REPORT OF GEOTECHNICAL INVESTIGATION AVENIDA DE LA PLAYA STORM DRAIN AND WATER AND SEWER GROUP 809

Prepared For:

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June 11, 2010

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Subject: **REPORT OF GEOTECHNICAL INVESTIGATION**
AVENIDA DE LA PLAYA STORM DRAIN AND
WATER AND SEWER GROUP JOB 809
CITY OF SAN DIEGO
AGE Project No. 119 GTS-08 (44A408)

Dear Brad:

Allied Geotechnical Engineers, Inc. is pleased to submit the accompanying report to present the findings, opinions, and recommendations of a geotechnical investigation that was performed for final design of the above-mentioned subject project.

We appreciate the opportunity to be of service on this interesting project. If you have any questions regarding the contents of this report or need further assistance, please give us a call.

Sincerely,

ALLIED GEOTECHNICAL ENGINEERS, INC.

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AVENIDA DE LA PLAYA STORM DRAIN AND WATER AND SEWER GROUP JOB 809 CITY OF SAN DIEGO

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1.0 INTRODUCTION

Allied Geotechnical Engineers, Inc. is pleased to submit this report to present the findings, opinions, and recommendations of a geotechnical investigation conducted for the design of the City of San Diego(City) Avenida De La Playa Storm Drain and Water and Sewer Group Job 809 project. The investigation was performed in general conformance with the authorized scope of work as outlined in the Subconsultant Agreement between Tetra Tech, Inc. (Tetra Tech) and Allied Geotechnical Engineers, Inc. (AGE), dated May 18, 2010.

This report has been prepared for the exclusive use of Tetra Tech and the City in their design of the project as described herein. The information presented in this report is not sufficient for any other uses or the purposes of other parties.

2.0 PROJECT DESCRIPTION

It is our understanding that the City of San Diego (City) plans to design and construct a new, approximately 1,300 feet long double box storm drain culvert along a portion of Avenida De La Playa, between La Vereda and Paseo Del Ocaso in La Jolla, California (see Vicinity Map, Figure 1). We further understand that the scope of this project also includes Water and Sewer Group Job 809, which involves the rehabilitation and realignment of approximately 1,768 linear feet of sewer line and approximately 150 linear feet of water line.

A copy of the preliminary plans were provided to us for review by Tetra Tech. Based on a review of these plans, it is our understanding that the new storm drain culvert will be installed at an average invert depth of about 6 feet below existing grade (bgs). The plans further indicate that the invert depth of the sewer line ranges from 3 feet to 6 feet bgs, and that the replacement water pipeline will be installed at an average invert depth of 3 to 4 feet bgs. The plans also show an approximately 600 feet long sewer line that is located along a private access driveway on the La Jolla Beach and Tennis Club property. This sewer line, however, is no longer included in the scope of this project. We further understand that the new storm drain culvert, and sewer and water pipelines will be constructed using conventional open cut methods.

Based on our understanding of the scope of the proposed project, AGE has conducted a geotechnical investigation which includes the performance of a geotechnical field exploration and laboratory testing program.

3.0 OBJECTIVE AND SCOPE OF INVESTIGATION

The objectives of this investigation were to characterize the subsurface conditions along the project alignments and to develop geotechnical recommendations for use in the design of the project as currently proposed. The scope of our investigation included several tasks which are described in more detail in the following sections.

3.1 Information Review

This task involved a review of readily available information pertaining to the project site, including published geologic literature and maps, topographic maps, and the preliminary plans.

3.2 Geotechnical Field Exploration

The field exploration program for this project was performed on May 27, 2010 and included the advancement of three Cone Penetrometer Test (CPT) soundings at the approximate locations shown on the Site Plan (Figure 1). The CPT soundings were advanced to a depth of 20 feet below existing ground surface (bgs) using a 30-ton CPT rig. A more detailed description of the CPT operations and the CPT logs are presented in Appendix A. In addition, AGE also collected discrete soil samples at approximately 5-foot interval using a 6600 Geoprobe rig.

Prior to commencement of the field exploration activities, a site reconnaissance visit was performed to observe existing conditions and to select suitable locations for the CPT Soundings. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed CPT locations with respect to existing buried utilities. In addition, boring permit waivers were obtained from the County of San Diego Department of Environmental Health Services. CASS Construction, Inc. (CASS) was retained to assist with obtaining encroachment and traffic control permits from the City of San Diego.

A more detailed description of the field exploration activities and logs of the CPT Soundings and push samples are presented in Appendix A.

3.3 Laboratory Testing

Selected soil samples obtained from the push probes were tested in the laboratory to verify field classifications and evaluate certain engineering characteristics. The geotechnical laboratory tests were performed in general conformance with the American Society for Testing and Materials (ASTM) or other generally accepted testing procedures.

The laboratory tests performed for this investigation included: in-place moisture content and unit dry weight, and sieve analysis. A description of the tests that were performed and the final test results are presented in Appendix B.

3.4 Archeological Survey

Tetra Tech retained the services of RECON Environmental, Inc. (RECON) to perform an archeological survey for this project. At the request of Tetra Tech, AGE provided assistance to RECON to facilitate the performance of their archaelogical survey which included the excavation of two test pits on Avenida De La Playa. AGE's services included: assisting with obtaining City of San Diego encroachment and traffic control permits; coordination with USA for utility clearance; providing traffic control measures, steel plates and shoring; backfilling of the test pit excavations with compacted import soil; and repair of the disturbed asphalt-concrete pavement.

4.0 GENERAL SITE CONDITIONS

The project alignments are located within the right-of-way of Avenida De La Playa at approximate Latitude 32°51'14.79"N and Longitude 117°15'25.24"W. Ground elevations vary from 0 feet to + 14 feet above Mean Sea Level Datum (MSL). The surrounding areas are mostly developed with commercial facilities along Avenida De La Playa and single family residences along the adjacent streets.

Based on the information gathered from our utility clearance efforts, we understand that buried utilities within the limits of the project area include storm drain, water supply, sanitary sewer, and natural gas pipelines, as well as communication, cable, and electrical conduits.

5.0 SUBSURFACE CONDITIONS

5.1 Geologic Units

Based on their origin and compositional characteristics, the soil types encountered in the exploratory borings can be categorized into three geologic units. A brief description of each unit (in order of increasing age) is presented below.

5.1.1 Fill Materials

Fill materials were encountered to an approximate depth of 5 feet bgs in CPT-1 through CPT-3. The fill materials encountered in these borings consist of a silty, fine to medium coarse sand with occasional small rocks.

Placement of the fill materials appear to be associated with roadway construction and/or filling of former low-lying areas. Fill materials can be expected to occur along other portions of the project alignment and may be anticipated to vary greatly in extent (both laterally and vertically), consistency, and composition.

5.1.2 Quaternary Alluvium and Slopewash

The Quaternary Alluvium and Slopewash material was encountered in CPT-1 and CPT-3 beneath the fill materials. This geologic unit is composed of a brown, silty, fine to medium coarse micaceous sand.

Alluvium and slopewash deposits are undifferentiated on the published geologic map of the project area (Kennedy, 1975a). The alluvium intertongues with Holocene slopewash located on the lower slopes. Both alluvium and slopewash material are typically poorly unconsolidated material derived from nearby bedrock sources (Kennedy, 1975a).

5.1.5 Quaternary Beach Sand

Beach sand deposits were encountered in CPT-2 at an approximate depth of 10 feet. This geologic unit is composed of a light gray fine sand with mica and trace silt. The beach deposits are largerly poorly consolidated and are derived from coastal processes and stream discharges.

At an approximate depth of 15 feet, a medium gray, clayey silt with trace sand was encountered in CPT-2. We interpret this to be a shallow marine deposit intertonguing with the overlying beach sand deposit.

5.2 Groundwater

The elevation of the groundwater table encountered in the push holes and indicated from the CPT results is estimated at approximately -4 feet MSL at the time of our field investigation. A limited geotechnical investigation which was performed for the City of San Diego Kellogg Park Green Lot Infiltration Project (Ninyo & Moore, 2009) reportedly encountered groundwater at elevations ranging from +2 to +5 feet MSL. The Green Lot Infiltration project is located approximately 0.25 mile north of the proposed project alignment at an approximate elevation of +8 feet MSL. It must be noted that variations in the elevation of the groundwater table should be expected in response to daily and seasonal tidal fluctuations.

6.0 DISCUSSIONS, OPINIONS AND RECOMMENDATIONS

6.1 Potential Geologic Hazards

6.1.1 Faulting and Seismicity

The Rose Canyon fault zone (RCFZ) represents the most significant seismic hazard in the San Diego metropolitan area. The RCFZ is a complex set of anastomosing and en-echelon, predominantly strike slip faults that extend from off the coast near Carlsbad to offshore south of downtown San Diego (Treiman, 1993). Investigations of the RCFZ in the Rose Creek area (Rockwell et. al., 1991) and in downtown San Diego (Patterson et. al., 1986) found evidence of multiple Holocene earthquakes. Based on these studies, several fault strands within the RCFZ have been classified as active faults, and are included in Alquist-Priolo Special Studies Zones. Within San Diego Bay, this fault zone is believed to splay into multiple, subparallel strands; the most pronounced of which are the Silver Strand, Spanish Bight and Coronado Bank faults. The Coronado Bank fault has been mapped within San Diego Bay at a distance of 13.1 mile from the project site. Local faults in San Diego Bay and the vicinity of the project site are depicted in Figure 2.

Based on known (published) geologic information, fault surface rupture is not considered to pose a significant geologic hazard to the proposed project. The computer program EQFAULT (Blake, 2000) was used to approximate the distance of known faults to the site. A copy of the search results is included in Appendix C. A summary of the seismic source characteristics of seven known active faults that are located within approximately 50 miles from the project site is presented below.

SUMMARY OF SEISMIC SOURCE CHARACTERISTICS

Fault	Maximum Magnitude (Mw)	Peak Site Acceleration (g)	Closest Distance to Site (miles)
Rose Canyon	6.9	0.492	0.75
Coronado Bank	7.4	0.292	13.1
Newport-Inglewood (offshore)	6.9	0.115	23.4
Elsinore - Julian	7.1	0.077	37.2
Earthquake Valley	6.5	0.036	45.0
Elsinore - Temecula	6.8	0.056	38.7
San Jacinto - Anza	7.2	0.036	59.6

It is our opinion that the major seismic hazards affecting the project area would be seismic-induced ground shaking. The project site will likely be subject to moderate to severe ground shaking in response to a local or more distant large magnitude earthquake occurring during the life of the planned facilities. For project design purposes, we recommend that the RCFZ be considered as the dominant seismic source.

