REPORT OF GEOTECHNICAL INVESTIGATION
MAPLE CANYON RESTORATION PHASE 1
CITY OF SAN DIEGO

Submitted to:

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Prepared By:

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August 19, 2014

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Mr. Gerard Dalziel, P.E.
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San Diego, CA 92111

Subject: **REPORT OF GEOTECHNICAL INVESTIGATION**
MAPLE CANYON RESTORATION PHASE 1
CITY OF SAN DIEGO
AGE Project No. 60C508

Dear Mr. Dalziel:

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 to assist AECOM with their design of the subject project.

We appreciate the opportunity to be of service on this project. If you have any questions regarding the contents of this report or need further assistance, please feel free to contact our office.

Sincerely,

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CITY OF SAN DIEGO

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1.0 INTRODUCTION

Allied Geotechnical Engineers, Inc. (AGE) is pleased to submit this report to present the findings, opinions, and recommendations of a geotechnical investigation conducted to assist AECOM with their design of the Maple Canyon Restoration Phase 1 project for the City of San Diego (City). The investigation was performed in conformance with AGE's proposal (revised) dated November 8, 2012, and the subconsultant agreement entered into by and between AECOM and AGE on March 19, 2013.

This report has been prepared for the exclusive use of AECOM and its design team subconsultants 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 SITE AND PROJECT DESCRIPTION

Maple Canyon is a southwest trending natural canyon located west of Balboa Park in the Bankers Hill and Middletown neighborhoods in the city of San Diego. The canyon is a designated open space that is maintained by the City Parks and Recreation Department Open Space Division. The canyon is approximately 3,000 feet in length, extending southwesterly from Redwood Street between 3rd and 4th Avenues to the Maple Street entrance east of Dove Street. A dirt access road /footpath extends the majority of the canyon's length, roughly paralleling the canyon bottom. The width of the canyon ranges from approximately 200 feet to 500 feet, and its bottom is approximately 100 to 150 feet wide. The canyon is spanned by a bridge along 1st Avenue and by the Quince Street pedestrian footbridge. The surrounding areas are developed with a mix of residential and commercial buildings.

The canyon bottom varies in elevation from approximately +225 feet above mean sea level (msl) at the northeast end to +80 feet msl at the Maple Street entrance (AECOM, undated). The canyon walls are on the order of 75 to 80 feet in height, with the lower walls typically at slope gradients of 2:1 (horizontal:vertical) or shallower. In many places, 1:5 : 1 (vertical : horizontal) fill slopes associated with the construction of existing runways and residential developments make up the upper portion of the canyon walls. Visual observations of erosion channels and gullies indicate that the bottom of the canyon is underlain by 8 feet to 10 feet of fill materials. Our observations further indicate that the fill materials contain abundant trash materials and occasional oversized construction debris.

Existing 12-inch to 18-inch diameter corrugated metal storm drain pipes empty onto the side walls of the canyon from adjoining streets at approximately 14 locations around the canyon rim. Erosion gullies locally exceeding 8 feet in height have developed downstream of some of these storm drain pipes. There is visual evidence that gullying has occurred over many years due to urban street runoff into the canyon. A catch basin is present in the canyon bottom near the Maple Street entrance. Visual observations in areas where the storm drain pipes are exposed, including within the erosion gullies, indicate that some of the pipes are heavily corroded.

Based on a review of the project plans prepared by AECOM (undated) it is our understanding that the scope of the proposed project will include the following:

Sytem 1A: Two curb inlets at Brant Street, south of Branson Place, a cleanout structure, a concrete energy dissipator at the bottom of the canyon, and approximately 225 feet of 24-inch diameter RCP pipeline. Pipe cover ranges from 2 feet to 10 feet.

System 2A: One curb inlet at the instersection of Albatross Street and Olive Street, 3 cleanout structures, and a concrete energy dissipator at the bottom of the canyon, and approximately 228 feet of 18-inch diameter RCP pipeline. Pipe cover ranges from 2 feet to 14 feet.

System 3A: One curb inlet on 2nd Avenue, 2 cleanout structures, and a concrete energy dissipator at the bottom of the canyon, and approximately 160 feet of 18-inch diameter RCP pipeline. Pipe cover ranges from 2 feet to 8 feet.

System 04: A connection to an existing inlet at Quince Street, 3 cleanout structures, and a concrete energy dissipator at the bottom of the canyon, and approximately 90 feet of 18-inch diameter RCP pipeline. Pipe cover ranges from 2 feet to 8 feet.

System 05: A catch basin on 3rd Avenue, a cleanout structure, a concrete energy dissipator at the bottom of the canyon, and approximately 136 feet of 18-inch diameter RCP pipeline. Pipe cover ranges from 2 feet to 9 feet.

System 6A & 6B: Three curb inlets on Redwood Street, 3 cleanout structures, a concrete energy dissipator at the bottom of the canyon, and approximately 240 feet of 42-inch diameter RCP pipeline. Pipe cover ranges from 2 feet to 9 feet.

System 7A & 7B: Two curb inlets on Quince Street, two cleanout structures, a concrete energy dissipator at the bottom of the canyon, and approximately 210 feet of 18-inch diameter RCP pipeline. Pipe cover ranges from 2 feet to 12 feet.

System 09: Two catch basins, a cleanout structure and approximately 92 feet of 18-inch diameter RCP pipeline in the vicinity of Palm Street and 3rd Avenue. Pipe cover ranges from 4 feet to 8 feet.

System 10: Two curb inlets from Palm Street and 3rd Avenue, 5 cleanout structures, a concrete energy dissipator at the bottom of the canyon and approximately 350 feet of 18-inch diameter RCP pipeline. Pipe cover ranges from 2 feet to 16 feet.

System 11: A connection to an existing 18-inch diameter RCP pipeline, 2 cleanout structures, a concrete energy dissipator at the bottom of the canyon and approximately 100 feet of 18-inch diameter RCP pipelines. Pipe cover ranges from 2 feet to 6 feet.

