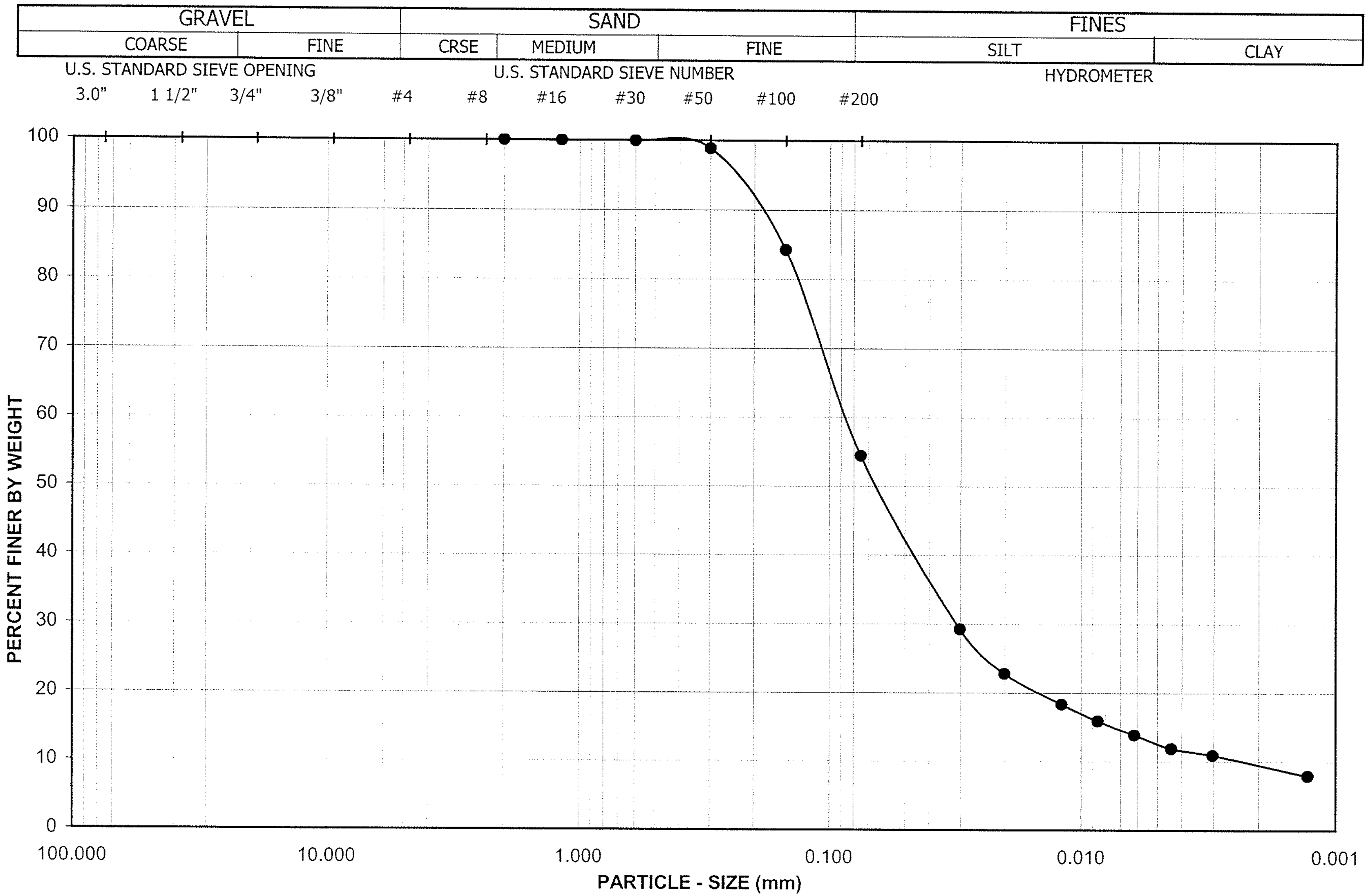
Depth (feet) : 30.0 Soil Type : (ML/MH)s

Soil Identification: Dark olive silt / elastic silt with sand (ML/MH)s

GR:SA:FI : (%) **0 : 25 : 75**

Apr-07

SA & Hyd B-1 S-5.xls



Project Name: Elizabeth LC

Project No.: 601506-001

Exploration No.: B-2 Sample No.: S-3

Depth (feet) : 25.0 Soil Type : s(ML)

Soil Identification: Olive sandy silt s(ML)

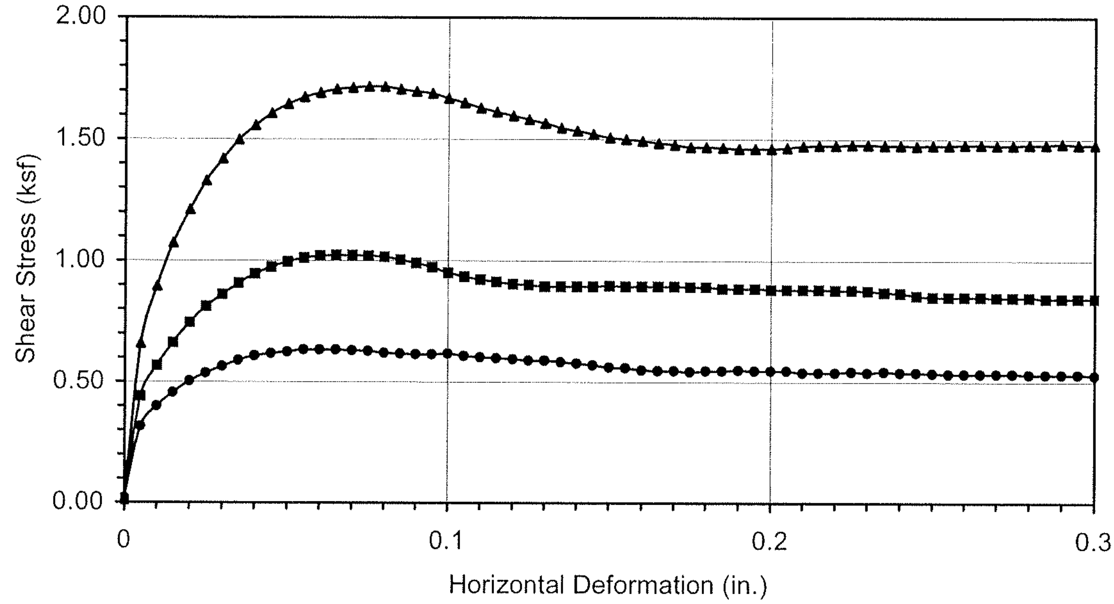
GR:SA:FI : (%) 0 : 46 : 54

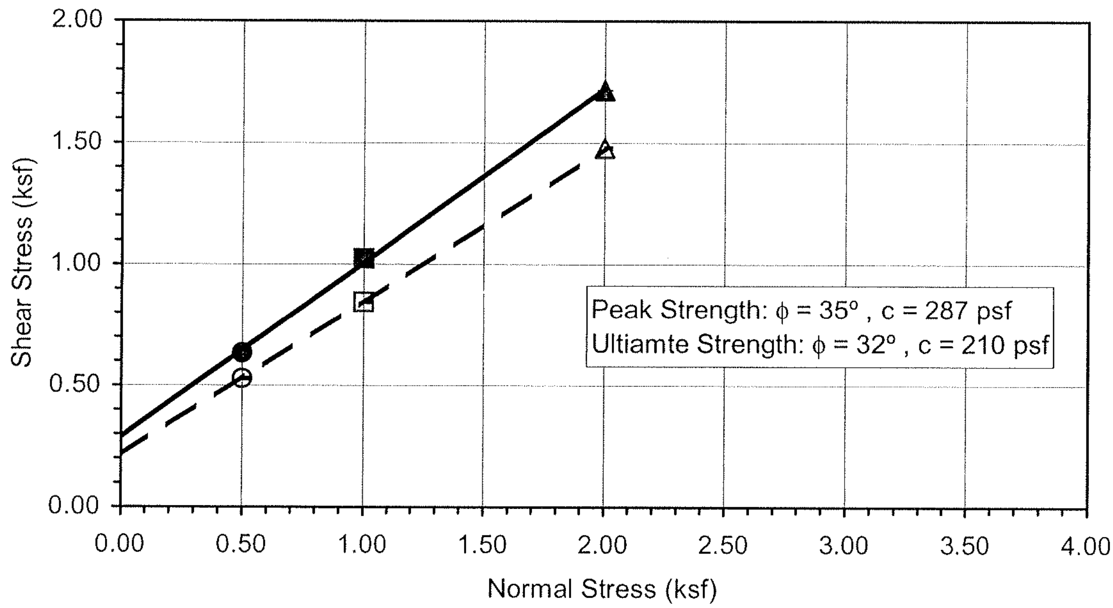
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

Apr-07

SA & Hyd B-2 S-3.xls





Boring No.	**B-1**
Sample No.	**R-2**
Depth (ft)	**5**

Sample Type:

Drive

Soil Identification:

Olive poorly graded sand with silt (SP-SM)

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.633	■ 1.023	▲ 1.715
Shear Stress @ End of Test (ksf)	○ 0.527	□ 0.845	△ 1.478
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.32	9.32	9.32
Dry Density (pcf)	97.2	97.5	97.9
Saturation (%)	34.3	34.6	34.8
Soil Height Before Shearing (in.)	0.9930	0.9883	0.9841
Final Moisture Content (%)	25.7	24.9	24.1