For structural design in accordance with the CBC 2007, a computer program, Earthquake Ground Motion Parameters Version 5.09, developed by the United States Geological Survey (USGS, 2008) was utilized to provide ground motion parameters for the subject site. The program includes hazard curves, uniform hazard response spectra and design parameters for sites in the 50 United States, Puerto Rico and United States Virgin Islands based on the latitude and longitude, and site classification. Seismic design parameters and spectral response for both short period and 1-second period are calculated. Other parameters include Mapped Spectral Response Acceleration Parameter, Site Coefficient, Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter and Design Spectral Response Acceleration Parameter. The program is based on USGS research and publication in cooperation with the California Geological Survey for evaluation of California faulting and seismicity (USGS, 1996a, 1996b, 2002, 2007, 2008).

The CBC 2007 seismic design coefficients which are shown below should be used for seismic design of the proposed structures having fundamental period of 0.5 seconds or less. Structures with higher period will need to be designed using Site Class F classification due to the potential for liquefaction. These criteria are based on the soil profile type as determined by existing subsurface geologic conditions, the proximity of the site to a nearby fault, the maximum moment magnitude, and the slip rate of the nearby fault. It is noted that seismic design coefficients based on Site Class F classification will require additional borings and/or CPT soundings to a minimum depth of 60 feet bgs.

CBC 2007 SEISMIC DESIGN COEFFICIENTS

REFERENCE	PARAMETER
Table 1613.5.2	Site Class = D
Figure 1613.5(3)	Spectral Response Class B (short), Ss = 1.716
Figure 1613.5(4)	Spectral Response Class B (1 second), S1 = 0.684
Figure 1613.5.3(1)	Site Coefficient, Fa = 1.0
Figure 1613.5.3(2)	Site Coefficient, Fv = 1.5
Section 1613.5.3 (Eqn. 16-37)	MCE Spectral Response Acceleration (short), $S_{MS} = 1.716$
Section 1613.5.3 (Eqn. 16-38)	MCE Spectral Response Acceleration (1 second), $S_{M1} = 1.026$
Section 1613.5.3 (Eqn. 16-39)	5% Damped Design Spectral Response Acceleration (short), $S_{DS} = 1.144$
Section 1613.5.3 (Eqn. 16-40)	5% Damped Design Spectral Response Acceleration (1 second), $S_{D1} = 0.684$

6.1.2 Soil Liquefaction

An evaluation of liquefaction potential at the project site was conducted on the basis of the results of the CPT soundings and the simplified procedures outlined by Seed, et al. (1983). The procedure empirically correlates in-situ soil resistance with intensity of ground shaking to evaluate whether a soil is susceptible to liquefaction or not. The correlation is based on data collected from sites where soils have or have not liquefied during a documented earthquake event. Equivalent SPT blow counts (N-values) converted from CPT sounding logs are used in the in-situ evaluation of soil resistance. For analysis of liquefaction potential at the project site, a ground acceleration of 0.5g was selected as a reasonable representative of repeatable level of ground acceleration associated with a MCE of 7 earthquake on the RCFZ.

The results of the liquefaction analyses indicate that the potential for liquefaction increases in a westerly (towards the beach) direction. Liquefaction at the project site most likely would manifest itself as local ground subsidence and settlement. Due to the relatively level ground surface elevation, lateral flow is not likely to occur.

Liquefaction settlement analyses based on the CPT results indicate that ground surface settlement on the order of zero at the location of CPT-1 to as much as 1.5 inches at the location of CPT2 may be anticipated during a seismic-induced soil liquefaction event at the site. Subterranean structures and pipelines may also subject to uplift pressures during a seismic event. Recommendations for mitigation measures to provide uplift resistance are presented in the following sections of this report. It is noted that the liquefaction settlement analyses were performed using data obtained from the upper 20 feet of subsurface material, and the liquefaction phenomenon have been observed to extend to depths on the order of 60 feet bgs. Therefore, the actual ground surface settlement may be larger than the above calculated values.

6.1.3 Landslides

A review of the published geologic maps indicate that the project alignment is not located on or below any known (mapped) ancient landslides (Kennedy, 1975a and City of San Diego, 1995).

6.2 Cut-and-Cover Construction

Since no changes to the existing ground surface along the cut-and-cover segment of the proposed pipeline alignment are planned, the net stress change in the underlying soils is considered negligible. Furthermore, the native soils at the proposed invert level along the pipeline alignment are expected to provide a stable trench bottom. In the event that loose or disturbed soils are encountered at the trench bottom, it is recommended that they be over-excavated and replaced with pipe bedding or other approved materials.

6.2.1 Soil and Excavation Characteristics

The soil materials within the anticipated depths of the pipe trench excavation consists of man-made fill, undifferentiated alluvial and slopewash deposits, and beach sand which can be readily excavated with conventional heavy-duty construction equipment.

The majority of the soil materials encountered in the borings are considered suitable for use as compacted trench backfill materials provided that they are free of biodegradable materials, trash, rocks or hard lumps greater than 4 inches in maximum dimension, hazardous substance contamination, or other deleterious debris. If the fill materials contain rocks or hard lumps, at least 70 percent (by weight) of its particles shall pass a U.S. Standard $^{3}/_{4}$-inch sieve.

6.2.2 Pipe Loads and Settlement

Pipes should be designed for all loads applied by surrounding soils including dead load from soils, loads applied at the ground surface, uplift loads, and earthquake loads. Soil loading above the groundwater level may be estimated assuming a density of 130 pcf for the backfill materials.

Where a pipe changes direction abruptly, resistance to thrust forces can be provided by means of thrust blocks. For design purposes, for the passive resistance against thrust blocks embedded in dense soil materials, an equivalent fluid density of 300 pcf may be used. Thrust blocks should be embedded a minimum of 3 feet beneath the ground surface.

Buried flexible pipes are generally designed to limit deflections caused by applied loads. The deflections can be estimated using the Modified Spangler equation. A modulus of soil reaction, E', equal to 1,000 and 2,000 psi may be used to represent a minimum of 6 inches of compacted pipe bedding materials of low plasticity (LL < 50) with less than 12 percent fines passing the #200 standard sieve and crushed rock materials, respectively.

6.2.3 Trench Backfill

Pipe Bedding Zone and Pipe Zone

"Pipe Bedding Zone" is defined as the area below the bottom of the pipe and extending over the full trench width, and should be at least 6 inches thick in order to provide a uniform firm foundation material directly beneath the pipe.

The "Pipe Zone" is defined as the full width of a trench from the bottom of the pipe to a horizontal level about 6 inches above the top (crown) of the pipe. In order to provide uniform support and to minimize external loads, trench widths should be selected such that a minimum clear space of 6 inches is provided on each side of the pipe. During backfilling, it is recommended that the backfill materials be placed on each side of the pipe simultaneously to avoid unbalanced loads on the pipe.

Backfill materials placed in the "Pipe Bedding Zone" and "Pipe Zone" should consist of clean, free draining sand or crushed rock. Sand should be free of clay, organic matter, and other deleterious materials and conform to the gradation shown in the following table.

Sieve Size	Percent Passing by Weight (percent)
½ inch	100
#4	75-100
#16	35-75
#50	10-40
#200	0-10

Crushed rock should conform to Section 200-1.2 and 200-1.3 of the Standard Specifications for Public Works Construction (SSPWC) for 3/4-inch crushed rock gradation. It must be noted that, since the native soil materials do not meet these specifications, import backfill materials will be required for the "Pipe Bedding Zone" and "Pipe Zone". If crushed rock is to be used for pipe zone and bedding backfill materials, we recommend that the rock materials be wrapped in geotextile filter fabric such as Mirafi 140N or equivalent. The purpose of the filter fabric is to prevent migration of finer grained materials from the backfill materials, and the sides and bottom of the trench into the rock bedding materials.

Above Pipe Zone

The "Above Pipe Zone" is defined as the full width of the trench from the top of the "Pipe Zone" to the finish grade or bottom of the pavement section. Backfill placed in this zone should have less than 40 percent passing the standard #200 sieve and not less than 70 percent passing the U.S. standard $^3/_4$-inch sieve, and should not contain any organic debris, rocks or hard lumps greater than 6 inches, or other deleterious materials.

6.2.4 Placement and Compaction of Backfill

Prior to placement, all backfill materials should be moisture-conditioned, spread and placed in lifts (layers) not-to-exceed 6 inches in loose (uncompacted) thickness, and uniformly compacted to at least 90 percent relative compaction. During backfilling, the soil moisture content should be maintained at or within 2 to 3 percent above the optimum moisture content of the backfill materials. It is recommended that the upper 24 inches directly beneath the roadway pavement and the base materials be compacted to at least 95 percent relative compaction. The maximum dry density and optimum moisture content of the backfill materials should be determined in the laboratory in accordance with the ASTM D1557-00 testing procedures.

Small hand-operated compacting equipment should be used for compaction of the backfill materials to an elevation of at least 4 feet above the top (crown) of the pipes. Flooding or jetting should not be used to densify the backfill.

6.3 Buried Structures

It is recommended that any proposed buried structures be founded on firm native soils or approved compacted materials. In areas where loose or soft soils are encountered at the bottom of the box structure excavations, it is recommended that the loose/soft materials be removed and replaced with 3/4-inch crushed rock materials wrapped in geotextile fabric which meets or exceeds the specifications shown on the next page.

Fabric Property	Min. Certified Values	Test Method
Grab Tensile Strength	300 lb	ASTM D 4632
Grab Tensile Elongation	35% (MAX)	ASTM D 4632
Burst Strength	600 psi	ASTM D 3786
Trapezoid Tear Strength	120 lb	ASTM D 4533
Puncture Strength	130 lb	ASTM D 4833

The actual extent of over-excavation of any loose/soft soil materials should be evaluated and determined in the field by the City's Resident Engineer.

6.3.1 Placement and Compaction of Backfill

Placement and compaction of backfill materials around the buried structures should be performed in accordance with the recommendations presented in Section 6.2.4 of this report.