System 12: A curb inlet at 1st Avenue, 3 cleanout structures, a concrete energy dissipator at the bottom of the canyon and approximately 176 feet of 18-inch diameter RCP pipelines. Pipe cover ranges from 2 feet to 16 feet.

System 13: Four curb inlets at 1st Avenue and Nutmeg Street, 6 cleanout structures, a concrete energy dissipator at the bottom of the canyon and approximately 620 feet of 18-, 24-, 36- and 42-inch diameter RCP pipelines. Pipe cover ranges from 2 feet to 12 feet.

System 14: Two curb inlets at Front Street, two cleanout structures, and connection to System 13 at approximate Station 101+17.

System 15: A curb inlet at the intersection of Albatross Street and Maple Street, 4 cleanout structures, a concrete energy dissipator at the bottom of the canyon and approximately 200 feet of 18-inch diameter RCP. Pipe cover ranges from 2 feet to 11 feet.

Although not shown on the plans, it is our understanding that the proposed improvements will also include slope reconstruction to repair erosion gullies caused by storm water run off.

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 currently proposed project. 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 the preliminary project plans, as-built utility maps, topographic maps, published geologic literature and maps, and AGE's in-house references and aerial photographs.

3.2 Geotechnical Field Exploration

The field exploration program for this project was performed during the period between July 7 and 15, 2014. A total of fifteen (15) hand-excavated test pits were performed at the approximate locations shown on Figure 1. The test pits were advanced to depths ranging from 5.5 feet to 11 feet below the existing ground surface (bgs). Logs of the test pits are presented in Appendix A.

Prior to commencement of the field exploration activities, several site reconnaissance visits were performed to observe existing conditions and to select suitable locations for the test pits. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed test pit locations with respect to existing buried utilities. Existing buried utilities in the vicinity of the project alignments are limited to potable water and sanitary sewer pipelines, and the existing storm drains. AECOM obtained DS 511 Review permit from the City of San Diego for the performance of the test pits.

3.3 Laboratory Testing

Selected soil samples obtained from the test pits 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 included: in-place density and moisture content, maximum density and optimum moisture content, sieve (wash) analysis, and shear strength. In addition, representative samples of the onsite soil materials were collected and delivered to Clarkson Laboratories and Supply, Inc. for chemical (analytical) testing to determine soil pH and resistivity, soluble sulfate and chloride concentrations, and bicarbonate content. A brief description of the tests that were performed and the final test results are presented in Appendix B.

4.0 GEOLOGIC CONDITIONS

4.1 Geologic Setting and Site Physiography

Maple Canyon is located in the western portion of the San Diego Embayment, a deep sedimentary-filled basin which is underlain at depth by a basement rock complex of Cretaceous age batholithic and metavolcanic and metasedimentary rocks of Jurassic age. The sedimentary formations consist of nearly flat-lying to gently southwest dipping, marine and non-marine sediments which range from Cretaceous to Holocene in age. Man-made fills occur in various locations within the canyon.

Maple Canyon is incised into the Linda Vista Terrace, a 10-kilometer wide wave-cut platform situated in the mid-portion of the San Diego Embayment, between the communities of Mira Mesa to the north and North Park to the south. Capping the terrace is the Very Old Paralic Deposit, also referred to as the Lindavista Formation (Kennedy, 1975, Kennedy and Tan, 2008). These deposits consist of a relatively thin marine and non-marine sandstone and conglomerate with a characteristic reddish brown color due to iron-oxide cement. The combination of strong cementation and locally abundant gravels and cobbles pose difficult excavation conditions even for heavy duty construction equipment. Fossil assemblages in the marine portion of the deposit suggest an early Pleistocene to late Pliocene age.

The Very Old Paralic Deposits are underlain by the Pliocene age San Diego Formation, which is generally composed of a fine to medium grained, friable marine sandstone with localized calcium carbonate-cemented zones and fossiliferous beds (Kennedy, 1975). The formation may also contain localized cobble-conglomerate layers and occasional thin beds of bentonite, marl, and brown mudstone.

None of our test pits penetrated through the San Diego Formation, which extends to a reported depth of at least -64 feet msl at the location of an abandoned water well in the south portion of Balboa Park (Demere, 2012). A dip of approximately 4 degrees to the southwest was measured in the San Diego Formation at the well location. The published geologic map (Kennedy, 1975) indicates a 3 degree southwesterly dip in the San Diego Formation in Maple Canyon.

4.2 Tectonic Setting

Tectonically, the San Diego region is situated in a broad zone of northwest-trending, predominantly right-slip faults that span the width of the Peninsular Ranges and extend offshore into the California Continental Borderland Province west of California and northern Baja California. At the latitude of San Diego, this zone extends from the San Clemente fault zone, located approximately 60 miles to the west, and the San Andreas fault located about 95 miles to the east.

Major active regional faults of tectonic significance include the Coronado Bank, San Diego Trough, San Clemente, and Newport Inglewood/Rose Canyon fault zones which are located offshore; the faults in Baja California, including the San Miguel-Vallecitos and Agua Blanca fault zones; and the faults located further to the east in Imperial Valley which include the Elsinore, San Jacinto and San Andreas fault zones. Maple Canyon is not located across or near any known (mapped) active or potentially active faults.

4.3 Geologic Units

Based on their origin and compositional characteristics, the soil types encountered in the test pit explorations can be categorized into three geologic units which include (in order of increasing age): Fill materials; Young Colluvial Deposits; and San Diego Formation. A brief description of each unit is presented below.

4.3.1 Fill Materials (Qaf)

Fill materials were encountered in test pits TP-1, TP-3 and 4, TP-6 thru 8, TP-11 and 12, and TP-14 and 15. Where encountered, the fill materials varied from approximately 1-foot in thickness to greater than the maximum depth of exploration. The fill soils generally consist of dry to damp, silty to slightly clayey sand with localized gravels/cobbles.

Construction debris including concrete, wood, asphalt and brick was encountered within the fill materials. In test pit TP-12 a buried concrete slab exceeding 2.5 feet in length and width was encountered at a depth of 5.5 feet bgs. Since we were unable to extend the depth of the test pit, a second test pit, TP-12A, was excavated approximately 20 feet northeast of TP-12. Documentation pertaining to the fill placement is unavailable.