Leighton

DIRECT SHEAR TEST RESULTS

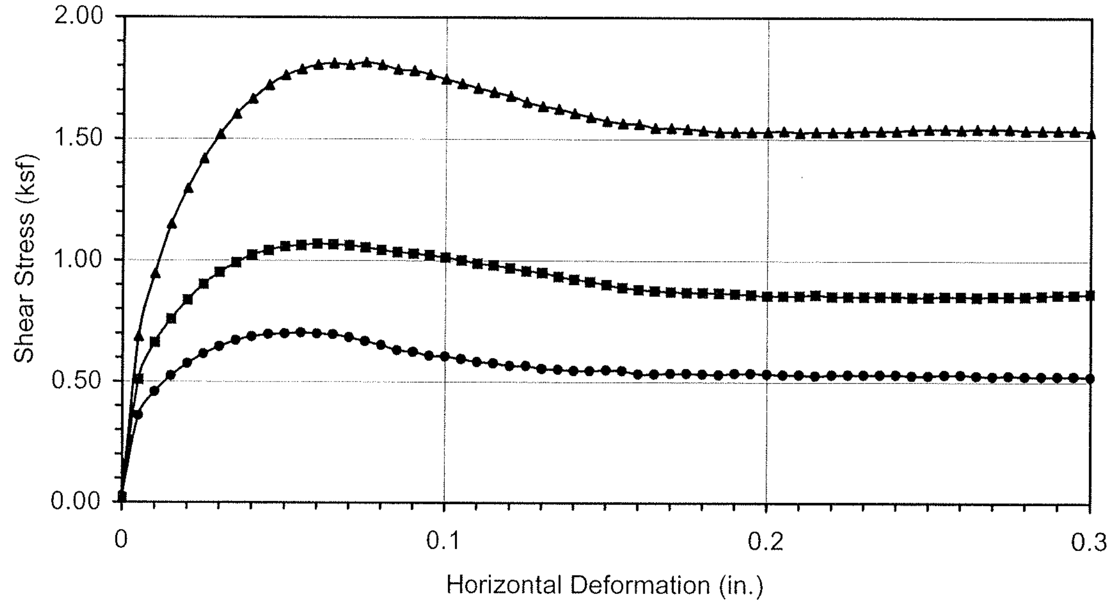
Consolidated Undrained

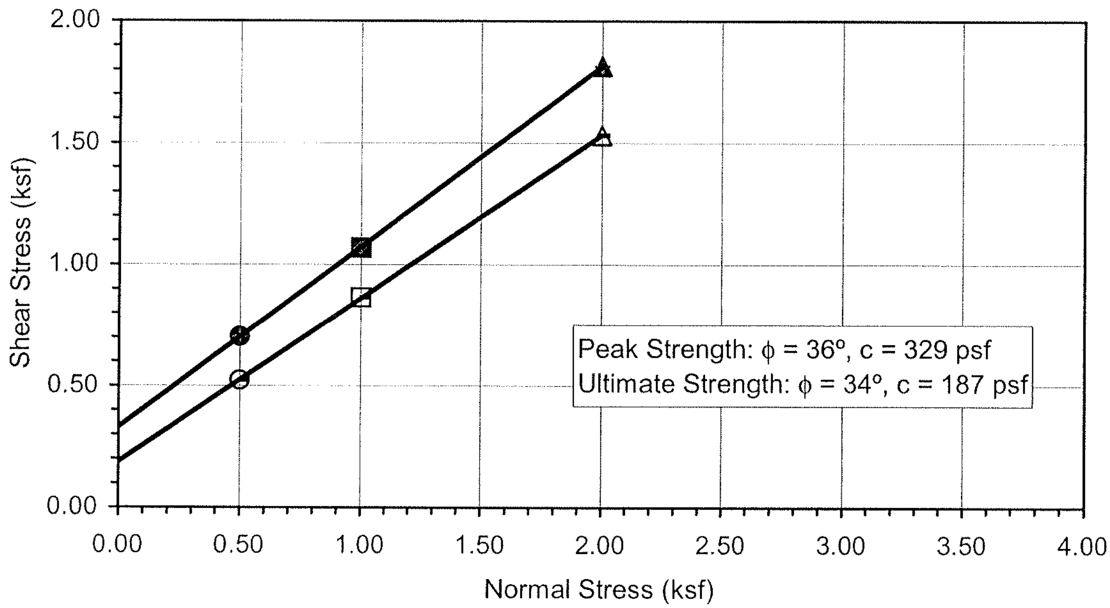
Project No.: 601506-001

Elizabeth LC

08-06

DS B-1 R-2.xls





Boring No.	**B-2**
Sample No.	**R-2**
Depth (ft)	**5**

Sample Type:

Drive

Soil Identification:
Olive poorly graded sand (SP)

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.702	■ 1.069	▲ 1.815
Shear Stress @ End of Test (ksf)	○ 0.521	□ 0.864	△ 1.531
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	3.95	3.95	3.95
Dry Density (pcf)	98.6	100.9	100.8
Saturation (%)	15.0	15.9	15.9
Soil Height Before Shearing (in.)	0.9926	0.9885	0.9810
Final Moisture Content (%)	24.0	23.5	23.7

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DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 601506-001

Elizabeth LC

08-06

DS B-2 R-2.xls

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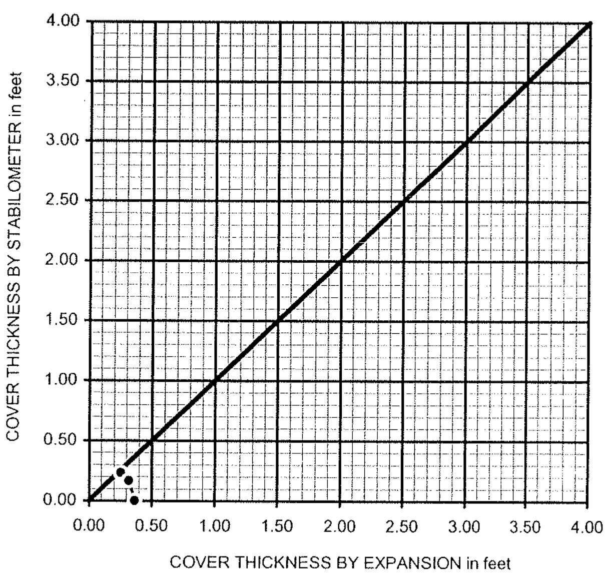
R-VALUE TEST RESULTS

PROJECT NAME:	Elizabeth LC	PROJECT NUMBER:	601506-001
SAMPLE NUMBER:	Bag-1	SAMPLE LOCATION:	B-2,0-5'
SAMPLE DESCRIPTION:	Sa.	TECHNICIAN:	SCF
		DATE COMPLETED	8/21/2006

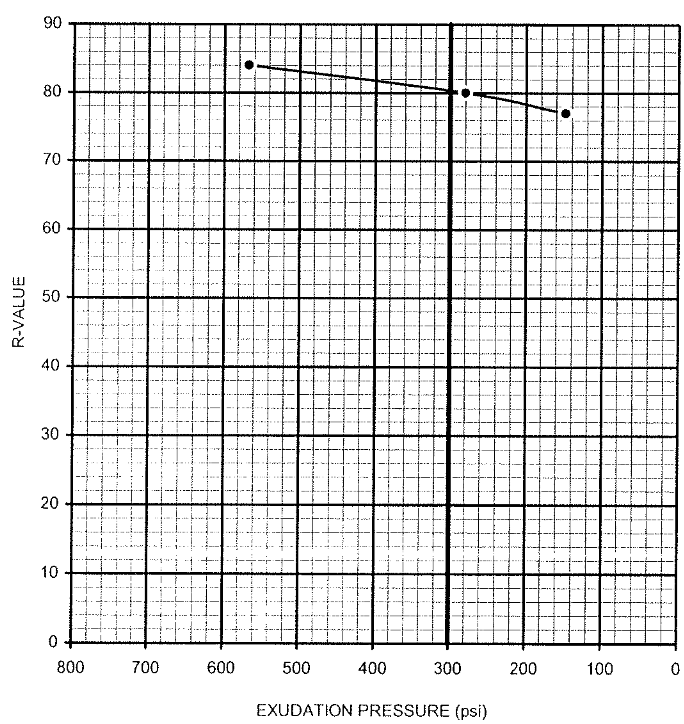
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	11.3	11.7	12.1
HEIGHT OF SAMPLE, Inches	2.46	2.51	2.49
DRY DENSITY, pcf	114.1	115.1	117.0
COMPACTOR PRESSURE, psi	275	240	200
EXUDATION PRESSURE, psi	567	280	148
EXPANSION, Inches x 10exp-4	7	5	0
STABILITY Ph 2,000 lbs (160 psi)	17	21	24
TURNS DISPLACEMENT	4.02	4.11	4.23
R-VALUE UNCORRECTED	84	80	77
R-VALUE CORRECTED	84	80	77

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.26	0.32	0.37
EXPANSION PRESSURE THICKNESS, ft.	0.23	0.17	0.00

EXPANSION PRESSURE CHART

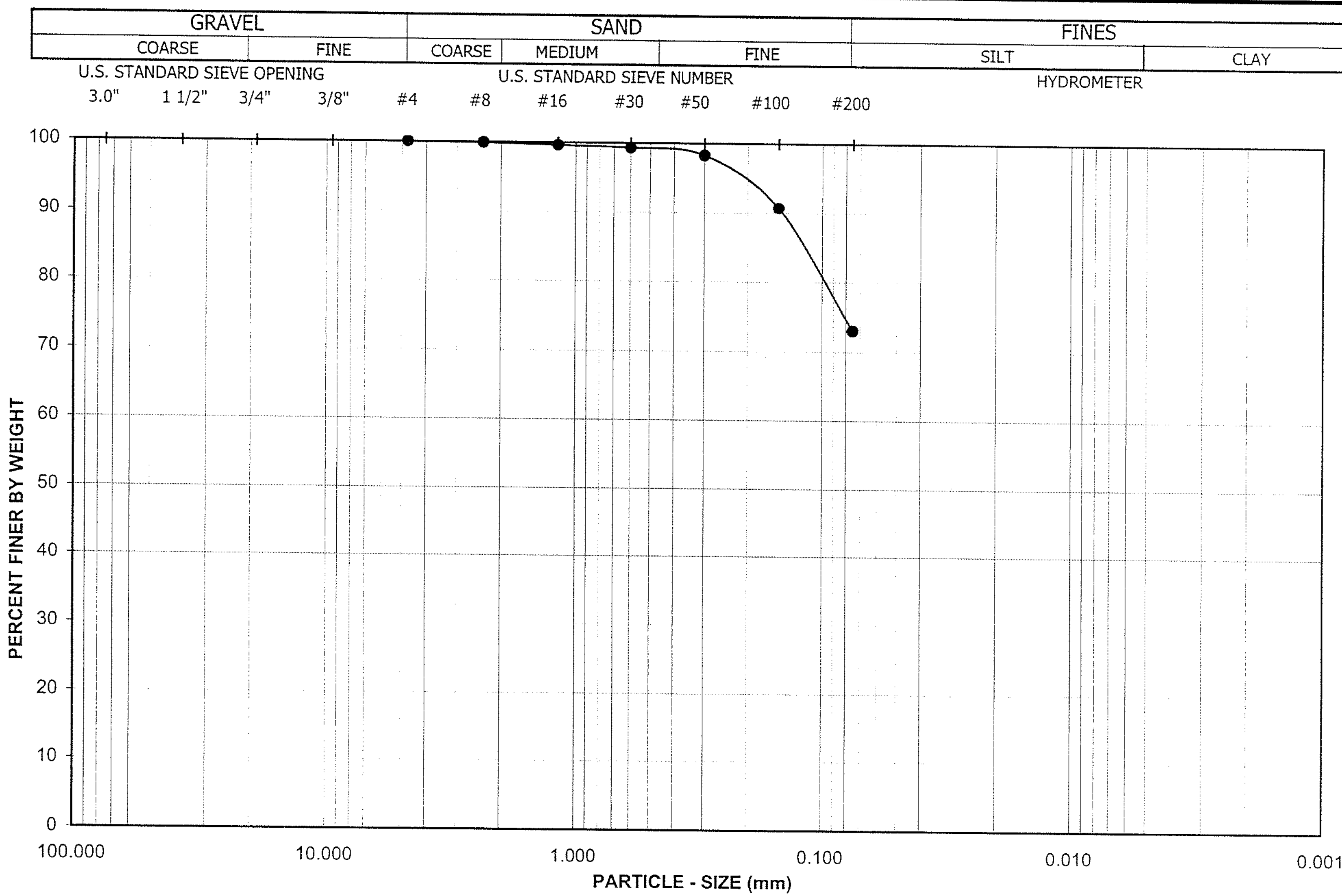


EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	85
R-VALUE BY EXUDATION:	80
EQUILIBRIUM R-VALUE:	80