6.3.2 Foundations

Bearing Capacity

For design of the buried structures which are founded on firm native soils or uniformly compacted fill materials an allowable soil bearing capacity of 3,000 and 2,000 psf may be used, respectively. This allowable soil bearing value is for total dead and live loads, and may be increased by one third when considering seismic loads.

Anticipated Settlement

Under static condition, total settlement of the slab foundation is estimated to be less than 0.25 inch. Differential settlement between the center and the edge of the slab foundation is expected not to exceed 0.25 inch. No permanent deformation and/or post-construction settlement is anticipated, provided that backfill around the structures is properly compacted in accordance with the project specifications.

Resistance to Lateral Loads

Resistance to lateral loads may be developed by a combination of friction acting at the base of the slab foundation and passive earth pressure developed against the sides of the foundations below grade. Passive pressure and friction may be used in combination, without reduction, in determining the total resistance to lateral loads.

An allowable passive earth pressure of 300 psf per foot of foundation embedment below grade may be used for the sides of foundations placed against competent native soils or properly compacted fill materials. The maximum recommended allowable passive pressure is 3,000 psf. A coefficient of friction of 0.4 may be used for foundation cast directly on competent native soils or approved compacted materials.

6.3.3 Walls Below Grade

Lateral earth pressures for walls below grade for structures less than 48 inches in horizontal dimensions may be treated as a shaft structure. Walls below grade for structures larger than 48 inches in horizontal dimensions should be designed to resist the lateral earth pressures presented in Figures 3 and 4 provided that the wall backfill materials are properly placed and compacted in conformance with the recommendations presented in this report. Surcharge and foundation loads occurring within a horizontal distance equal to the wall height should be added to the lateral pressures as presented in Figures 5 and 6.

6.3.4 Uplift Resistance

Buried structures located below the groundwater table will be subject to buoyant uplift forces. Geotechnical parameters for use in calculating uplift resistance of the surrounding backfill soil materials is presented in Figures 7 and 8.

7.0 CONSTRUCTION-RELATED CONSIDERATIONS

7.1 Temporary Shoring

Since the anticipated pipe invert depths will be more than 5 feet below the ground surface, prevailing Federal and Cal OSHA safety regulations require that the trenched excavation be either sloped (if sufficient construction space or easement is available), shored, braced, or protected with approved sliding trench shield. Limited construction space, the presence of other buried utilities, and the need to avoid excessive community disruption dictate that a shored excavation will be needed along the entire pipeline alignment. Design and construction of temporary shoring should be the sole responsibility of the contractor.

7.1.1 Settlement

Settlement of existing street improvements and/or utilties adjacent to the shoring may occur in proportion to both the distance between shoring system and adjacent structures or utilities and the amount of horizontal deflection of the shoring system. Vertical settlement will be maximum directly adjacent to the shoring system, and decreases as the distance from the shoring increases. At a distance equal to the height of the shoring, settlement is expected to be negligible. Maximum vertical settlement is estimated to be on the order of 75 percent of the horizontal deflection of the shoring system. It is recommended that shoring be designed to limit the maximum horizontal deflection to 1-inch or less where structures or utilities are to be supported.

7.1.2 Lateral Earth Pressures

Temporary shoring should be designed to resist the pressure exerted by the retained soils and any additional lateral forces due to loads placed near the top of the excavation. For design of braced shorings supporting fill materials, topsoil, alluvial deposits, or slopewash, the recommended lateral earth pressure should be 32H psf, where H is equal to the height of the retained earth in feet. For braced shoring supporting bedrock, the recommended lateral earth pressures may be reduced to 20H psf. Any surcharge loads would impose uniform lateral pressure of 0.3q, where "q" equals the uniform surcharge pressure. The surcharge pressure should be applied starting at a depth equal to the distance of the surcharge load from the top of the excavation.

The recommended lateral earth pressures have been prepared based on the assumptions that the shored earth is level at the surface, no hydrostatic pressures above the bottom of the excavation, and that the shoring system is temporary in nature.

7.1.3 Lateral Bearing Capacity

Resistance to lateral loads will be provided by passive soil resistance. The allowable passive pressure for the native soils may be assumed to be equivalent to a fluid weighing 300 pcf. Allowable lateral bearing pressure should not exceed 3,000 psf.

7.2 Construction Dewatering

Based on the information obtained from the borings, groundwater is not anticipated to be encountered within the majority of the project alignment. It must be noted , however, that shallow water conditions will be encountered as the project alignment extends closer to the shoreline. It is anticipated that dewatering in these areas, if necessary, can be accomplished by sump and pump methods.

7.3 Environmental Considerations

The authorized scope of our investigation for this project did not include the performance of a Phase I environmental site assessment to evaluate the possible presence of contaminated soil and/or groundwater conditions at or near the project site. Although the possibility that hazardous materials occur along the project alignments is considered very low, we recommend that the contract documents include provisions for the contractor to be prepared to handle and dispose such materials, if encountered, in accordance with current industry standard of care and applicable Federal, State and local laws and regulations. It is further recommended that the contractor be requested to prepare and submit a health and safety plan which includes contingency measures to deal with potential hazardous materials issues for review and approval by the City prior to commencement of construction.

8.0 GENERAL CONDITIONS

8.1 Post-Investigation Services

Post-investigation geotechnical services are an important continuation of this investigation, and we recommend that the City retains our firm to perform the necessary geotechnical observation and testing services during construction.

Sufficient and timely observation and testing should be performed during excavation, pipeline installation, backfilling and other related earthwork operations. The purpose of the geotechnical observation and testing is to correlate findings of this investigation with the actual subsurface conditions encountered during construction and to provide supplemental recommendations, if necessary. The geotechnical observation and testing are also intended to confirm that the pipeline is placed on competent soil materials, and that suitable pipe bedding and backfill materials have been properly placed and compacted in conformance with the specifications.

8.2 Uncertainties and Limitations

The information presented in this report is intended for the sole use of Tetra Tech and the City for project design purposes only and may not provide sufficient data to prepare an accurate bid. The contractor should be required to perform an independent evaluation of the subsurface conditions at the project site prior to submitting his/her bid.

Our firm has observed and investigated the subsurface conditions only at specific locations along the proposed project alignment. The findings and recommendations presented in this report are based on the assumption that the subsurface conditions along the entire length of the project alignment do not deviate substantially from those encountered in the exploratory CPT soundings. Consequently, modifications or changes to the recommendations presented herein may be necessary based on the actual subsurface conditions encountered during construction.

California, including San Diego County, is in an area of high seismic risk. It is generally considered economically unfeasible to build a totally earthquake-resistant project and it is, therefore, possible that a nearby large magnitude earthquake could cause damage at the project site.

Geotechnical engineering and geologic sciences are characterized by uncertainty. Professional judgments and opinions presented in this report are based partly on our evaluation and analysis of the technical data gathered during our present study, partly on our understanding of the scope of the proposed project, and partly on our general experience in geotechnical engineering.

In the performance of our professional services, we have complied with that level of care and skill ordinarily exercised by other members of the geotechnical engineering profession currently practicing under similar circumstances in southern California. Our services consist of professional consultation only, and no warranty of any kind whatsoever, expressed or implied, is made or intended in connection with the work performed. Furthermore, our firm does not guarantee the performance of the project in any respect.

Our firm does not practice or consult in the field of safety engineering. The contractor will be responsible for the health and safety of his/her personnel and all subcontractors at the construction site. The contractor should notify the City if he or she considers any of the recommendations presented in this report to be unsafe.

9.0 REFERENCES

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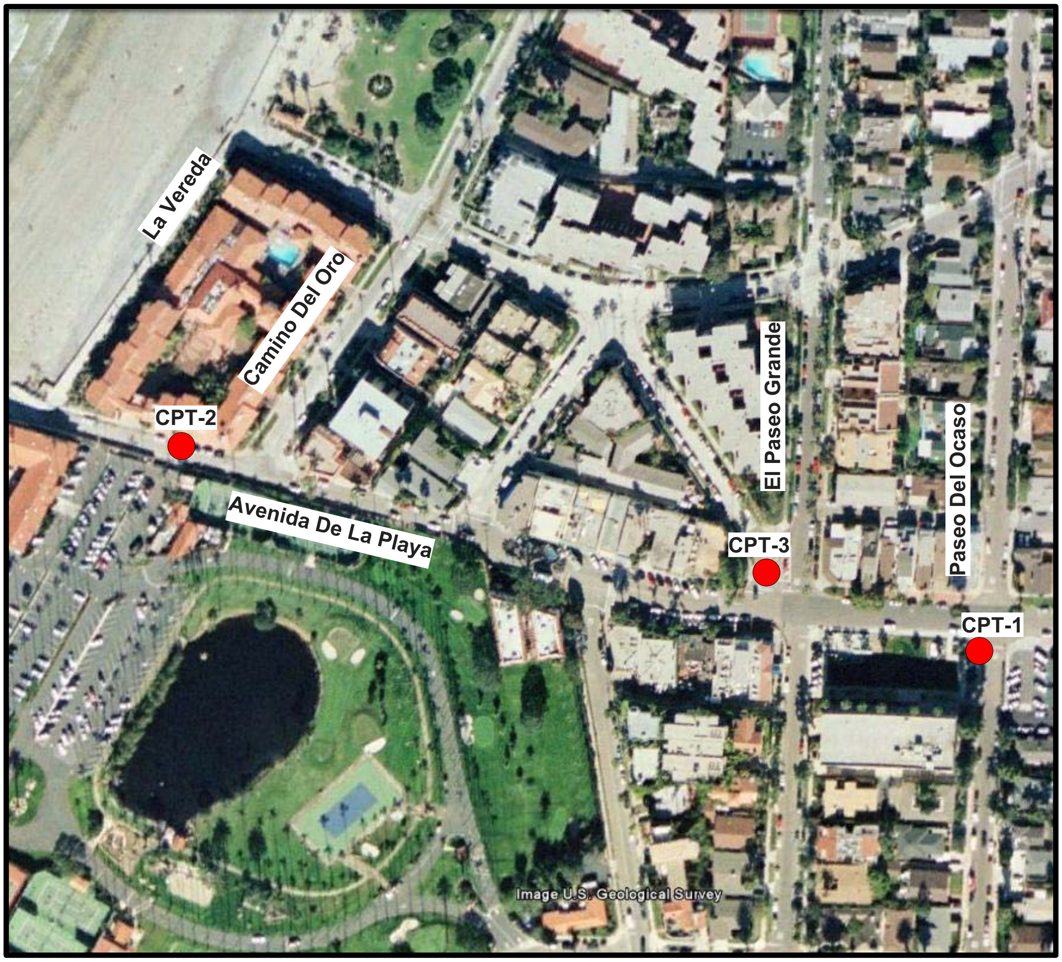
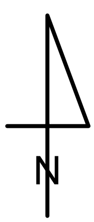
FIGURES