4.3.2 Young Colluvial Deposits (Qyc)

Young Colluvial Deposits of Holocene to late Quaternary age (Kennedy and Tan, 2008) were encountered in TP-2, TP-4 thru 6, TP-9 and 10, TP-13, and TP-15. The deposits are of limited areal extent in Maple Canyon, and are not depicted on the referenced geologic map.

In TP-15 the Young Colluvial Deposits were found to extend to the maximum depth of exploration, with the deposits extending to a maximum depth of 5 feet bgs in the remaining test pit locations. The deposits are composed of dry to damp, medium dense to dense reddish to yellow brown silty sand with locally abundant gravels and cobbles. The soil varies from uncemented to moderately cemented.

4.3.3 Very Old Paralic Deposits (Qvop)

Although not encountered in our test pits, Very Old Paralic Deposits are mapped on top of the mesa and encountered at eight of twelve percolation test borings performed around the rim of Maple Canyon during our previous investigation (AGE, 2013).These materials generally consisted of yellow brown to reddish brown sandstone and conglomerate. Drilling refusal was encountered on dense conglomerate in some of the test holes. The deposits occur as a relatively thin cap on the Linda Vista Terrace, and are exposed in outcrops near the canyon rim.

4.3.4 San Diego Formation (Tsd)

The mid to late Pliocene age San Diego Formation was encountered below the fill materials and Young Colluvial Deposits in all of the test pit locations with the exception of TP-1, TP-14 and TP-15. TP-1 and TP-14 were terminated within fill materials, and TP-15 was terminated within the Young Colluvial Deposits.

In our test pits the San Diego Formation consisted of damp, pale olive to olive gray, medium dense/stiff, massive, fine to very fine-grained silty sandstone and sandy siltstone. Where encountered, the formation extended to the maximum depth of exploration.

4.4 Groundwater

No groundwater or seepage was encountered in any of our test pit excavations. Based on the elevation of the bottom of the canyon, the depth to groundwater is anticipated to be well below the bottom of the canyon. Localized shallow perched groundwater and storm water run off can be expected to occur, during the wet (rainy) season. Seasonally shallow groundwater should also be expected in the vicinity of the catch basin near the southwest end of the canyon.

5.0 DISCUSSIONS, OPINIONS AND RECOMMENDATIONS

Based on the results of our study, it is our opinion that the design and construction of the conceptual slope repair as proposed are feasible, provided the recommendations presented in this section are followed.

5.1 Potential Geologic and Seismic Hazards

Geologic hazards are those hazards that could impact a site due to local and surrounding area geologic and seismic conditions. Seismic hazards include phenomena that occur during an earthquake such as ground shaking, surface fault rupture, liquefaction, differential seismic-induced settlement, lateral spread displacement, ground lurching, tsunami or seiches, and seismic-induced flooding. Geologic hazards also include subsidence, landslides, and adverse soil conditions (expansive or collapsible soil). The potential impact of these hazards to the site has been assessed and is summarized in the following sections.

5.1.1 Faulting

The site is not located within a currently designated California Geological Survey (CGS) Earthquake Fault Special Study Zone. The nearest mapped active faults to the project site are the Rose Canyon fault zone San Diego Section, Silver Strand section-Silver Stran Fault and Silver Strand section-Downtown Graben fault which are located approximately 0.7 miles, 1.68 miles and 1.71 miles from the project site, respectively. A summary of the fault parameters is shown in Table 1 on the next page.

Table 1

Summary of Fault Parameters

	Rose Canyon Fault Zone (San Diego Section)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup	0.7 km
Rjb	0.7 km
Rx	0.7 km
Fnorm	0
Frev	0

	Rose Canyon fault zone (Silver Strand section-Silverstrand fault)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup	1.68 km
Rjb	1.68 km
Rx	1.68 km
Fnorm	0
Frev	0

	Rose Canyon fault zone (Silver Strand section-Downtown Graben fault)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup	1.71 km
Rjb	1.71 km
Rx	0.81 km
Fnorm	0
Frev	0

*** Definition of Terms in Table 1**

Rrup - Closest distance (km) to the fault rupture plane.

Rjb - Joyner-Boore distance - The shortest horizontal distance to the surface projection of the rupture area. Think of this as the nearest horizontal distance to the area directly overlying the fault. RJB is zero if the site is located within that area.

Rx - Horizontal distance to the fault trace or surface projection of the top of rupture plane. It is measured perpendicular to the fault (or the fictitious extension of the fault).

Fnorm - Fault normal

Frev - Fault reverse

The Rose Canyon fault zone (RCFZ) represents the most significant source of seismic hazard in the San Diego metropolitan area. Geologic studies performed on the RCFZ in the Rose Creek and downtown San Diego areas have discovered evidence of fault displacement in Holocene age alluvial and colluvial deposits (Patterson, 1986; Rockwell, 1991; Woodward-Clyde Consultants, 1994; Lindvall and Rockwell, 1995). Based on the results of these studies, the Rose Creek and downtown segments of the RCFZ have been classified as "active" and are designated as Alquist-Priolo Earthquake Fault Zones by the State of California. The project site is not located within the Alquist-Priolo Earthquake Fault Zones.

In the San Diego Bay area, the RCFZ is believed to splay into multiple, relatively closely spaced, subparallel strands; the most prominent of which are the Silver Strand, Coronado and Spanish Bight faults. Other, shorter and less distinctive unnamed faults are mapped within the western portion of San Diego Bay. Recent earthquakes, centered in San Diego Bay, indicates the potential for seismic activity along any of these smaller faults exists.

The project site is subject to moderate to severe ground shaking in response to a major earthquake occurring on the RCFZ or on one of the major regional active faults. The closest active regional faults to the site with recurring magnitude 4.0 and greater earthquakes are the Coronado Bank, the Vallecitos-San Miguel, and the Elsinore fault zones. Other more distant, active regional faults that are considered potential sources of seismic activity include the offshore located San Diego Trough and San Clemente fault zones and some of the faults in Imperial Valley which include the San Jacinto and San Andreas fault zones.