Leighton 2007 Investigation



Project Name: Elizabeth Learning Center

Project No.: 601506-002

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

Exploration No.: B-1 Sample No.: S-3

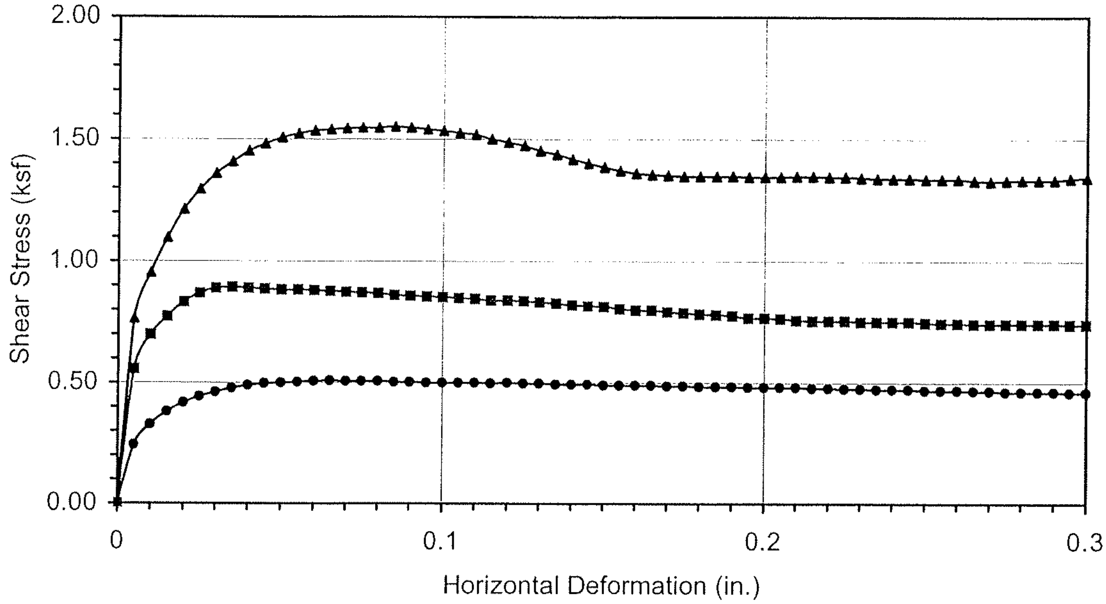
Depth (feet): 30.0 Soil Type : (ML)s

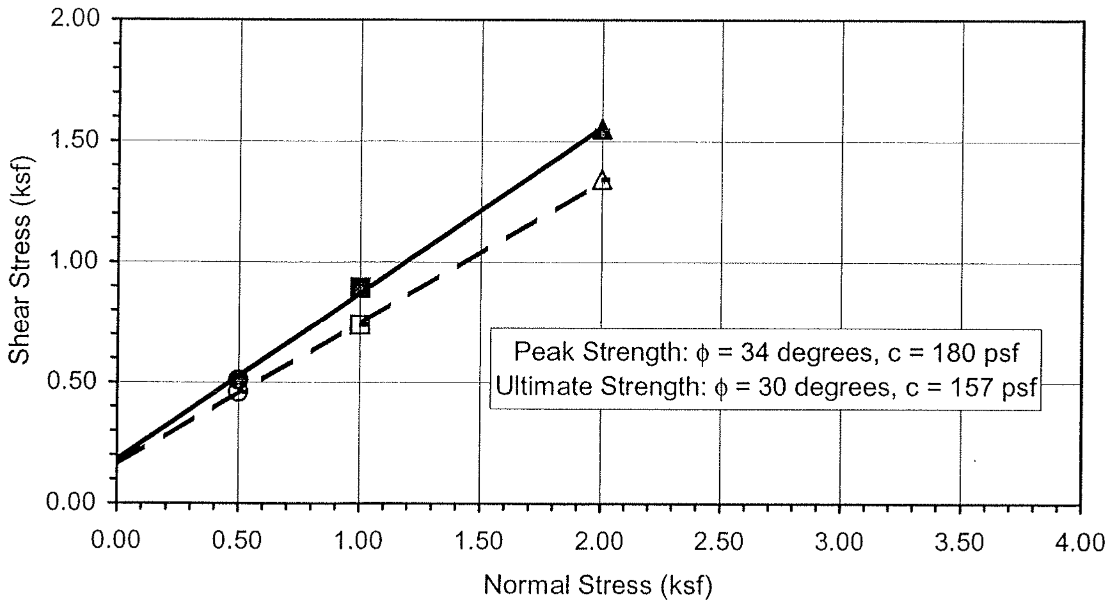
Soil Identification: Dark olive gray silt with sand (ML)s

GR:SA:FI : (%) **0 : 27 : 73**

Mar-07

SA B-1 S-3



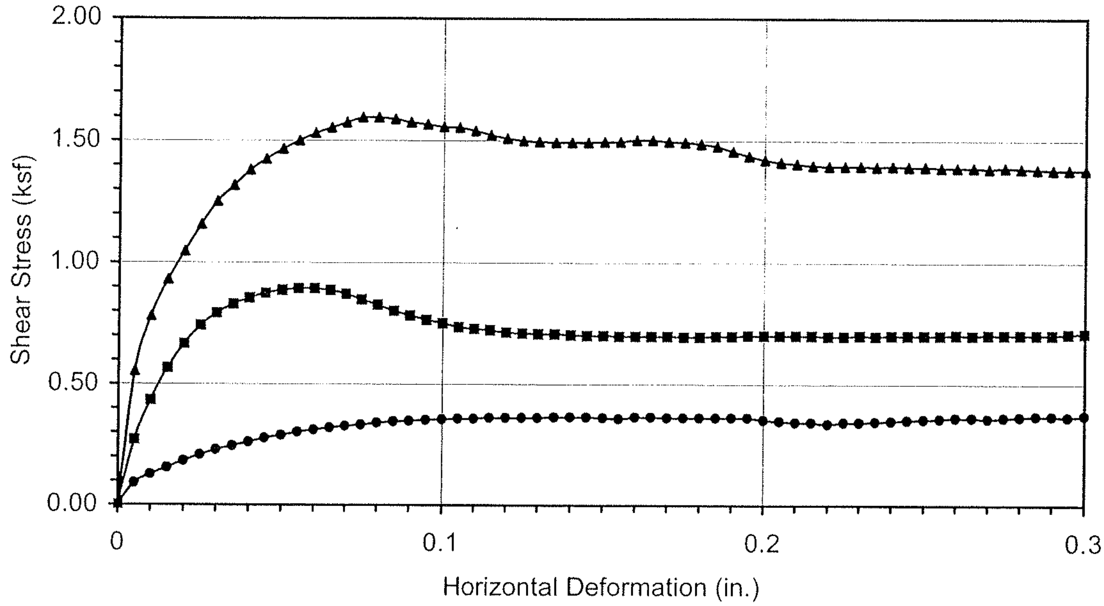


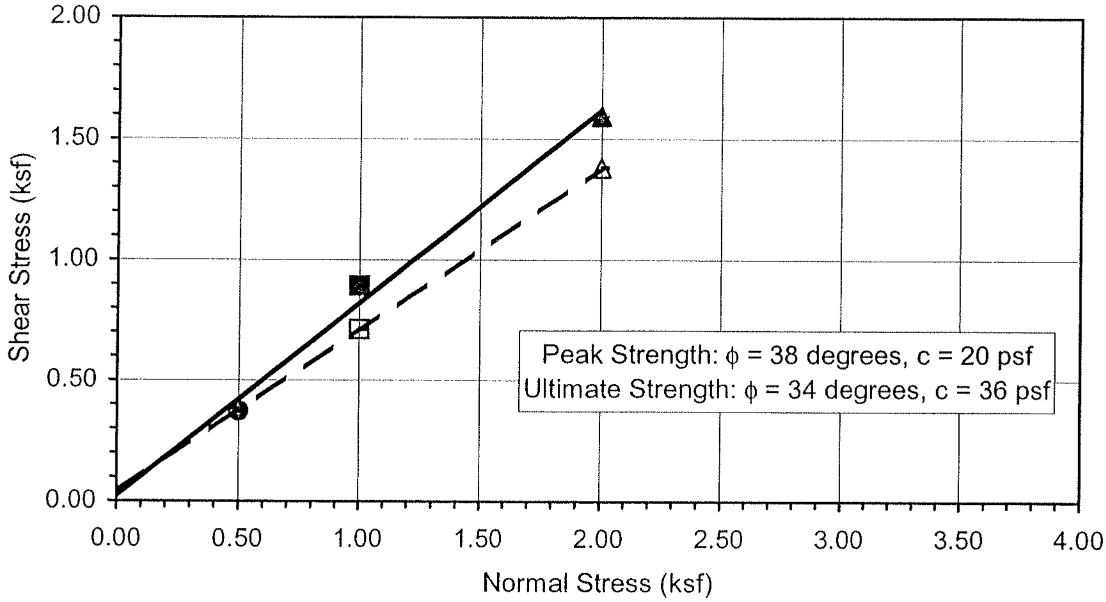
Boring No.	**B-1**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type: Drive	
Soil Identification: Olive brown silty sand (SM)	

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.509	■ 0.893	▲ 1.550
Shear Stress @ End of Test (ksf)	○ 0.459	□ 0.739	△ 1.342
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	4.36	4.36	4.36
Dry Density (pcf)	96.6	97.5	98.3
Saturation (%)	15.8	16.2	16.5
Soil Height Before Shearing (in.)	0.9959	0.9933	0.9888
Final Moisture Content (%)	23.5	23.2	23.1

Leighton	**DIRECT SHEAR TEST RESULTS** **Consolidated Undrained**	Project No.: 601506-002 Elizabeth Learning Center 02-07

DS B-1 R-1_.xls





Boring No.	**B-1**
Sample No.	**R-2**
Depth (ft)	**7.5**
Sample Type: Drive	
Soil Identification: Olive brown sandy silt s(ML)	

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.371	■ 0.893	▲ 1.594
Shear Stress @ End of Test (ksf)	○ 0.371	□ 0.710	△ 1.380
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.25	8.25	8.25
Dry Density (pcf)	96.3	107.9	112.5
Saturation (%)	29.7	39.6	44.7
Soil Height Before Shearing (in.)	0.9956	0.9956	0.9780
Final Moisture Content (%)	32.1	21.1	28.7

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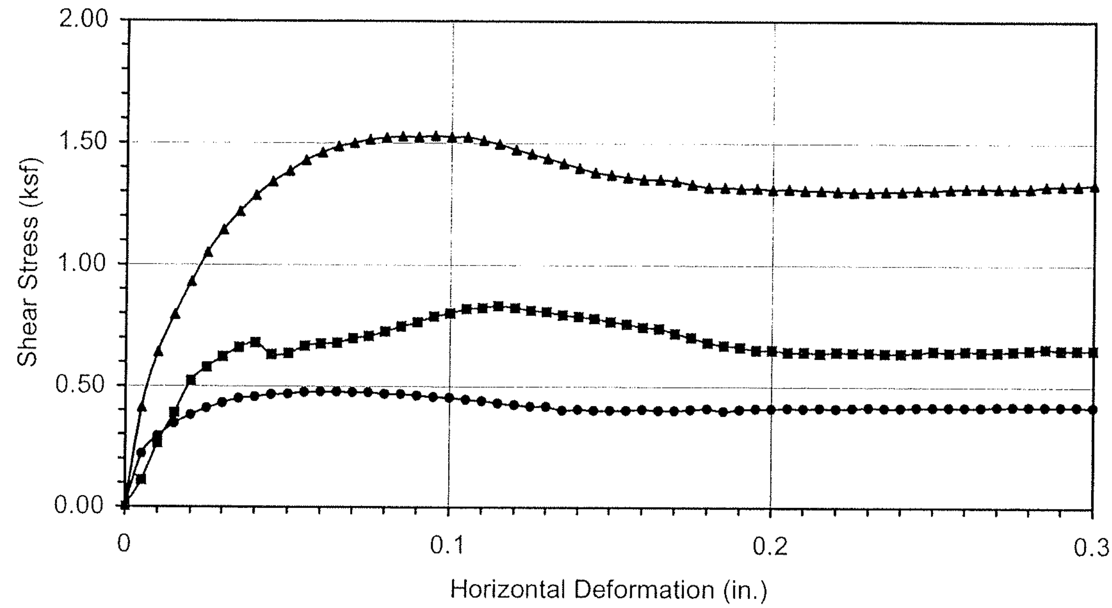
DIRECT SHEAR TEST RESULTS
Consolidated Undrained