LEGEND:

CPT-1 ● **Approximate CPT Sounding Location**



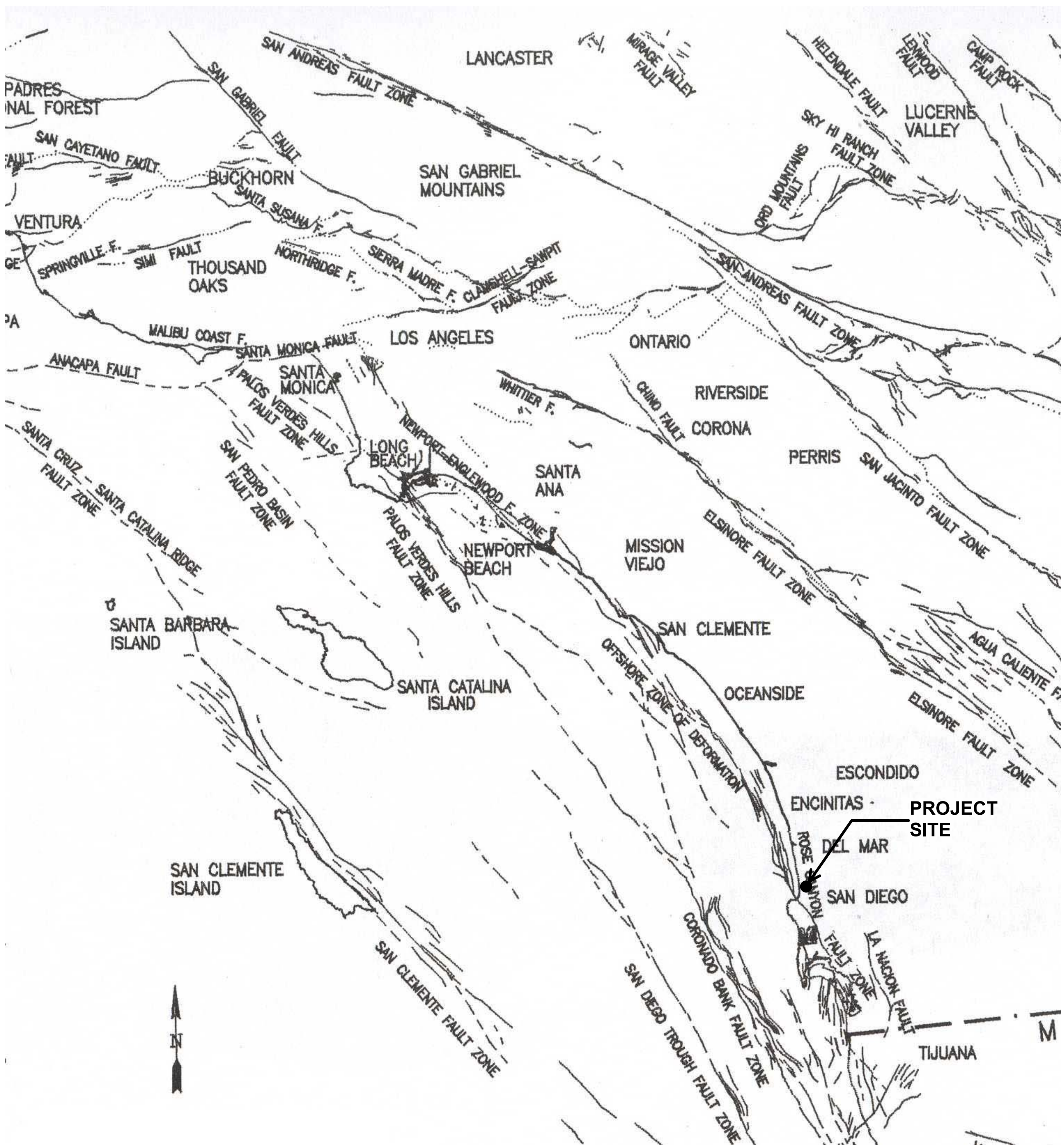
NOT TO SCALE

SOURCE:

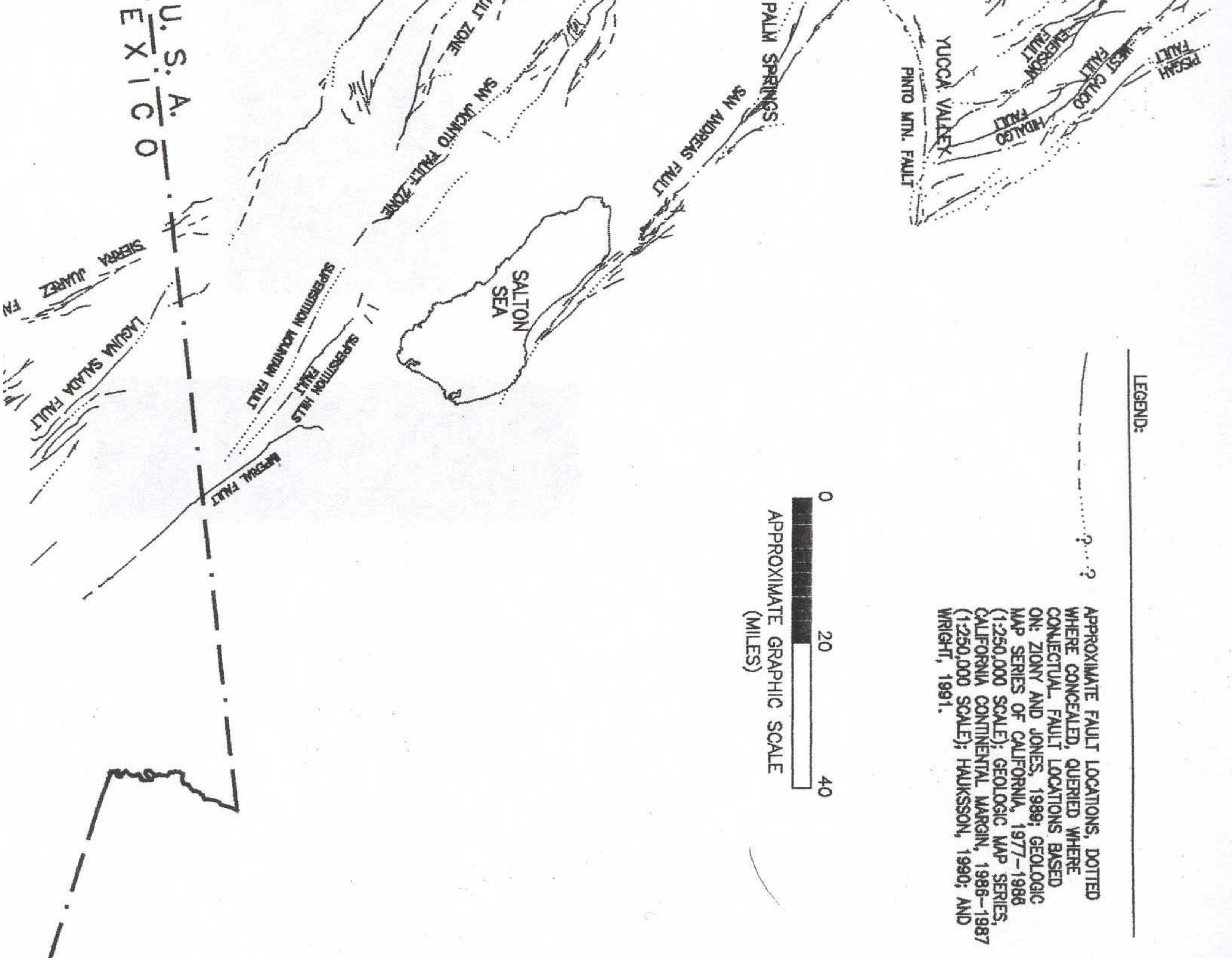
Google Earth, 2010

VICINITY MAP
AVENIDA DE LA PLAYA STORM DRAIN AND WATER SEWER GROUP 809

PROJECT NO. 44A408	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE 1**

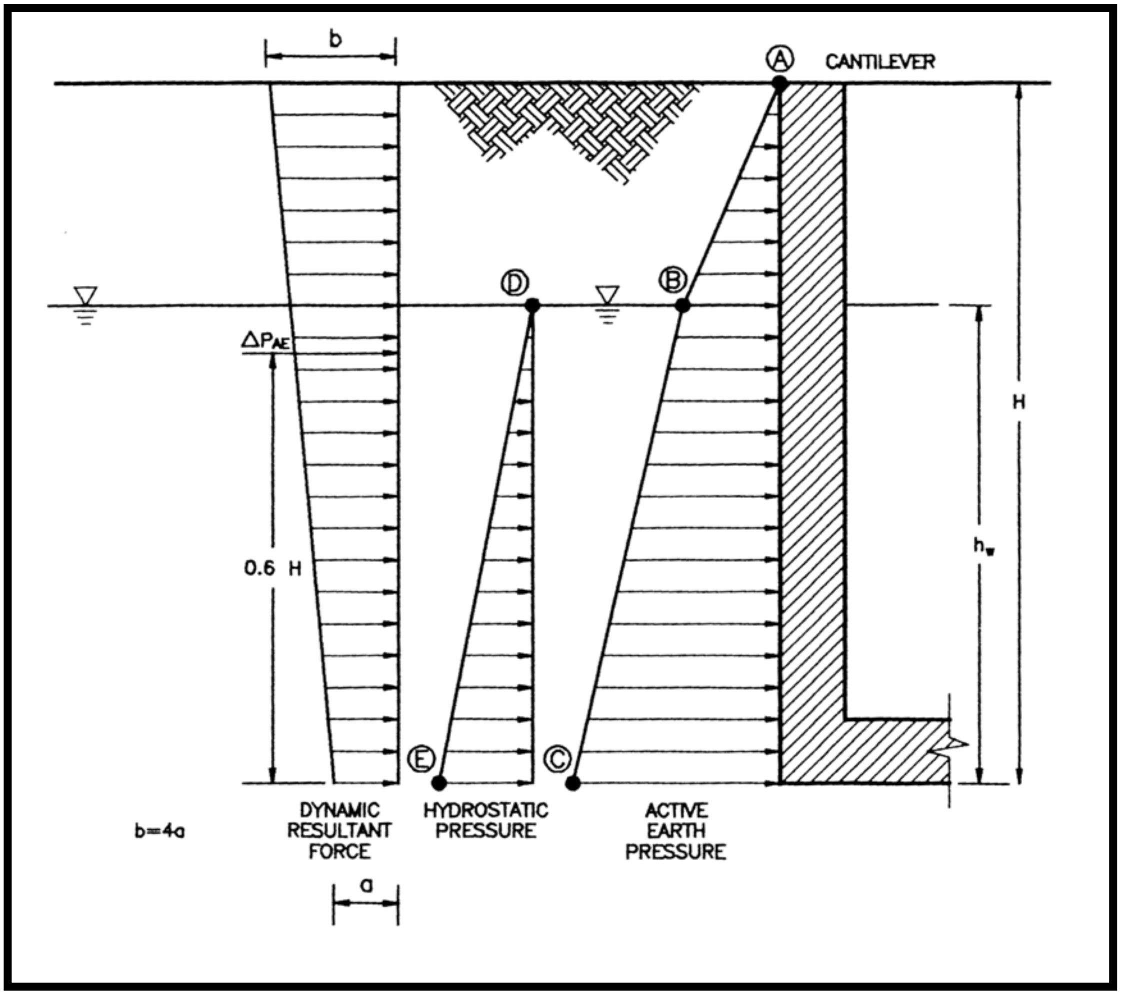


SAN ANDREAS FAULT ZONE
LANCASTER
MIRAGE VALLEY FAULT
HELENDALE FAULT
LENWOOD FAULT
CAMP ROCK FAULT
LUCERNE VALLEY
PADRES
NAL FOREST
SAN GABRIEL FAULT
SAN CAYETANO FAULT
FAULT
SKY HI RANCH FAULT ZONE
BUCKHORN
SAN GABRIEL MOUNTAINS
ORD MOUNTAINS FAULT
SANTA SUSANA F.
VENTURA
SPRINGVILLE F.
SIMI FAULT
THOUSAND OAKS
NORTHRIDGE F.
SIERRA MADRE F.
CLAMSHELL-SAWPIT FAULT ZONE
SAN ANDREAS FAULT ZONE
PA
MALIBU COAST F.
SANTA MONICA FAULT
LOS ANGELES
ONTARIO
ANACAPA FAULT
SANTA MONICA
PALOS VERDES HILLS FAULT ZONE
WHITTIER F.
CHINO FAULT
RIVERSIDE
CORONA
SANTA CRUZ - SANTA CATALINA RIDGE FAULT ZONE
SAN PEDRO BASIN FAULT ZONE
LONG BEACH
NEWPORT-ENGLEWOOD F. ZONE
SANTA ANA
PERRIS
SAN JACINTO FAULT ZONE
PALOS VERDES HILLS FAULT ZONE
NEWPORT BEACH
MISSION VIEJO
ELSINORE FAULT ZONE
SANTA BARBARA ISLAND
SAN CLEMENTE
AGUA CALIENTE FA
OFFSHORE ZONE OF DEFORMATION
SANTA CATALINA ISLAND
OCEANSIDE
ELSINORE FAULT ZONE
ESCONDIDO
ENCINITAS
PROJECT SITE
ROSE CANYON
DEL MAR
SAN CLEMENTE ISLAND
SAN DIEGO
FAULT ZONE
LA NACION FAULT
SAN CLEMENTE FAULT ZONE
CORONADO BANK FAULT ZONE
SAN DIEGO TROUGH FAULT ZONE
TIJUANA
M
N



REGIONAL FAULT MAP
AVENIDA DE LA PLAYA STORM DRAIN AND WATER SEWER GROUP 809

PROJECT NO. 44A408	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 2



NOTES

H = wall height in feet

h_w = water height above bottom of structure in feet

Lateral pressure values presented herein are based on the assumption that non-expansive backfill materials will be used to backfill behind walls

LATERAL PRESSURES

Earth Pressure

A = 0
B = 35 $(H-h_w)$, psf
C = 35 $(H-h_w) + 20h_w$, psf

Hydrostatic Pressure

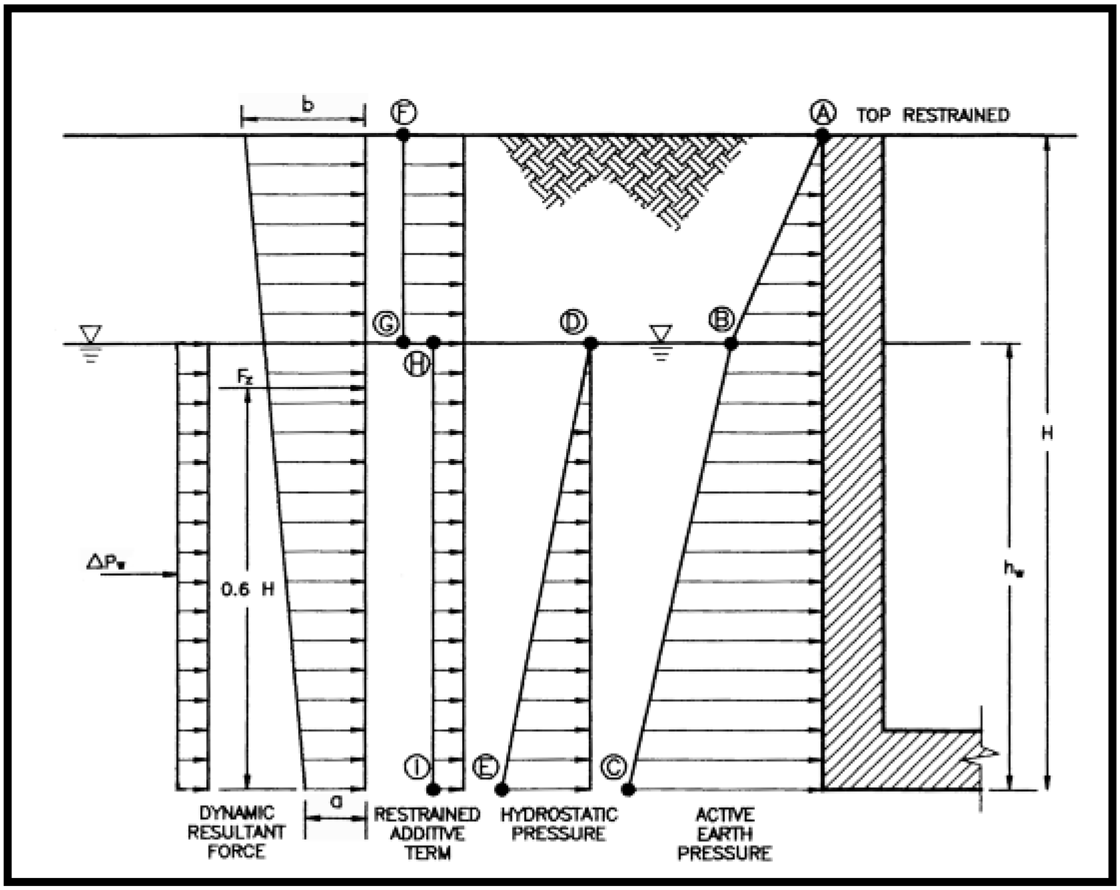
D = 0
E = $62.4h_w$

Dynamic Resultant Force

a = 2.9H
b = 4a = 11.6H
$\Delta P_{AE} = 7.25H^2$, lb/ft @ 0.6H

LATERAL PRESSURES FOR CANTILEVER WALLS
AVENIDA DE LA PLAYA STORM DRAIN AND WATER SEWER GROUP 809

PROJECT NO. 44A408	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 3



NOTES

H = wall height in feet

h_w = water height above bottom of structure in feet

Lateral pressure values presented herein are based on the assumption that non-expansive backfill materials will be used to backfill behind walls

LATERAL PRESSURES

Earth Pressure
A = 0
B = 35 ($H-h_w$), psf
C = 35 ($H-h_w$) + $20h_w$, psf

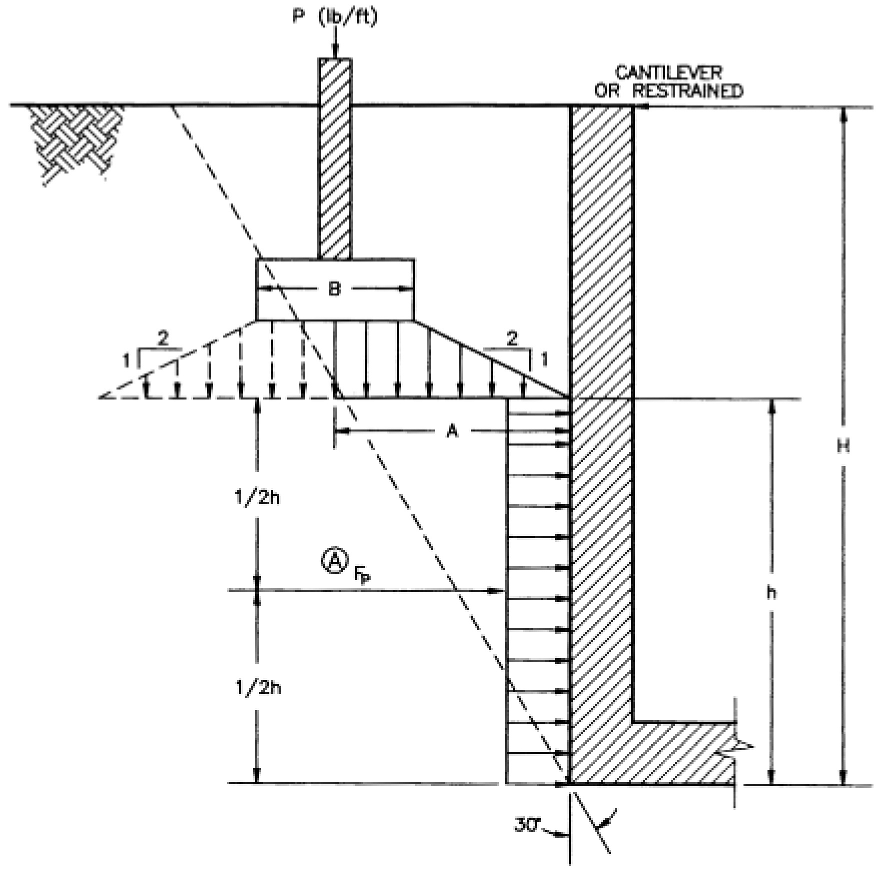
Hydrostatic Pressure
D = 0
E = $62.4h_w$