The location of the site in relation to the active faults in the region is shown on the Regional Fault Map (Figure 2). The computer program EQFAULT (Blake, 2010) was used to approximate the distance of known faults to the site. Seven known active faults are identified within a search radius of 50 miles from the site. A summary of seismic source characteristics for faults that present the most significant seismic hazard potential to the project area is presented in Table 2 on the next page.

Table 2

Summary of Seismic Source Characteristics

Fault	Maximum Magnitude (Mw)	Peak Site Acceleration (g)	Closest Distance to Site (miles)
Rose Canyon	6.8	0.493	0.43
Coronado Bank	7.6	0.226	13.5
Newport - Inglewood (offshore)	7.5	0.068	33.0
Elsinore - Julian	7.6	0.063	40.8
Elsinore - Temecula	7.6	0.046	45.4
Earthquake Valley	6.6	0.037	49.3
Elsinore - Coyote Mountain	7.6	0.042	49.3

5.1.2 Historical Seismicity

EQSEARCH is a program that performs automated searches of a catalog of historical Southern California earthquakes. As the program searches the catalog, it computes and prints the epicentral distance from a selected site to each of the earthquakes within a specified radius (100 kilometers). From the computed distance, the program also estimates (using an appropriate attenuation relation) the peak horizontal ground acceleration that may have occurred at the site due to each earthquake.

Based on the results of the geotechnical investigation, we have use an average V_{S30} of 1,000 feet/s for the project site. Based on the estimated shear wave velocities and our visual classification of the geologic units encountered in the soil borings, site Class D attenuation was used for all of our analysis. We used a combined earthquake catalog for magnitude 5.0 or larger events which occur within 100 kilometers from the site between 1800 and December 1999. The earthquake catalog for events prior to about 1933 is limited to the higher magnitude events.

The search results indicate that the nearest earthquake of magnitude 5.7 occurred on May 27, 1862 on a strand of the RCFZ which is located approximately 3.1 miles from the project site. This earthquake resulted in a calculated ground acceleration of 0.391 which is also the largest ground acceleration calculated from the search. The largest magnitude earthquake reported was a magnitude 7.0 event December 16, 1858, located 89 miles from the project site on a strand of the Fontana Fault in Ontario, California which resulted in a calculated ground acceleration of 0.036 g.

It is our opinion that the major seismic hazard affecting the project site 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 proposed facilities. For project design purposes, we recommend that the RCFZ be considered as the dominant seismic source.

5.1.3 Seismic Design Parameter

For design purposes, we recommend that a Peak Ground Acceleration (PGA) of 0.54g be used for the project site. The PGA value presented herein was calculated using the Minimum Design Loads for Buildings and Other Structures procedures which has been adopted for the California Building Codes 2013.

5.1.4 Fault Ground Rupture

Maple Canyon is not located astride any known (mapped) active or potentially active faults (Kennedy, 1975; City of San Diego, 1995). Therefore, the potential for fault ground rupture at the site is considered insignificant.

5.1.5 Soil Liquefaction

Seismically-induced soil liquefaction is a phenomenon in which loose to medium dense, saturated granular materials undergo matrix rearrangement, develop high pore water pressure, and lose shear strength due to cyclic ground vibrations induced by earthquakes or other means.

The project site is underlain by very dense to hard formational material which is not considered susceptible to seismic-induced soil liquefaction or ground settlement. Furthermore, a review of the State of California Seismic Hazard Zones (2009) indicates that the site is not located within an area that is considered susceptible to soil liquefaction during a seismic event.

5.1.6 Lateral Spread Displacement

The project site has very low susceptibility to liquefaction, therefore, the risk of lateral spread displacement during a seismic event is considered remote.

5.1.7 Differential Seismic-Induced Settlement

Differential seismic settlement occurs when seismic shaking causes one type of soil to settle more than another type. It may also occur within a soil deposit with largely homogeneous properties if the seismic shaking is uneven due to variable geometry or thickness of soil deposit. Based on our investigation, the subsurface soils are found to be fairly uniform throughout the site; therefore, the potential of differential settlement is considered low.

5.1.8 Ground Lurching

Ground lurching is permanent displacement or shift of the ground in response to seismic shaking. Ground lurching occurs in areas with high topographic relief, and usually occurs near the source of an earthquake. These displacement can results in permanent cracks in the ground surface. Considering the distance from the project site to the nearest potential source of seismic event, it is our opinion that ground lurching does not present a potential hazard for the proposed project.

5.1.9 Landslides

A review of the pertinent geologic map indicates that the project site is not located on or below any known (mapped) ancient landslides (Kennedy, 1975; City of San Diego, 1995). Furthermore, a review of the State of California Seismic Hazard Zones (2009) indicates that the site is not located in an area that is susceptible to landslide hazards.

5.1.10 Other Seismic-induced Hazards

The elevation of the site and its distance from the coastal area preclude the potential of property damage from a seismic-induced tsunami and or seiches.

5.2 Soil Corrosivity

Soil is generally considered corrosive to concrete if its chloride concentration is greater than 500 parts per million (ppm) or sulfate concentration is greater than 2,000 ppm, or if the pH is 5.5 or less.

Analytical testing was performed on fifteen samples collected from the test pits to determine pH, resistivity, soluble sulfate, chlorides and bicarbonates content. The tests were performed in accordance with California Test Method Nos. 643, 417 and 422. The results which are summarized in Table 3 on the next page indicates that the majority of the onsite soil is not considered corrosive against concrete. Copies of the analytical laboratory test data reports are included in Appendix B.

AGE does not practice in the field of corrosion engineering. In the event that corrosion sensitive facilities are planned, we recommend that a corrosion engineer be retained to perform the necessary corrosion protection evaluation and design.