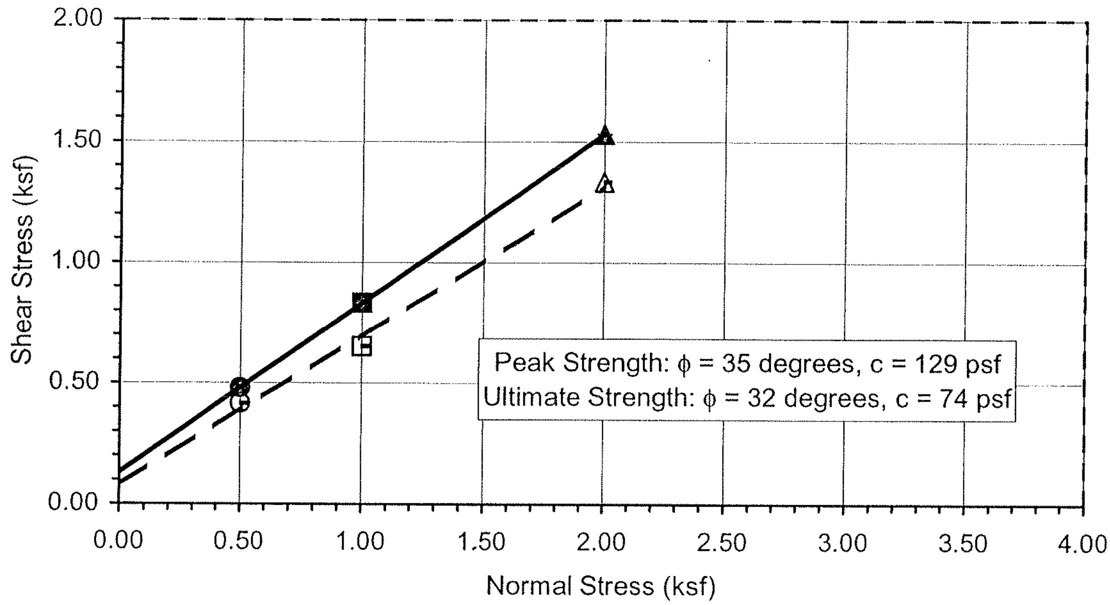
Project No.: 601506-002

Elizabeth Learning Center

02-07

DS B-1 R-2_.xls





Boring No.	**B-2**
Sample No.	**R-1**
Depth (ft)	**5**

Sample Type:

Drive

Soil Identification:

Grayish brown silty fine sand (SM)

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.478	■ 0.833	▲ 1.531
Shear Stress @ End of Test (ksf)	○ 0.415	□ 0.651	△ 1.333
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	1.99	1.99	1.99
Dry Density (pcf)	97.4	97.5	97.8
Saturation (%)	7.4	7.4	7.4
Soil Height Before Shearing (in.)	0.9955	0.9900	0.9902
Final Moisture Content (%)	24.5	23.8	23.9

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DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 601506-002

Elizabeth Learning Center

03-07

DS B-2 R-1_.xls

R-VALUE TEST RESULTS

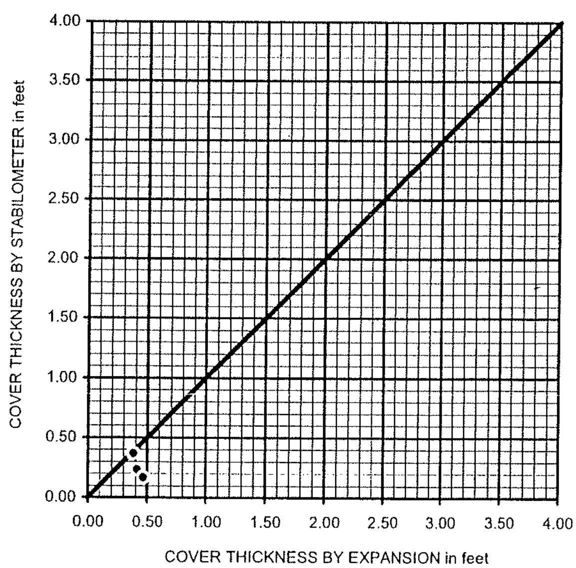
Leighton

PROJECT NAME:	Elizabeth Learning Center	PROJECT NUMBER:	601506-002
SAMPLE NUMBER:	Bag-1	SAMPLE LOCATION:	B-1 & B-2 @ 0-5' combined
SAMPLE DESCRIPTION:	SP	TECHNICIAN:	SCF
		DATE COMPLETED	3/1/2007

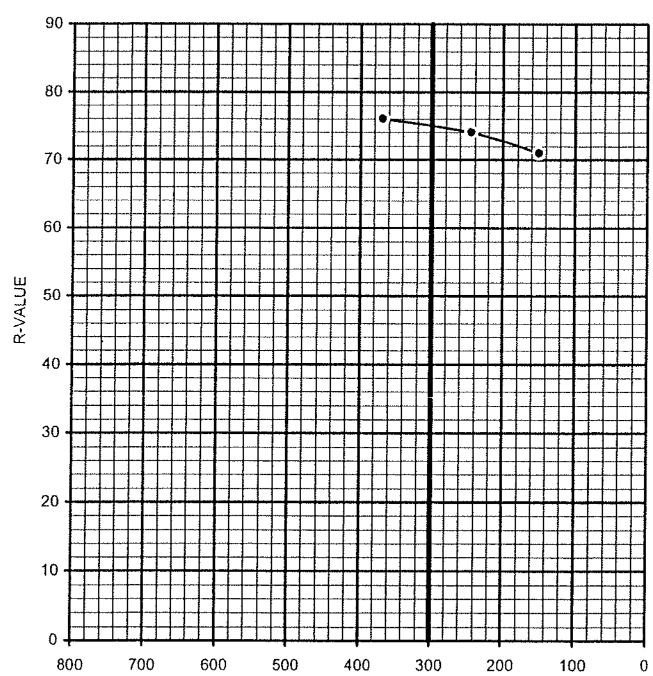
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	12.4	12.6	12.9
HEIGHT OF SAMPLE, Inches	2.50	2.49	2.57
DRY DENSITY, pcf	111.9	110.9	110.2
COMPACTOR PRESSURE, psi	250	225	200
EXUDATION PRESSURE, psi	368	245	150
EXPANSION, Inches x 10exp-4	11	7	5
STABILITY Ph 2,000 lbs (160 psi)	22	24	26
TURNS DISPLACEMENT	4.96	5.04	5.22
R-VALUE UNCORRECTED	76	74	71
R-VALUE CORRECTED	76	74	71

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.38	0.42	0.46
EXPANSION PRESSURE THICKNESS, ft.	0.37	0.23	0.17

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	76
R-VALUE BY EXUDATION:	75
EQUILIBRIUM R-VALUE:	75

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TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Elizabeth Learning Center | Tested By : VJ | Date: 02/23/07

Project No. : 601506-002 | Data Input By: LF | Date: 03/01/07

Boring No.	B-1 & B-2 combined			
Sample No.	Bag-1			
Sample Depth (ft)	0-5			
Soil Identification:	SP			
Wet Weight of Soil + Container (g)	186.83			
Dry Weight of Soil + Container (g)	179.87			
Weight of Container (g)	57.81			
Moisture Content (%)	5.70			
Weight of Soaked Soil (g)	100.67			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	1			
Crucible No.	3			
Furnace Temperature (°C)	830			
Time In / Time Out	8:10 / 8:55			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	18.5278			
Wt. of Crucible (g)	18.5248			
Wt. of Residue (g) (A)	0.0030			
PPM of Sulfate (A) x 41150	123.45			
PPM of Sulfate, Dry Weight Basis	**131**			

CHLORIDE CONTENT, DOT California Test 422

ml of Chloride Soln. For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.7			
PPM of Chloride (C -0.2) * 100 * 30 / B	50			
PPM of Chloride, Dry Wt. Basis	**53**			

pH TEST, DOT California Test 532/643

pH Value	**7.46**			
Temperature °C	19.8			

Leighton

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

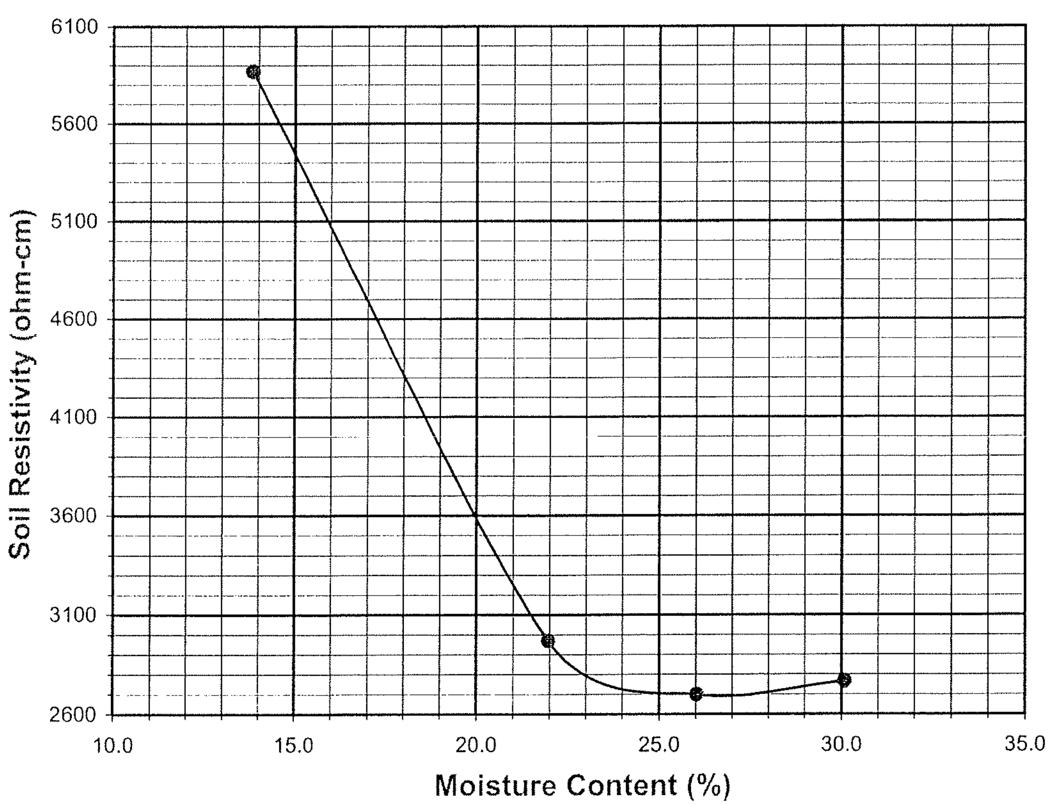
Project Name: Elizabeth Learning Center
Project No. : 601506-002
Boring No.: B-1 & B-2 combined
Sample No. : Bag-1
Soil Identification: SP

Tested By : VJ Date: 02/23/07
Data Input By: LF Date: 03/01/07
Depth (ft.) : 0-5

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	100	13.83	870	5869
2	200	21.96	440	2968
3	250	26.03	400	2698
4	300	30.09	410	2766
5				

Moisture Content (%) (MCi)	5.70
Wet Wt. of Soil + Cont. (g)	186.83
Dry Wt. of Soil + Cont. (g)	179.87
Wt. of Container (g)	57.81
Container No.	
Initial Soil Wt. (g) (Wt)	1300.00
Box Constant	6.746
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
2698	**26.0**	**131**	**53**	**7.46**	**19.8**



APPENDIX C – SITE-SPECIFIC SEISMIC HAZARD ANALYSIS – CBC 2016/ASCE 7-10

APPENDIX C

SITE-SPECIFIC SEISMIC HAZARD ANALYSIS

1.0 INTRODUCTION

This section presents the results of the site-specific seismic hazard analysis per the 2016 California Building Code (CBC) and ASCE 7-10 (ASCE/SEI 2013) for the proposed Elizabeth Learning Center Campus Modernization Program in Cudahy, California. The subsurface soil conditions used in this study were obtained from our field exploration program.