Restrained Additive Term
F = G = 10H, psf
H = I = 5H, psf

Dynamic Resultant Force
a = 9.9H
b = 4a = 39.6H
$F_z = 24.75H^2$, lb/ft @ 0.6H (Soil)
$\Delta P_w = 21.6h_w^2$, lb/ft @ $0.5h_w$ (Water)

LATERAL PRESSURES FOR RESTRAINED WALLS
AVENIDA DE LA PLAYA STORM DRAIN AND WATER SEWER GROUP 809

PROJECT NO. 44A408	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 4



NON-EXPANSIVE BACKFILL

F_p = M (A/B) P, lb/ft

A = h tan 30°, ft
M = 0.3 for cantilever wall
M = 0.4 for restrained wall

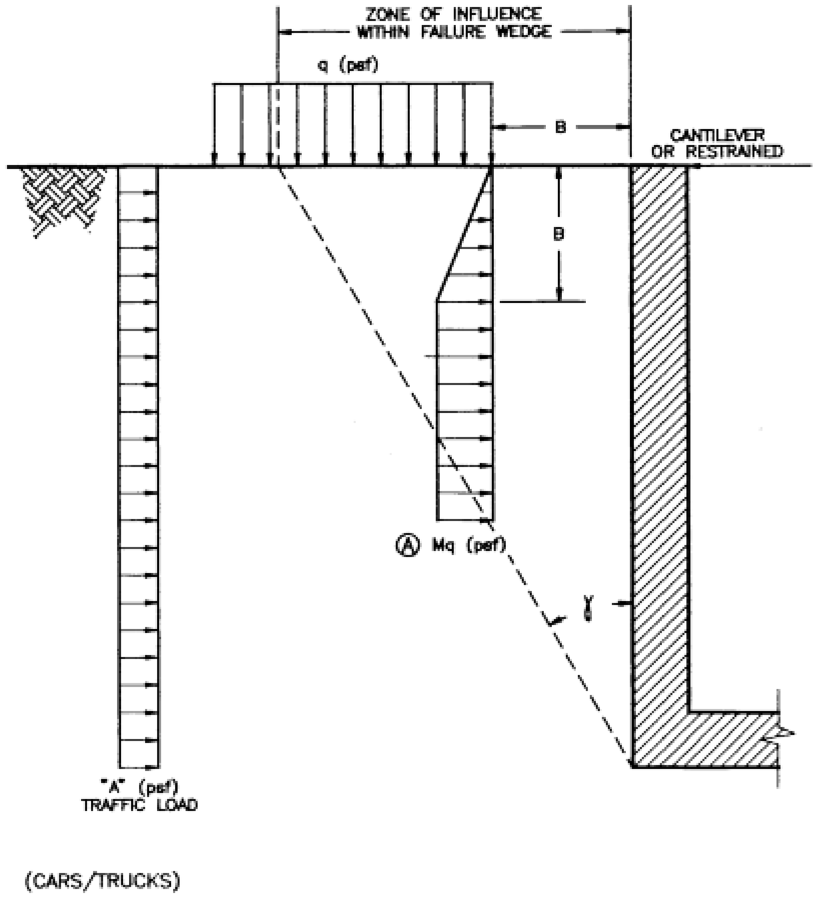
NOTES:

1. Surcharge pressure acting on wall is not affected by groundwater elevation.

2. Surcharge pressures shown are applicable for continuous footing only. Spread footings need to be evaluated individually.

FOUNDATION INDUCED WALL PRESSURES
AVENIDA DE LA PLAYA STORM DRAIN AND WATER SEWER GROUP 809

PROJECT NO. 44A408	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 5



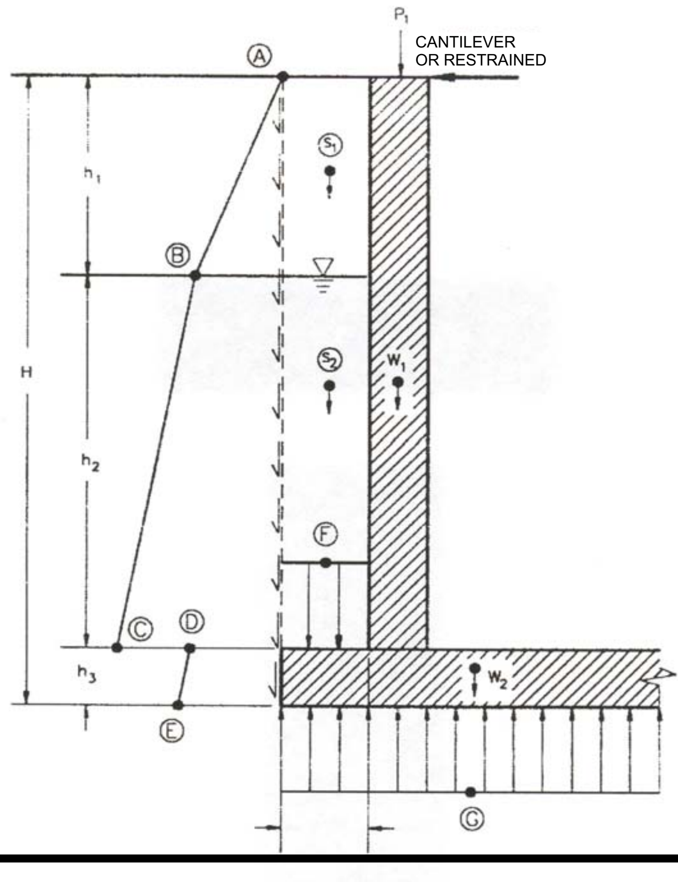
NON-EXPANSIVE BACKFILL

q = surcharge load (psf)
B = distance between wall and surcharge load, ft
M = 0.3 for cantilever wall
M = 0.4 for restrained wall
A = Mq, psf
"A" = 75 psf
$\gamma = 30^{o}$

NOTE: Surcharge pressure acting on wall is not affected by groundwater elevation.

TRAFFIC AND SURCHARGE PRESSURES
AVENIDA DE LA PLAYA STORM DRAIN AND WATER SEWER GROUP 809

PROJECT NO. 44A408	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 6



PROPERLY COMPACTED BACKFILL

Soil Friction, psf

A = 0
B = $40h_1$
C = $40h_1 + 20\ h_2$
D = $24h_1 + 12\ h_2$
E = $24h_1 + 12\ h_2 + 12\ h_3$

Soil Weights - Within Vertical Prism, pcf

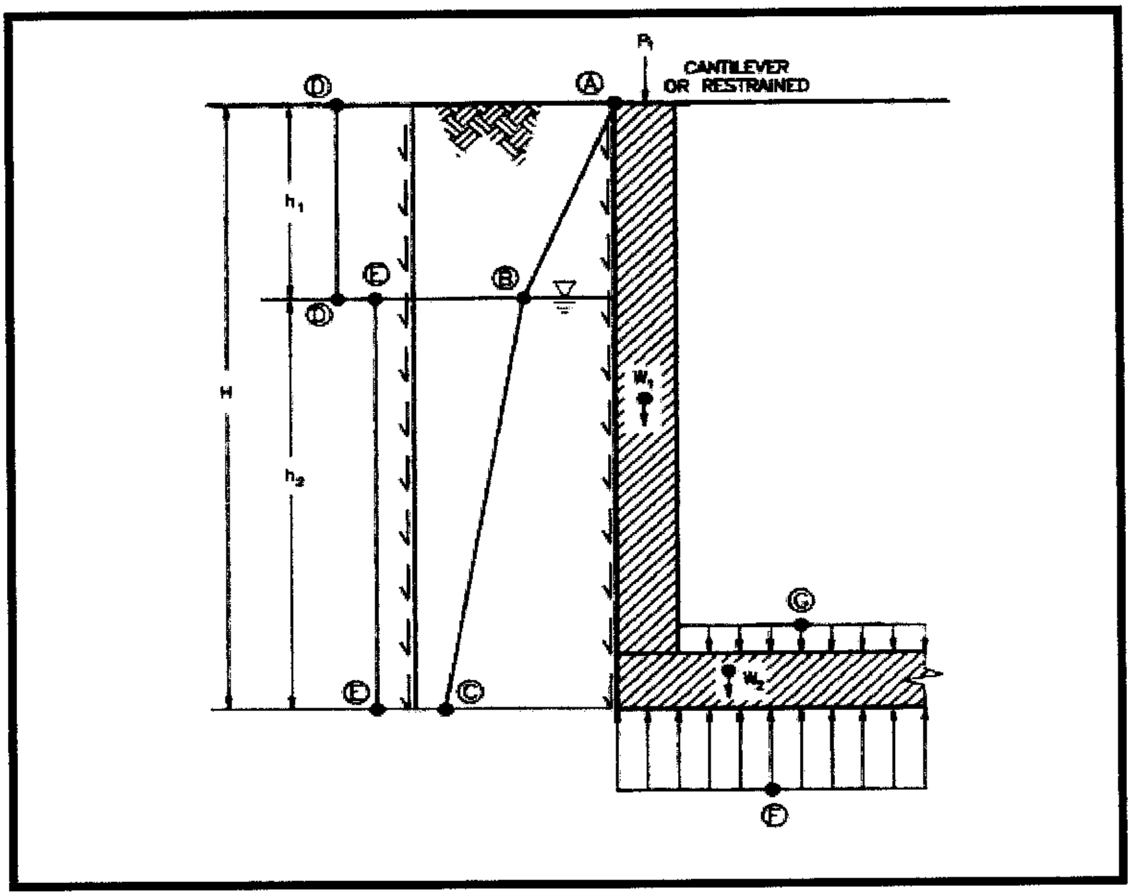
S_1 = 130 (above groundwater)
S_2 = 62 (below groundwater)

Hydrostatic Pressure, psf

F = $62.4\ h_2$
G = $62.4\ (h_2 + h_3)$

UPLIFT RESISTANCE FOR WALLS WITH EXTENSION
AVENIDA DE LA PLAYA STORM DRAIN AND WATER SEWER GROUP 809

PROJECT NO. 44A408	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 8



PROPERLY COMPACTED BACKFILL

Soil Friction, psf

A = 0
B = $22h_1$
C = $22h_1 + 11\,h_2$
D = 7H*
E = 4H*

Hydrostatic Pressure, psf

F = $62.4\,h_2$

NOTE: * D and E are only applicable for restrained walls and should be ignored if walls are to be designed as simple cantilever

UPLIFT RESISTANCE FOR WALLS WITHOUT EXTENSION AVENIDA DE LA PLAYA STORM DRAIN AND WATER SEWER GROUP 809		
PROJECT NO. 44A408	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 7

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

The field exploration program for this project included the advancement of three CPT soundings, and three push hole samples adjacent to the CPT soundings to an approximate depth of 20 feet below the existing ground surface. The field exploration activities were performed on May 27, 2010. The CPT soundings were advanced using a 30-ton CPT rig by Kehoe Testing & Engineering, Inc. Discrete soil samples were collected at approximately 5-foot interval using a 6600 Geoprobe rig. A description of the samples collected from the push holes is shown in Appendix B.