Table 3

Summary of Corrosivity Test Results

Sample No	pH	Resistivity (ohm-cm)	Soluble Sulfate (ppm)	Soluble Chloride (ppm)	Bicarbonates (ppm)
TP-1 @ 6'-7'	9.0	1100	72	64	49
TP-2 @ 6.5'-7'	9.1	1100	140	75	56
TP-3 @6.5'-7'	7.9	290	1890	470	44
TP-4 @6.5'-7'	8.2	3500	15	21	34
TP-5 @6'-6.5'	7.6	4200	<11	21	26
TP-6 @6.5'-7'	7.9	1300	54	96	23
TP-7 @6'-6.5'	8.1	370	670	530	36
TP-8 @8.5'-9'	8.5	1000	100	75	38
TP-9 @6'-6.5'	7.7	310	170	1070	20
TP-10 @6'-6.5'	8.8	470	150	430	38
TP-11 @6.5'-7'	7.8	440	230	690	31
TP-12 @6'-6.5'	8.5	410	420	300	40
TP-13 @ 6'-7'	8.2	1900	110	120	50
TP-14 @ 7.5'-8'	8.5	480	420	530	130
TP-15 @6'-6.5'	6.8	1500	36	32	20

5.3 Expansive Soil

Based on visual observations, it is our opinion that the majority of on-site materials are considered non-expansive or to have a low expansion potential.

5.4 Cut-and-Cover Construction

Since no changes to the existing ground surface along the cut-and-cover segment of the proposed storm drain 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 storm drain 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. The depth of the overexcavation should be determined during construction by the Resident Geotechnical Engineer.

5.4.1 Soil and Excavation Characteristics

The soil materials within the anticipated depths of the excavation for the proposed alignments consist of man-made fill, Young Colluvial Deposits, Very Old Paralic Deposits and San Diego Formation which can generally be readily excavated with conventional heavy-duty construction equipment. It must be noted that the Very Old Paralic Deposits are known to contain localized highly cemented zones which may pose difficult excavation conditions with conventional equipment. Although not as common, the same conditions have also been encountered within the San Diego Formation.

The majority of the soil materials encountered in the test pits 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 6 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.

Materials generated from excavations made in the Very Old Paralic Deposits and Young Colluvial Deposits are likely to contain abundant gravels and cobbles. The San Diego Formation is also known to contain localized lenses of gravels and cobbles. If these materials are used as fill materials for this project, the contractor may need to employ selective screening methods to remove large (in excess of 6 inches in maximum dimension) rock clasts prior to use and placement as compacted structural and/or trench backfill.

5.4.2 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.

5.4.3 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.

5.4.4 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. The maximum dry density and optimum moisture content of the backfill materials should be determined in the laboratory in accordance with the ASTM D1557 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.

5.4.5 Concrete Anchor/Cutoff Wall

We recommend that for segments of the proposed storm drain which will be installed at a slope of 3 : 1 (horizontal : vertical), or steeper, concrete anchors and/or cutoff walls be used to provide support for both the storm drain pipe and the trench backfill. Concrete anchor and/or cutoff wall may be designed in accordance with Drawing Numbers SDS-114 or SDS-115 of the City of San Diego - Standard Drawings for Public Works Construction. Based on the slope gradient, subsurface conditions, and depth of excavation, when and if it is necessary, we recommend that the anchor and/or cutoff wall be installed at approximate 30-foot intervals.

5.5 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.

5.5.1 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 5.4.4 of this report.

5.5.2 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 3,500 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 250 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 2,500 psf. A coefficient of friction of 0.45 may be used for foundation cast directly on competent native soils or approved compacted materials.

5.5.3 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.

5.6 Slope Repair

Prior to the slope repair operations, the surface of the existing erosion gully and the adjacent slope face will need to be cleared of all vegetation and trash or other deleterious materials. Any loose and compressible soil materials shall be removed until competent materials are encountered. The extent of the cleared and excavated areas should be checked and determined in the field by the City's Resident Engineer during the grading operation.

The subsurface conditions encountered in the test pits indicate that the toe of the proposed backfill for the slope repair will be in fill or Young Colluvium. We recommend that a key be constructed at the toe of the proposed slope backfill. The key should consists of a minimum 10 feet wide by 2-foot deep bench which should be excavated into the slope with a minimum inclination of 2 percent.

Additional horizontal benches shall be cut into the existing slope in order to provide both lateral and vertical stability for the new fill materials. The purpose of the benches is to provide a horizontal base so that each layer is placed and compacted on a horizontal plane. The width and frequency of all benches should be determined in the field during the earthwork operation based on the actual soil conditions and the gradient of the existing slope.

Placement and compaction of backfill materials for the slope repair should be performed in accordance with the recommendations presented in Section 5.4.4 of this report. Backfill material 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.

5.7 Trenchless Construction

It is our understanding that portions of the pipeline alignments may be installed using trenchless excavation methods. Based on the results of our investigation, we recommend that these methods not be employed in areas where the pipelines extend through the fill materials which were found to contain abundant oversized demolition (concrete) debris. The majority of the fill materials were found along the rim and at the bottom of the canyon. Depending on the selected method, trenchless excavation may also encounter difficulties in the areas that are underlain by Young Colluvial and/or Very Old Paralic Deposits which are locally cemented and contain large (6 inches or larger) cobbles. Trenchless excavation in the San Diego Formation is not considered to pose any problems.

6.0 CONSTRUCTION-RELATED CONSIDERATIONS

6.1 Construction Dewatering

Groundwater was not encountered in any of the test pit explorations at the time of our field investigation. The depth of the local groundwater table is expected to be well below the proposed pipe invert depths. We therefore do not anticipate the need for dewatering of the trenched excavations made during construction. The contractor should, however, anticipate the possible need for sump pumps in the event that localized perched water conditions are encountered during construction. The design, installation, and operation of any construction dewatering measures necessary for the project shall be the sole responsibility of the contractor.

6.2 Temporary Shoring

Since the anticipated pipe invert depths will be more than 4 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 and the need to avoid excessive community and environmental disruption dictate that a shored excavation will be needed along the entire pipeline alignments. Design and construction of temporary shoring should be the sole responsibility of the contractor.

6.2.1 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.

It is recommended that pre- and post-construction surveys be conducted to document existing site conditions. Documentation should include photographic and video surveys of the existing facilities and site improvements, as well as field surveys of adjacent facilities and pavement structures. We further recommend that a weekly survey of existing utilities be performed during the construction phase.