According to 2016 CBC and ASCE 7-10, ground motions are supposed to be developed for the Risk-Targeted Maximum Considered Earthquake (MCE_R) and the Design Earthquake. The site-specific MCE_R spectrum is calculated as the lesser of the probabilistic spectrum (two percent probability of exceedance in 50 years) and the deterministic spectrum. The MCE_R is associated with a risk associated with one percent probability of collapse in 50 years. It should be noted that the MCE_R should be based on the values in the maximum rotated direction. The deterministic spectrum is calculated as 84^{th}-percentile five percent damped spectral response acceleration in the direction of largest maximum horizontal response. As stipulated by ASCE 7-10 Section 21.3, the design response spectral accelerations are calculated as two-thirds of the MCE_R spectral accelerations except that the design spectral accelerations shall not be taken as less than 80 percent of spectral accelerations determined in accordance with ASCE 7-10 Section 11.4.5 using the mapped values of S_S and S_1. The 2008 USGS seismic sources developed for the seismic national zoning map were used in this study.

The steps involved in this section are outlined in the bullets below and presented in detail in the following sections.

- Site characterization to define Site Class per 2016 CBC and ASCE 7-10;
- Perform a site-specific probabilistic seismic hazard analysis for the Risk-Targeted Maximum Considered Earthquake (MCE_R) per ASCE 7-10 Section 21.2.1;
- Perform a site-specific deterministic seismic hazard analysis for the MCE_R per ASCE 7-10 Section 21.2.2;
- Develop the site-specific Risk-Targeted MCE_R, which is the lesser of the spectral accelerations from the probabilistic MCE_R and deterministic MCE_R (ASCE 7-10 Section 21.2.3);

- Develop the design response spectrum and design acceleration parameters in accordance with ASCE 7-10 Section 21.3 and Section 21.4 respectively;
- Calculate the site-specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration; and,

2.0 PROJECT LOCATION

The site is located in Cudahy, California. The address and site coordinates are:

Address:	4811 Elizabeth Street Cudahy, California
Latitude:	33.9635° N
Longitude:	118.18305° W

3.0 SITE CHARACTERIZATION

In developing site-specific ground motions, the characteristics of the soils underlying the site are an important input to evaluate the site response at a given site. Based on the field exploration we performed, the site is classified as Site Class D as presented in Table 20.3-1, ASCE 7-10 and Chapter 20 of ASCE 7-10. Site Class D is defined as stiff soil with average shear wave velocities (V_S) between 600 ft/s (about 183 m/s) and 1,200 ft/s (about 366 m/s), average standard penetration resistance (N) between 15 and 50, or average undrained shear strength (S_U) between 1,000 psf and 2,000 psf for the upper 100 feet (about 30 meters). We assumed a V_{s30} value of 270 m/s for this site. For our site-specific analyses, we used Site Class D. These assumptions were deemed appropriate by using correlations of V_s with SPT blowcounts (Brandenburg, Bellana, and Shantz (2010) and Dickenson (1994)) and cone penetration test data (Robertson (2009)) and approximating V_{s30} using our explorations shown in Figure 2.

4.0 GROUND MOTION PREDICTION EQUATIONS

Site-specific ground motions can be influenced by the types of faulting, magnitudes of the earthquakes, and local soil conditions. Ground Motion Prediction Equations (GMPE) account for these effects and are used to make estimates of ground motion at a site resulting from a scenario earthquake.

Many GMPEs have been developed to estimate the variation of spectral acceleration with earthquake magnitude and distance from the site to the source of an earthquake. Under a

Pacific Earthquake Engineering Research (PEER) Center project entitled "Next Generation Attenuation of Ground Motions (NGA)," five teams have developed and presented GMPEs for shallow crustal earthquakes in Western North America. These relationships are: Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Chiou and Youngs (2008), and Idriss (2008).

The NGA GMPEs were developed from statistical analyses of recorded worldwide earthquakes, including the records from the 1989 Loma Prieta earthquake, the 1992 Landers earthquake, the 1994 Northridge earthquake, the 1995 Kobe earthquake, and more recent important earthquakes that were not included in the 1997 relationships like the 1999 Kocaeli (Turkey) earthquake and the 1999 Chi-Chi (Taiwan) earthquake. The NGA GMPEs provide geomean (GMRotI50) values of ground motions. To account for the direction of largest maximum horizontal response we used the method by Whittaker (2009).

We have not used Idriss (2008) as this GMPE is only applicable to V_{S30} > 450 m/s. For this project we used the models listed in the Table C-1 below.

TABLE C-1

GMPES USED IN THE SEISMIC HAZARD ANALYSIS

GMPE	Seismic Source
Boore and Atkinson (2008)	Fault/Background
Campbell and Bozorgnia (2008)	Fault/Background
Chiou and Youngs (2008)	Fault/Background

Some of the GMPEs require input for $Z_{1.0}$ (defined as the depth in meters to a layer with V_s = 1,000 m/s) and $Z_{2.5}$ (depth in km to a layer with V_s= 2,500 m/s). These two parameters intend to capture the basin effect on site response. We have used $Z_{1.0}$= 800m and $Z_{2.5}$=5.22 km. The depth to bedrock (Z1.0 and Z2.5) was calculated using Caltrans ARS online tools. For sites in southern California, the online tool utilizes data from the Community Velocity Model (CVM) Version 4 (http://scec.usc.edu/scecpedia/Community_Velocity_Model).

5.0 PROBABILISTIC SEISMIC HAZARD ANALYSIS

We have developed a response spectrum for the probability of exceedance of 2% in 50 years (return period of about 2,475 years) using a probabilistic seismic hazard analysis (PSHA). The PSHA analysis involves the selection of appropriate GMPEs to estimate the ground motion parameters, and through probabilistic methods, determination of spectral accelerations.

5.1 Probabilistic Analysis

The theory behind this analysis has been developed over many years (Cornell 1968, 1971, Merz and Cornell 1973, McGuire 2004) and is based on the "total probability theorem" and on the assumption that earthquakes are events that are independent of time and space from one another. According to this approach and assuming a Poisson process for ground motion occurrences, the probability of an event, *P*, is related to the annual frequency of exceedance of the ground motion γ and the exposure time *t* through

$$P = 1 - \exp(-\gamma\ t)$$

One earthquake hazard level, associated with the MCE_R, is defined to have a two percent probability of exceedance in 50 years, which corresponds to an exposure time or return period of about 2,475 years and an annual frequency of exceedance of 0.00040/year.

The PSHA can be explained through a four-step procedure as follows.

1. The first step involves identification and characterization of seismic sources and probability distribution of potential rupture within the source. Usually, uniform probability distributions are assigned to each source. The probability distribution of site distance is obtained by combining potential rupture distributions with source geometry.
2. The second step involves characterization of seismicity distribution of earthquake recurrence. An earthquake recurrence relationship such as Gutenberg-Richter recurrence is used to characterize the seismicity of each source.
3. The third step involves the use of GMPEs in assessing the ground motion produced at the site by considering the applicable sources and the distance of the sources to site. The variability of the GMPEs is also included in the analysis. The effects of site soil conditions and mechanism of faulting are accounted for in these GMPEs.

4. The fourth and the last step involve combining all of these uncertainties to obtain the probability of ground motion exceedance during a particular time period.

We used the commercially available computer program EZ-FRISK Version 7.65 (Risk Engineering, 2015) for our analysis.

5.2 Probabilistic Response Spectrum

The site-specific probabilistic response spectrum MCE_R for this project was developed based on a uniform-hazard approach. The uniform hazard approach assumes that the same level of hazard is uniformly applied to the entire response spectrum. Response spectral values for the MCE_R in the direction of maximum horizontal response were represented by damping factor five percent of critical that are expected to achieve a one percent probability of collapse within a 50-year period.

The probabilistic MCE_R spectrum was defined as the product of the risk coefficient, C_R, and the spectral response acceleration from a five percent damped acceleration response spectrum having a two percent probability of exceedance within a 50-year period (Method 1, Section 21.2.1.1, ASCE 7-10). C_R may take different values depending on spectral periods, i.e., C_{RS} for periods less than or equal to 0.2 second, C_{R1} for periods greater than or equal to 1.0 second, and linear interpolation between C_{RS} and C_{R1} for periods between 0.2 and 1.0 second. The values of the risk coefficients C_{RS} and C_{R1} were obtained from the USGS website http://geohazards.usgs.gov/designmaps/us/application.php These values were found to be C_{RS}=0.968 and C_{R1}=0.991 respectively. The MCE_R probabilistic response spectrum is presented on Plate C-1.

6.0 DETERMINISTIC SEISMIC HAZARD ANALYSIS

Deterministic seismic hazard analysis (DSHA) is based on the characteristics of the earthquake and of the causative fault associated with the earthquake. These characteristics include such items as distance from the site to the causative fault and maximum magnitude of earthquake associated with that fault. The effects of local soil conditions and mechanism of faulting are accounted for in the GMPEs for the project site.

The DSHA can be explained through a four-step procedure as follows.

1. The first step involves identification and characterization of all seismic sources capable of producing significant ground motions at the site.

2. The second step involves estimating maximum magnitude of earthquake associated with the known seismic sources and establishing site to source distance. The distance may be expressed as closest distance to fault rupture plane (R_{RUP}), Joyner-Boore distance (R_{JB}) or Horizontal distance to the fault trace or surface projection of the top of rupture plane (R_X) depending on the GMPE.
3. The third step involves determining the *controlling earthquake(s)* and use of GMPEs in determining the ground motion produced at the site by considering the size of the earthquake occurring at the source and the distance of the source to site. The effects of the soil conditions and mechanism of faulting are accounted for in these GMPEs.
4. The fourth and last step involves formally defining the hazard in terms of spectral accelerations.

Deterministic procedure was used to estimate the 84th percentile five percent damped spectral response acceleration in the direction of maximum horizontal response at every spectral period. The largest such acceleration calculated for the characteristic earthquakes on all known active faults within the region was used. In calculating the spectral accelerations, we used the same GMPEs as in our PSHA.

The deterministic response acceleration spectrum should not be lower than the Deterministic Lower Limit (DLL) on MCE_R Response Spectrum presented on Figure 21.2.1, ASCE 7-10. Plate C-2 presents the Deterministic MCE_R Response Spectrum and the DLL for the project site.

7.0 DETERMINATION OF SITE-SPECIFIC MCE_R RESPONSE SPECTRUM

The site-specific MCE_R response spectrum was defined according to Section 21.2.3, ASCE 7-10 as the lesser spectral accelerations from the probabilistic or deterministic response spectrum. The MCE_R response spectrum for this site is presented in Plate C-3.

8.0 DETERMINATION OF SITE-SPECIFIC DESIGN RESPONSE SPECTRA

The site-specific design response spectrum (DE) was determined according to Section 21.3, ASCE 7-10, as the two thirds of the values of the spectral accelerations calculated for the site-specific MCE_R response spectrum. As per ASCE 7-10, the design spectrum is greater than the 80% of the spectral amplitudes of the general map based design response spectrum except for periods between 0.05 and 0.125 second. For this range of periods the 80% of the spectral amplitudes of the general map based design response spectrum governs. The mapped or code-based spectra were determined using the USGS website:

http://geohazards.usgs.gov/designmaps/us/application.php.

The design response spectrum determination is presented in Plate C-4. The MCE_R and DE response spectra in digitized form is presented in Table C-2 below.