The CPT operation consists of hydraulically pushing an integrated electronic piezocone at an average rate of about 2 centimeters per second into a soil deposit. The piezocone consists of a conical tip and cylindrical friction sleeve mounted at the end of a series of sounding rods. The cone has a tip area of 10 cm2 and a friction sleeve area of 150 cm2. A pore water pressure filter, located directly behind the cone tip, was used to measure hydrostatic water pressures which can be utilized to estimate depth of groundwater table.

The CPT provides a continuous log of cone tip and sleeve resistance with depth. Measurements of resistance encountered during sounding are useful in evaluating the variation of material types, engineering properties, and liquefaction potential of soils. Information recorded along the depth of sounding consisted of tip resistance (Qt), sleeve friction (Fs), and pore water pressure (U). The results of CPT soundings are presented graphically and included herein.

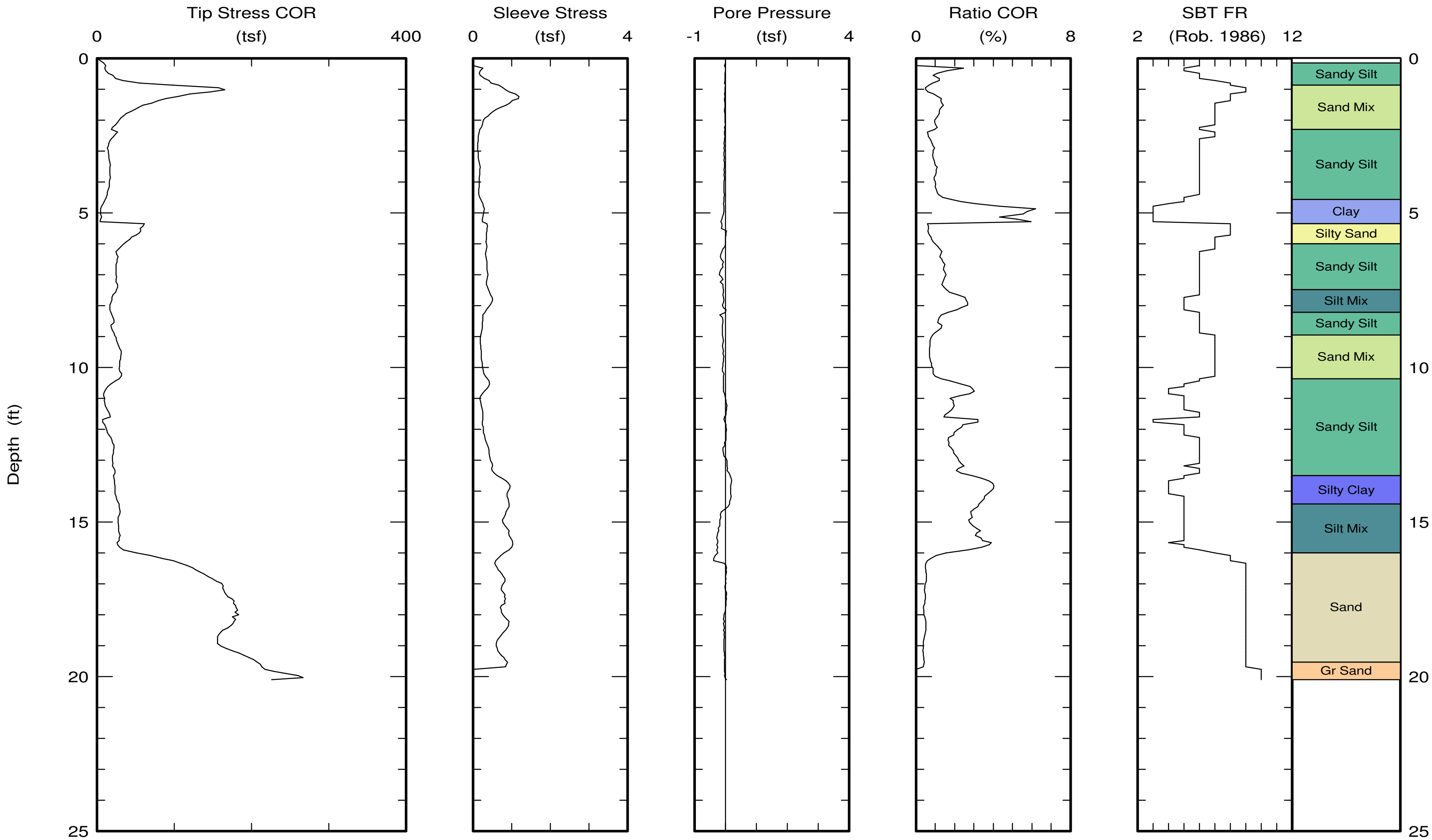
Soil behavior can be estimated on the basis of CPT results by using a correlation developed by Robertson and Campanella (1988) which compares tip resistance and friction ratio, Rf=Fs/Qt, expressed as a percent.

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
www.kehoetesting.com

CPT Data
30 ton rig

Date: 27/May/2010
Test ID: CPT-1
Project: SanDiego

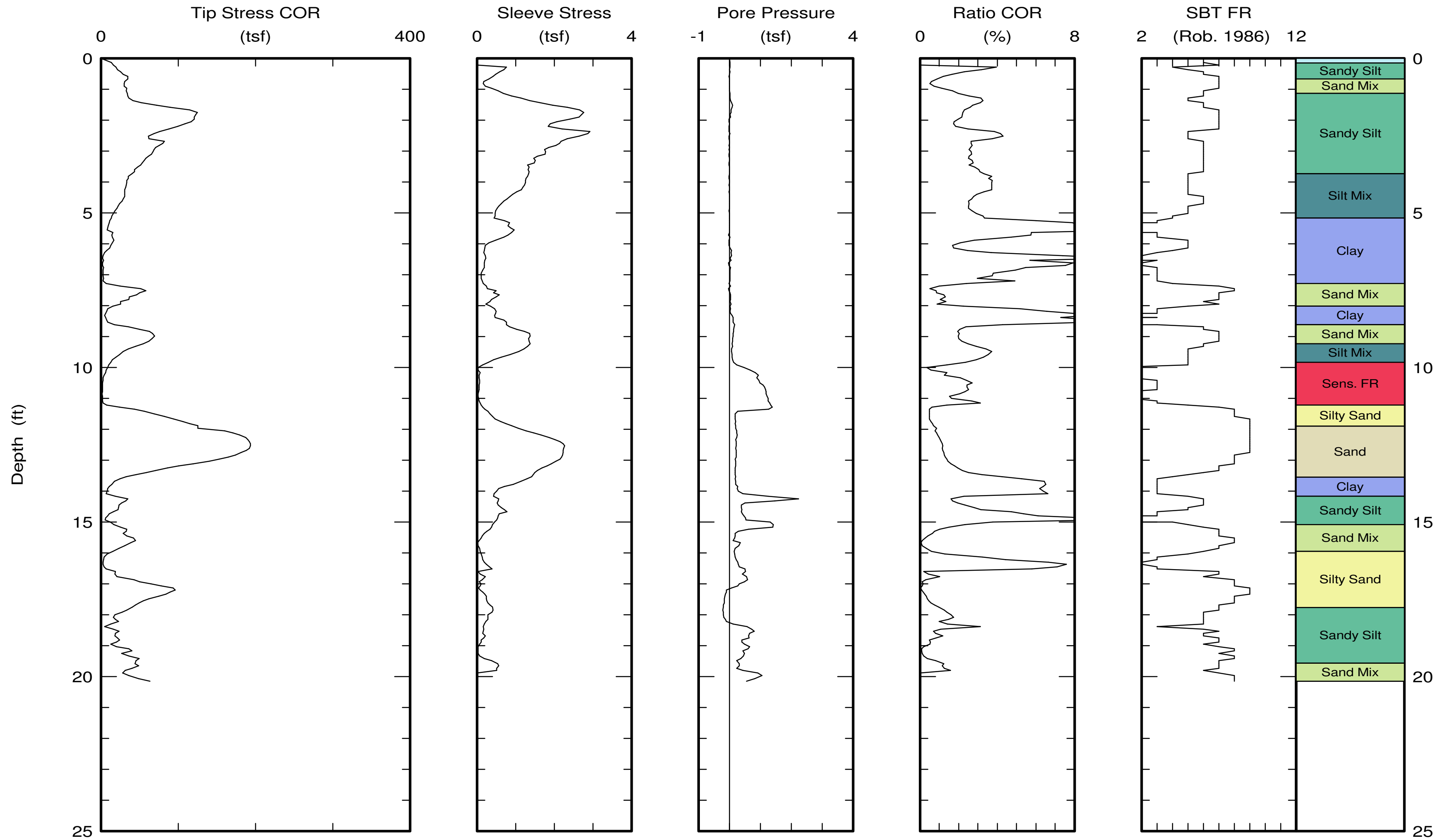
Customer: Allied Geotechnical
Job Site: Aveinda De La Playa