6.2.2 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 and young colluvium, 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 Very Old Paralic Deposits and the San Diego Formation, 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. In the event that the bottom of the excavation is located below the groundwater level, hydrostatic pressure should be added to the lateral loads.

The recommended lateral earth pressures have been prepared based on the assumptions that the shored earth is level at the surface and that the shoring system is temporary in nature.

6.2.3 Lateral Bearing Capacity

Resistance to lateral loads will be provided by passive soil resistance. The allowable passive pressure for the fill materials, young alluvium and colluvium may be assumed to be equivalent to a fluid weighing 200 pcf. Allowable lateral bearing pressure in these material should not exceed 2,000 psf. Allowable passive pressure for the Very Old Paralic Deposits and San Diego Formation may be assumed to be equivalent to a fluid weighing 350 pcf, with maximum allowable lateral bearing pressure of 3,500 psf.

6.3 Environmental Considerations

The scope of AGE's investigation did not include the performance of a Phase I Environmental Site Assessment (Phase I ESA) to evaluate the possible presence of soil and/or groundwater contamination beneath the project storm drain alignments. A review of the Geotracker website (www.Geotracker.com) did not reveal evidence of historical unauthorized releases of hazardous materials within Maple Canyon, nor did the website reveal evidence of active Leaking Underground Storage Tank (LUST) cleanup sites in the immediate vicinity of the canyon.

In the event that hazardous or toxic materials are encountered during the construction phase, the contractor should immediately notify the City and be prepared to handle and dispose of such materials in accordance with current industry practices and applicable Local, State and Federal regulations.

7.0 GENERAL CONDITIONS

7.1 Post-Investigation Services

Post-investigation geotechnical services are an important continuation of this investigation, and we recommend that the City's Construction Inspection Division performs the necessary geotechnical observation and testing services during construction. In the event that the City is unable to perform said services, it is recommended that our firm be retained to provide the services.

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.

7.2 Uncertainties and Limitations

The information presented in this report is intended for the sole use of AECOM and other members of the project design team 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.

AGE has observed and investigated the subsurface conditions only at selected locations along the project alignments. The findings and recommendations presented in this report are based on the assumption that the subsurface conditions beneath all project alignments do not deviate substantially from those encountered in the exploratory soil borings. 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.

AGE 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.

8.0 REFERENCES

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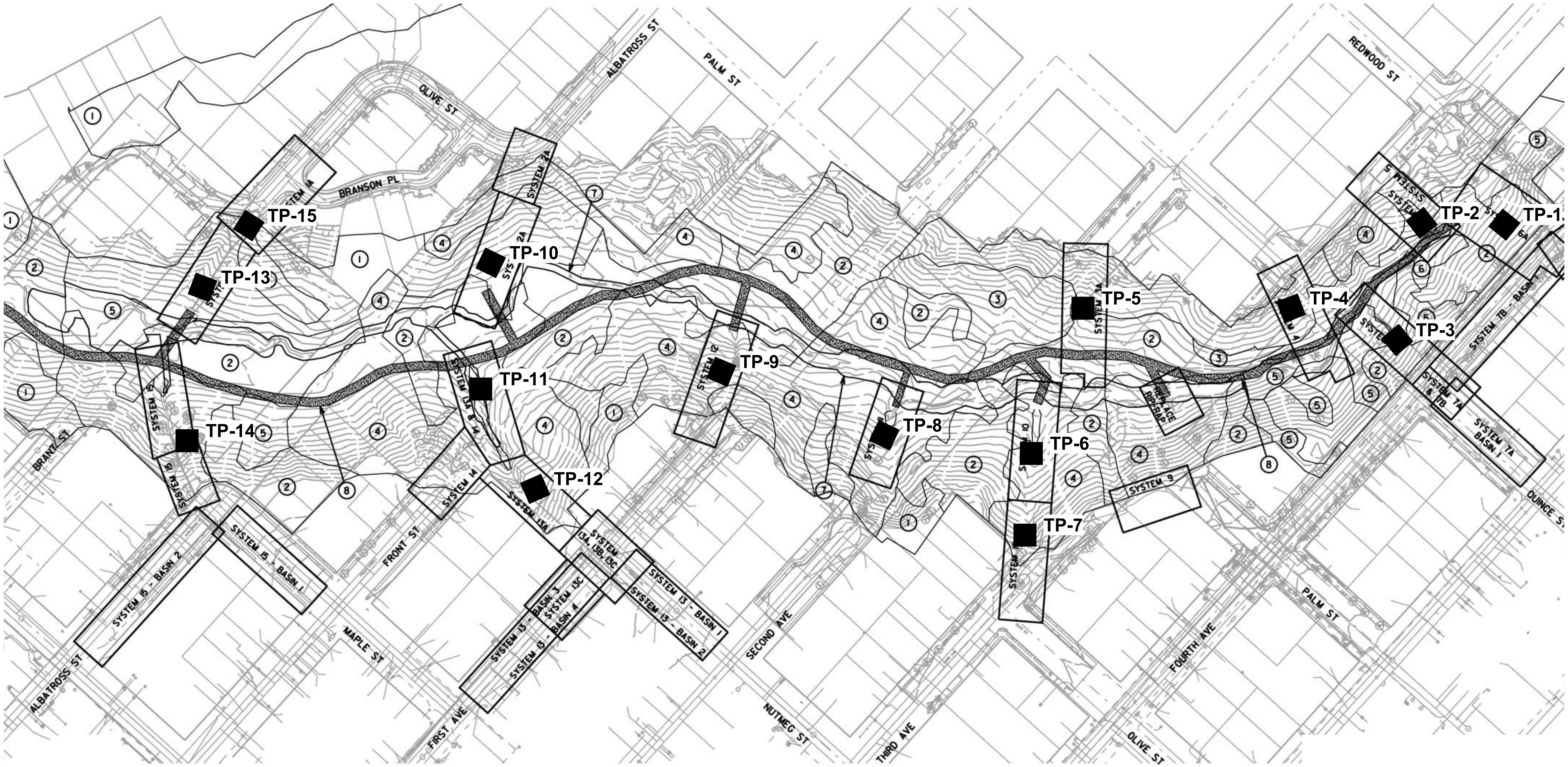
Aerial Photographs

U.S. Department of Agriculture black and white aerial photograph Nos. AXN-3M-195 and 196, (dated 1953).