TABLE C-2. SITE-SPECIFIC HORIZONTAL RESPONSE SPECTRA

Period (s)	MCE_R Sa(g)	DE Sa (g)
0.01	0.82	0.54
0.05	0.98	0.72
0.06	1.07	0.78
0.08	1.21	0.90
0.1	1.34	1.02
0.125	1.48	1.06
0.15	1.60	1.06
0.2	1.71	1.14
0.25	1.76	1.18
0.3	1.81	1.20
0.4	1.79	1.19
0.5	1.85	1.23
0.75	1.66	1.11
1	1.47	0.98
1.5	1.07	0.71
2	0.81	0.54
2.5	0.63	0.42
3	0.52	0.35
3.5	0.44	0.29
4	0.39	0.26

9.0 SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

The short period design spectral acceleration (S_{DS}) and 1-second period design spectral acceleration (S_{D1}) parameters were determined in accordance with ASCE 7-10 Section 21.4. The parameter S_{DS} is taken as the spectral acceleration at a period of 0.2 seconds or 90 percent of the highest spectral acceleration at periods larger than 0.2 seconds, whichever is greater. The parameter S_{D1} is taken as the design spectral acceleration at a period of 1 second or two times the spectral acceleration at the 2 second period, whichever is greater. The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1} respectively. The values so obtained shall not be less than 80 percent of the values determined in accordance with ASCE 7-10, Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.4 for S_{DS} and S_{D1}. Table C-3 presents the site-specific design acceleration parameters.

TABLE C-3. SEISMIC DESIGN PARAMETERS

Design Parameters	General Seismic Design Parameter (ASCE 7-10 Sectopm 11.4)	Site-Specific Seismic Desgin Parameters (ASCE 7-10 Section 21.4)
S_s (g)	1.98	-
S_1 (g)	0.697	-
Site Class	D	D
F_a	1.0	-
F_v	1.5	-
S_{MS} (g)	1.98	1.71
S_{M1} (g)	1.05	1.62
S_{DS} (g)	1.32	1.14
S_{D1} (g)	0.70	1.08

10.0 REFERENCES

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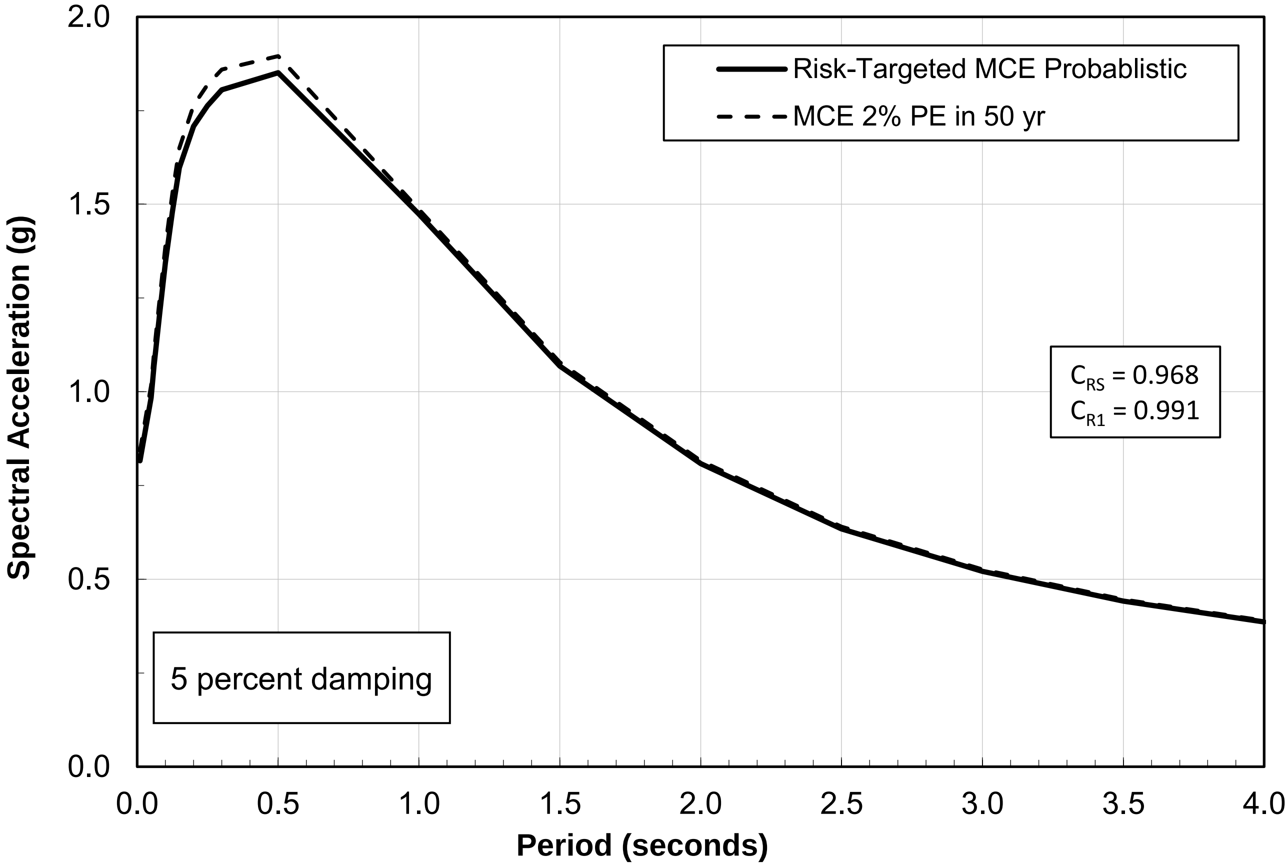
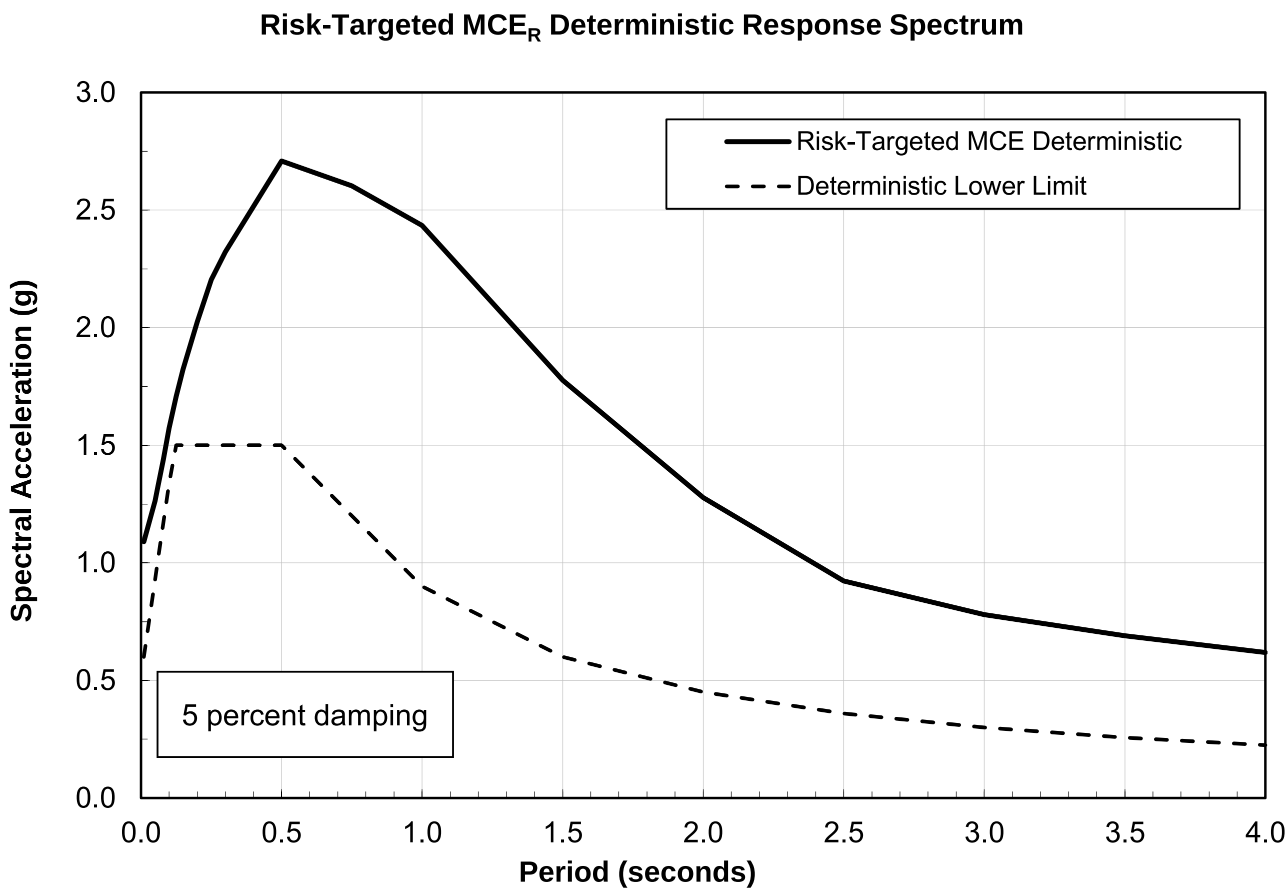
PLATES

Risk-Targeted MCE_R Probablisitic Response Spectrum



		MCE_R Probabilistic Response Spectrum Site Class D	PLATE
By: CW	Reviewed by: ET	LAUSD ELIZABETH LC	C-1
Project # LA-1321	4/27/2017		



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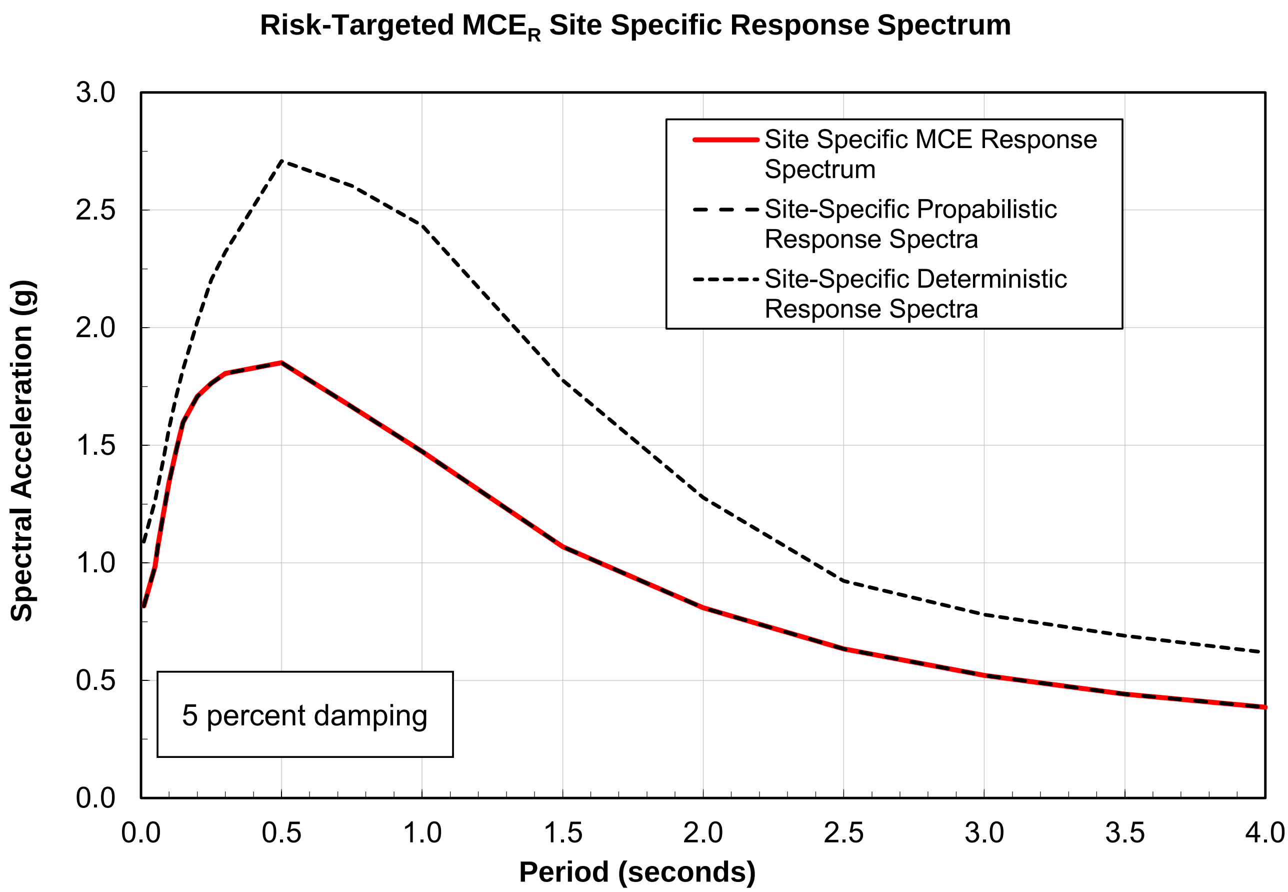
Project # LA-1321	4/27/2017		