Maximum depth: 20.11 (ft)

Test ID: CPT-1
File: Z27Y1001C.ECP

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 27/May/2010 Test ID: CPT-2 Project: SanDiego
	Customer: Allied Geotechnical Job Site: Aveinda De La Playa	



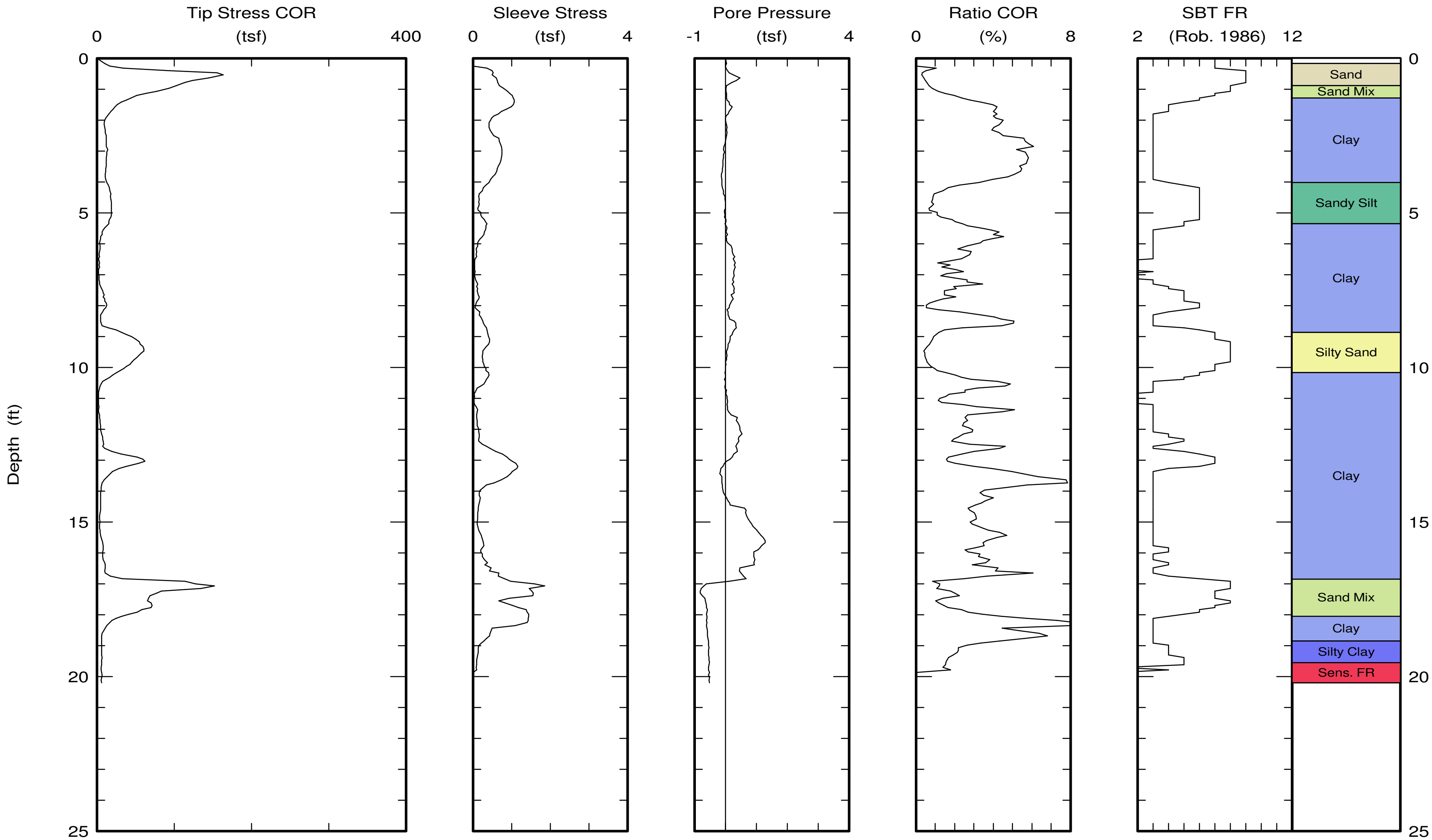
Maximum depth: 20.16 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
www.kehoetesting.com

CPT Data
30 ton rig

Date: 27/May/2010
Test ID: CPT-3
Project: SanDiego

Customer: Allied Geotechnical
Job Site: Aveinda De La Playa



Maximum depth: 20.21 (ft)

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Selected soil samples were tested in the laboratory to verify visual field classifications and to evaluate certain engineering characteristics. The testing was performed in accordance with the American Society for Testing and Materials (ASTM) or other generally accepted test methods, and included the following

- Determination of in-place dry density and moisture content (ASTM D2937) based on relatively undisturbed drive samples. The final test results are presented herein; and

- Mechanical sieve analyses (ASTM D422), and the final test results are plotted as gradation curves on Figure B-1 and B-2.

CPT-1

Sample No.	Depth (ft.)	Description	Moist. Content (%)	Density (pcf)
1	5	Damp, yellow brown, micaceous silty fine sand (SM)	22.0	99.6
2	10	Damp to moist, yellow brown, micaceous silty fine to medium sand (SM)	11.2	114.0
3	15	Moist, brown, clayey fine to coarse sand (SC)	13.8	123.6

CPT-2

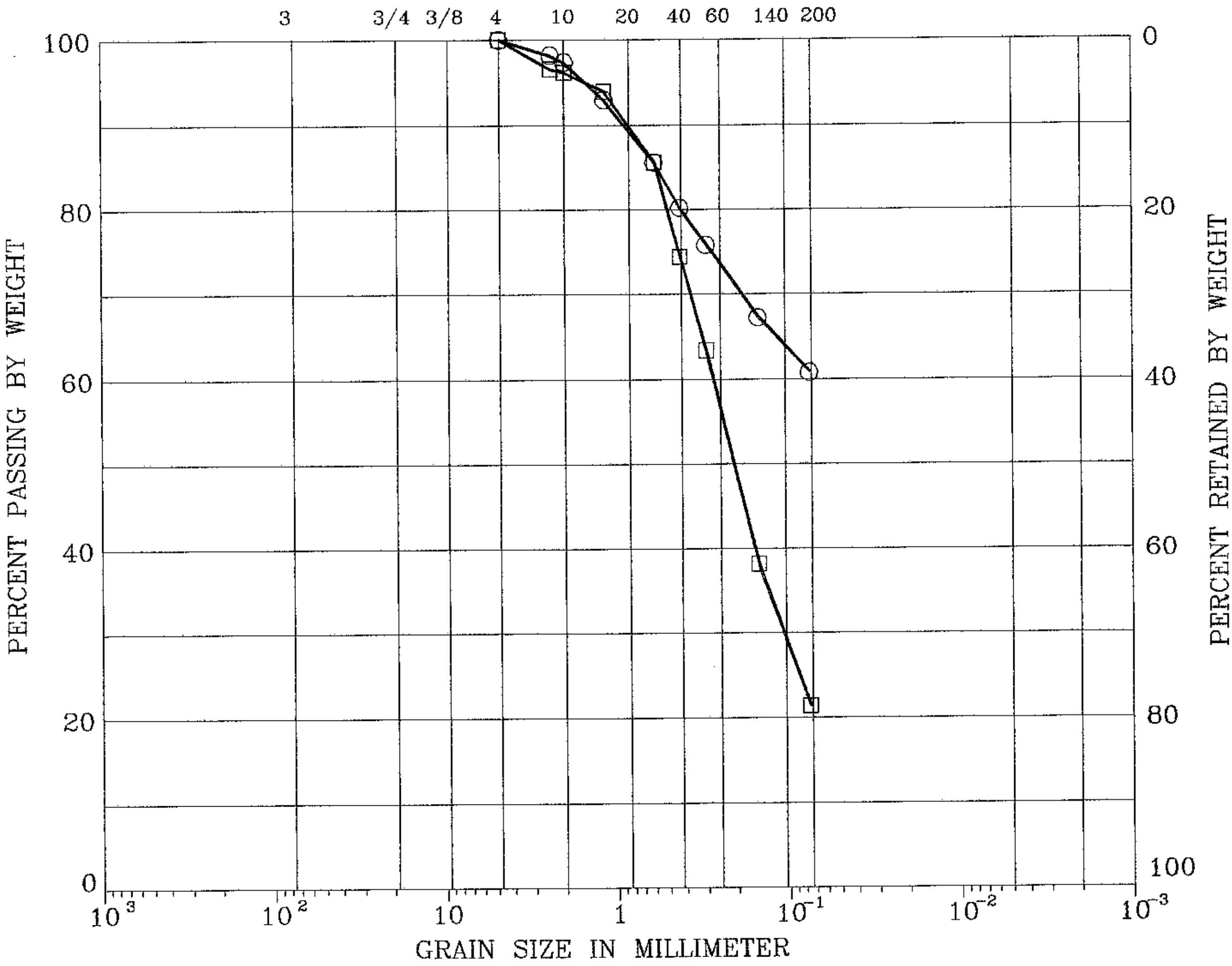
Sample No.	Depth (ft.)	Description	Moist. Content (%)	Density (pcf)
1	5	Moist, brown, micaceous silty medium to coarse sand (SM)	18.2	113.6
2	10	Moist to wet, light gray micaceous fine sand (SP) with traces of silt	34.5	89.1
3	15	Wet, gray clayey silt (ML) with traces of sand	34.8	90.9

CPT-3

Sample No.	Depth (ft.)	Description	Moist. Content (%)	Density (pcf)
1	5	Damp, yellow brown, micaceous silty fine sand (SM)	21.3	99.4
2	10	Wet, yellow brown, micaceous silty fine to medium sand (SM)	21.3	108.2
3	15	Moist, brown, micaceous fine sandy silt (ML)	23.6	105.2

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
U.S. SIEVE SIZE IN INCHES			U.S. STANDARD SIEVE No.			HYDROMETER



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	CPT2-15'	15'			SANDY SILT (ML)
□	CPT3-10'	10'			SILTY SAND (SM)

Remark :

Project No. 44A408	AVENIDA DE LA PLAYA
ALLIED GEOTECHNICAL ENGINEERS, INC.	GRAIN SIZE DISTRIBUTION Figure B-1