FIGURES



¹ Approximate test pit locations

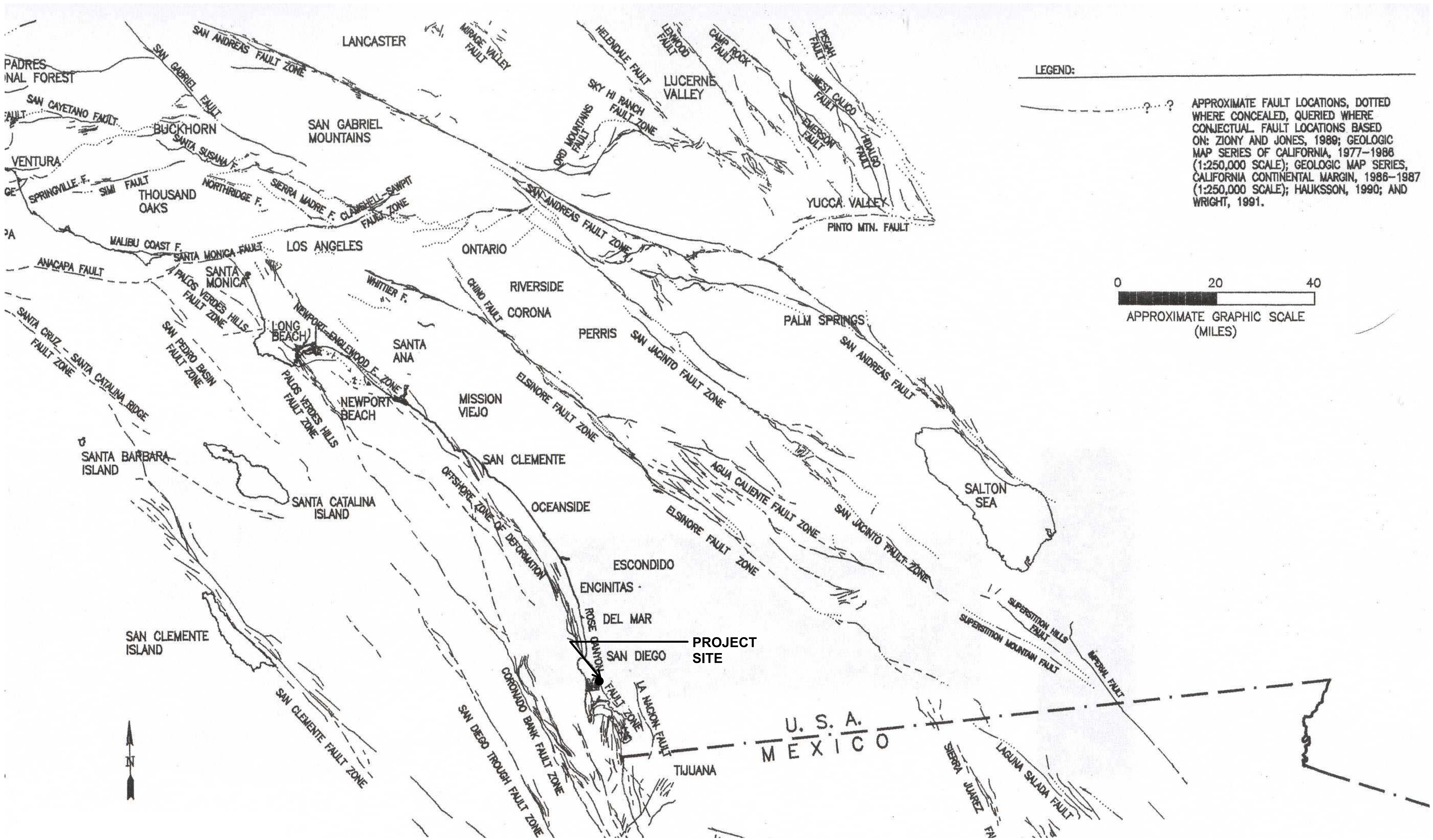
NOT TO SCALE

SITE PLAN SHOWING APPROXIMATE TEST PIT LOCATIONS
MAPLE CANYON RESTORATION PHASE 1

ALLIED GEOTECHNICAL ENGINEERS, iNC.