		Site-Specific MCE$_G$ Response Spectrum Site Class D	PLATE
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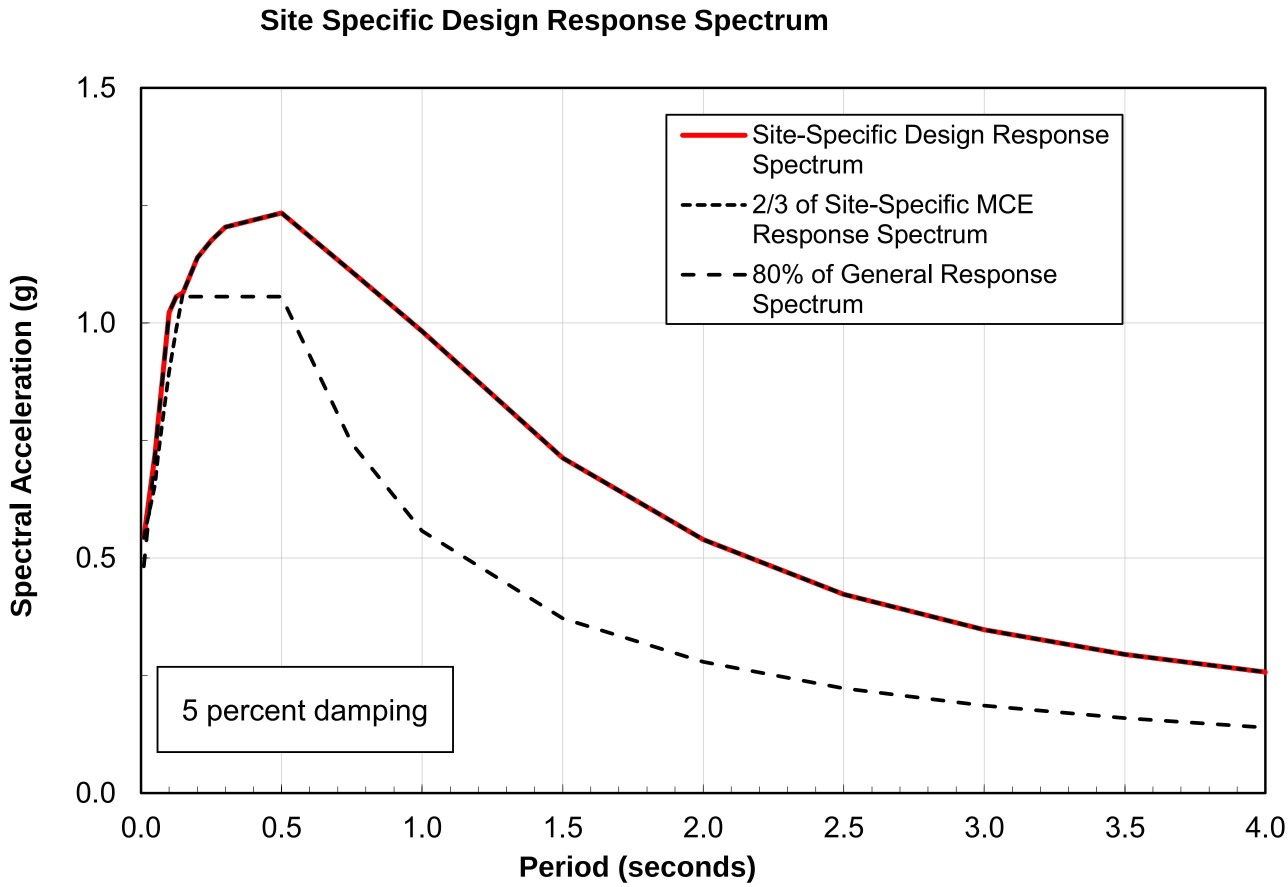
Project # LA-1321	4/27/2017		



		Site-Specific Design Response Spectrum Site Class D	**PLATE**
By: CW	Reviewed by: ET	**LAUSD ELIZABETH LC**	**C-4**
Project # LA-1321	4/27/2017		

APPENDIX D – CLIQ LIQUEFACTION ANALYSES

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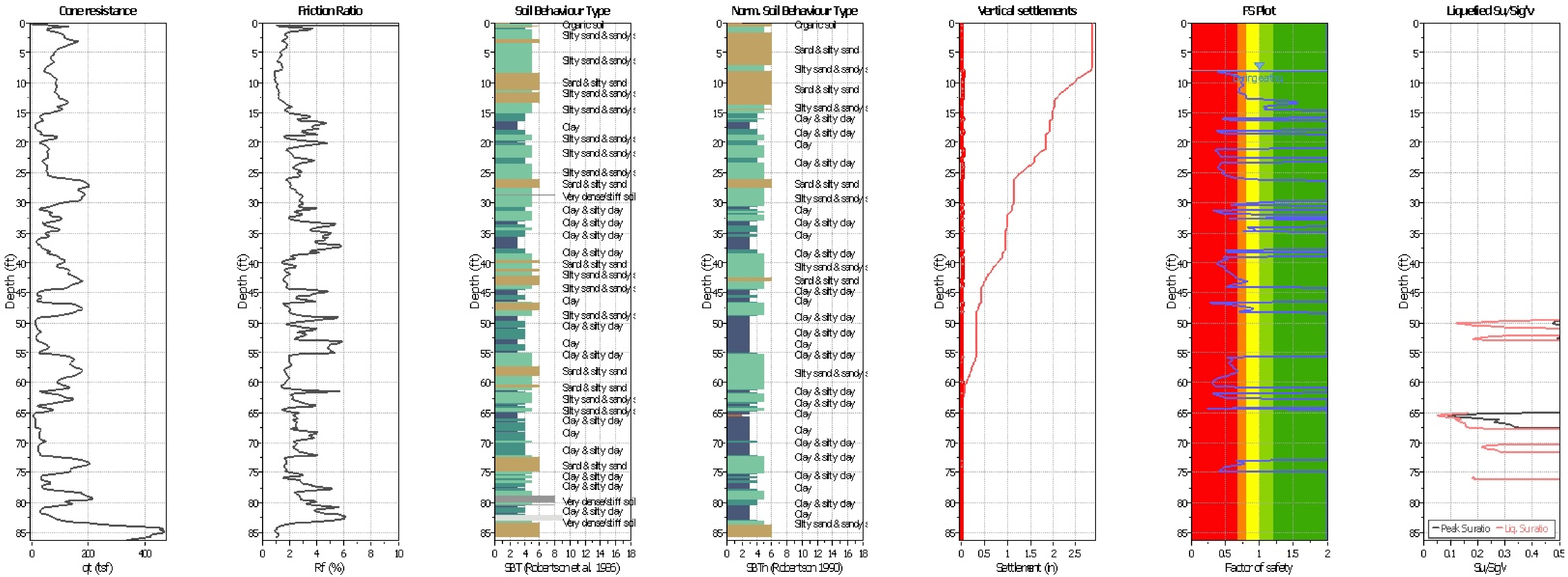
Project: **LA1321 Elizabeth LC**
Location: **4811 Elizabeth Street, Cudahy, CA**

CPT: CPT-02
Total depth: 86.29 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	75.00 ft
Earthquake magnitude M_w:	6.66	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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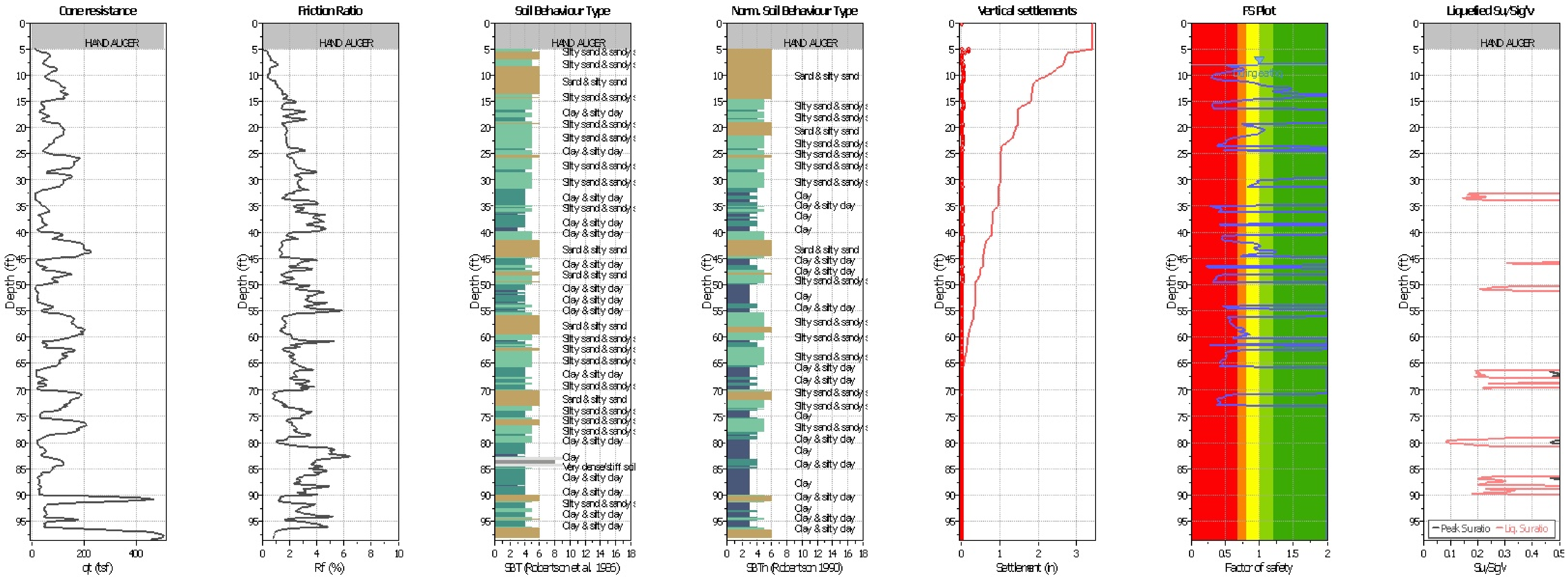
Project: **LA1321 Elizabeth LC** | **CPT: CPT-04**

Location: **4811 Elizabeth Street, Cudahy, CA** | Total depth: 98.75 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	75.00 ft
Earthquake magnitude M_w:	6.66	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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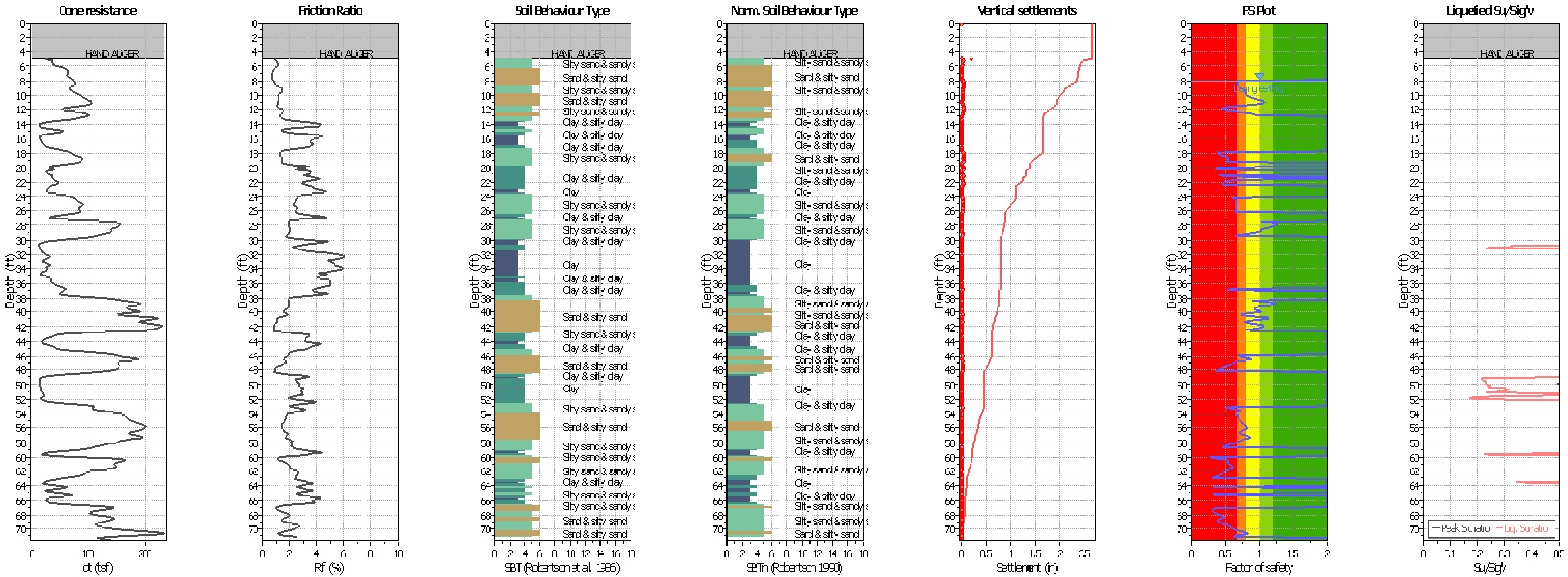
Project: LA1321 Elizabeth LC
Location: 4811 Elizabeth Street, Cudahy, CA

CPT: CPT-05
Total depth: 71.52 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	75.00 ft
Earthquake magnitude M_w:	6.66	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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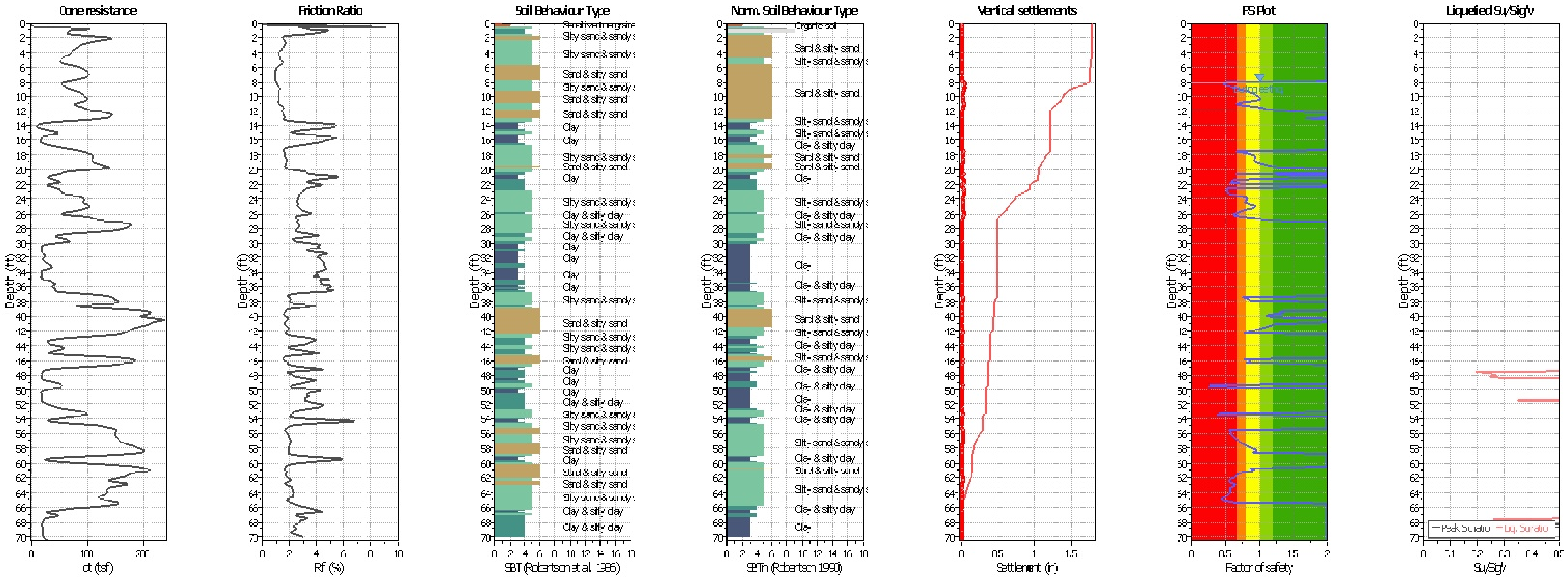
Project: LA1321 Elizabeth LC
Location: 4811 Elizabeth Street, Cudahy, CA

CPT: CPT-07
Total depth: 70.54 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	75.00 ft
Earthquake magnitude M_w:	6.66	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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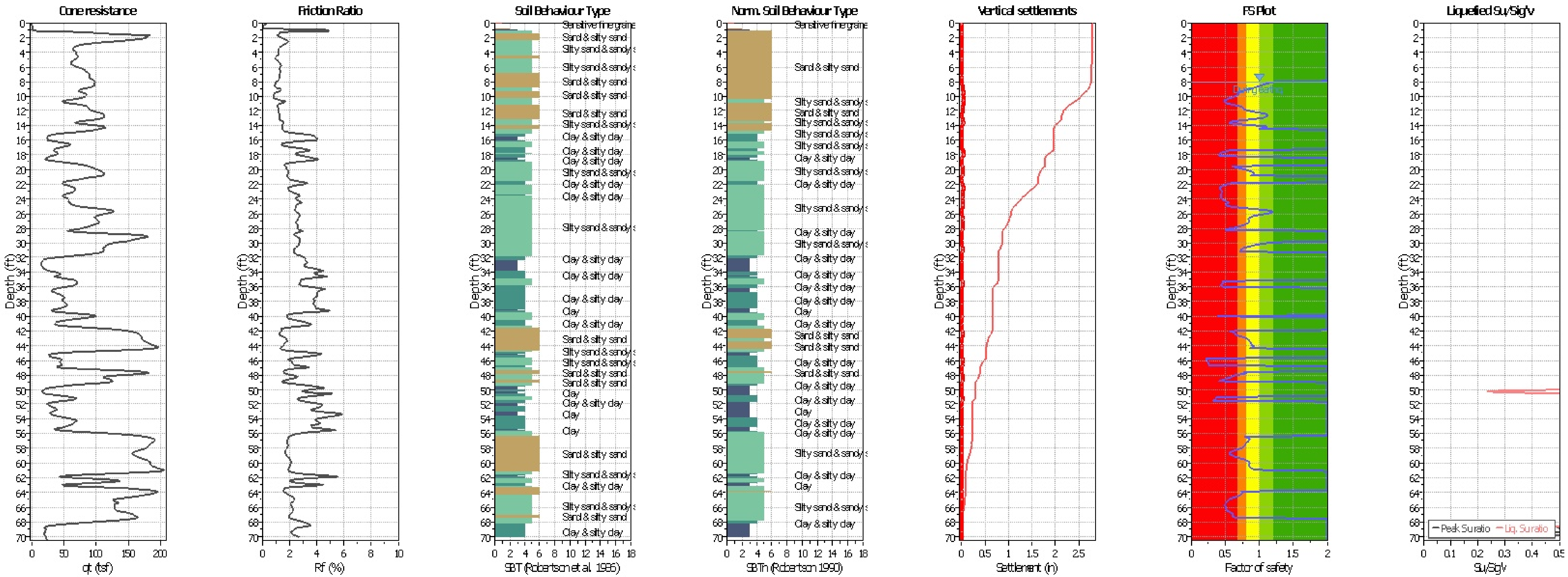
Project: **LA1321 Elizabeth LC** | **CPT: CPT-09**
Location: **4811 Elizabeth Street, Cudahy, CA** | Total depth: 70.54 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	75.00 ft
Earthquake magnitude M_w:	6.66	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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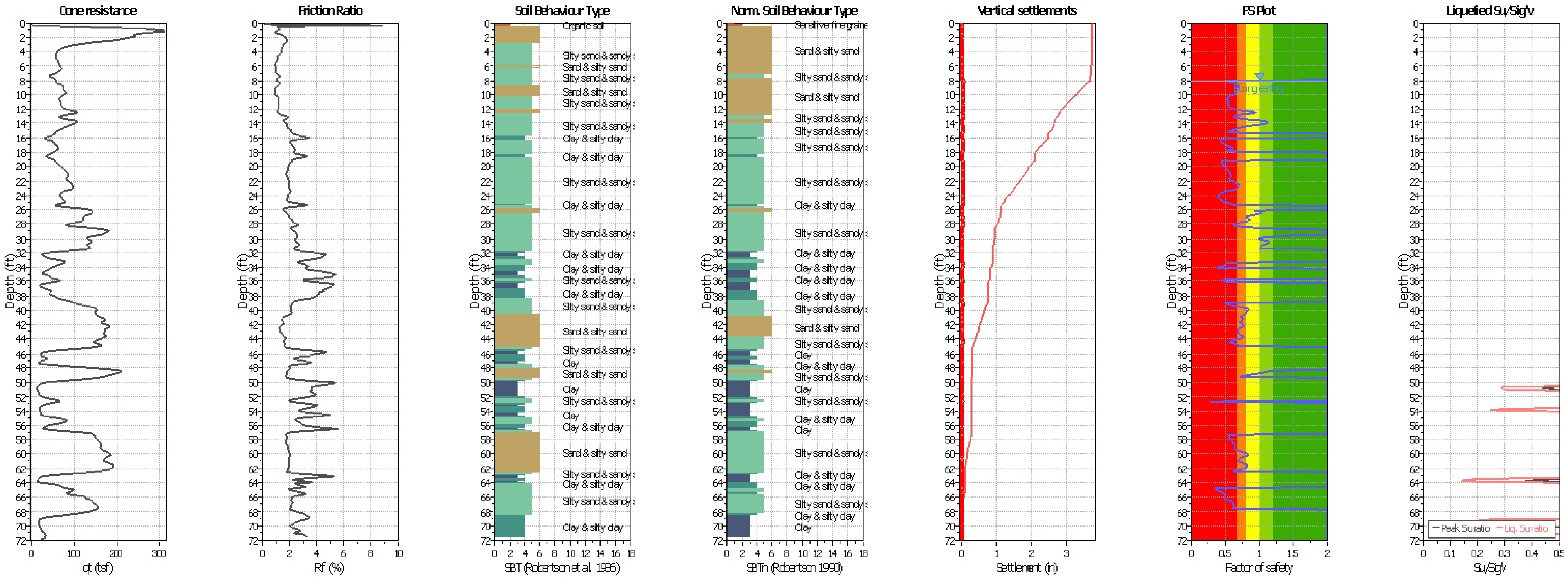
Project: LA1321 Elizabeth LC

Location: 4811 Elizabeth Street, Cudahy, CA

CPT: CPT-10

Total depth: 72.01 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	75.00 ft
Earthquake magnitude M_w:	6.66	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		