FIGURE 1

PROJECT NO.
60C508

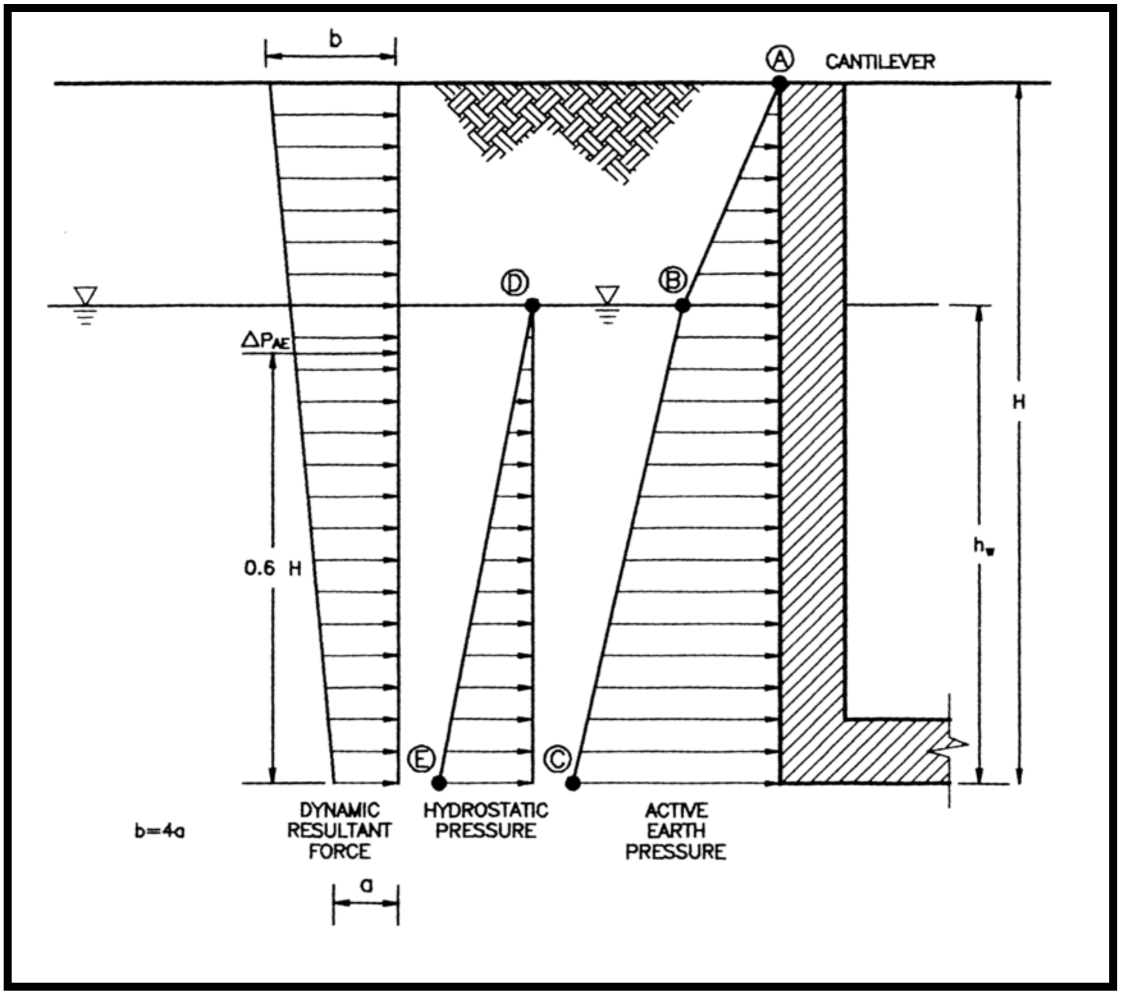


REGIONAL FAULT MAP
MAPLE CANYON RESTORATION PHASE 1

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE 2

PROJECT NO.
60C508



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

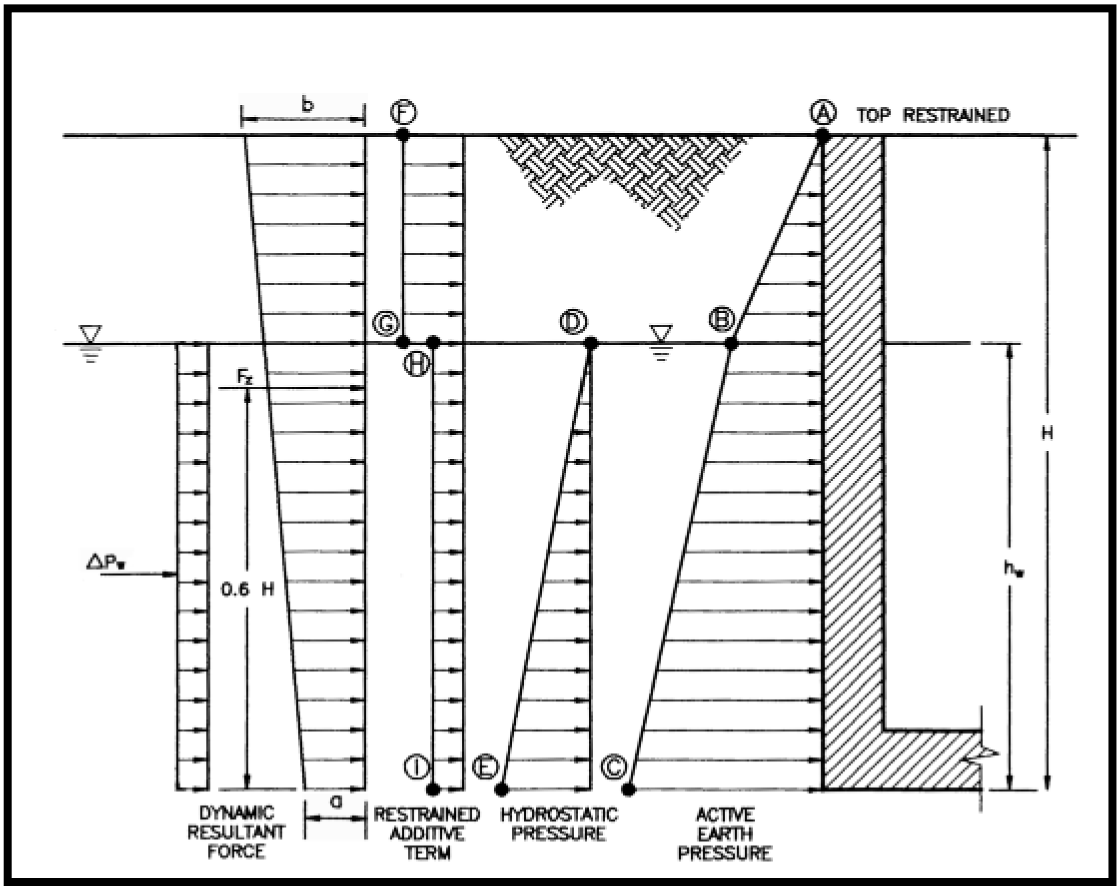
Earth Pressure
A = 0
B = 35 ($H-h_w$), psf
C = 35 ($H-h_w$) + 20h_w, psf

Hydrostatic Pressure
D = 0
E = 62.4h_w

Dynamic Resultant Force
a = 2.9H
b = 4a = 11.6H
ΔP_{AE} = 7.25H^2, lb/ft @ 0.6H

LATERAL PRESSURES FOR CANTILEVER WALLS
MAPLE CANYON RESTORATION PHASE 1

PROJECT NO. 60C508	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

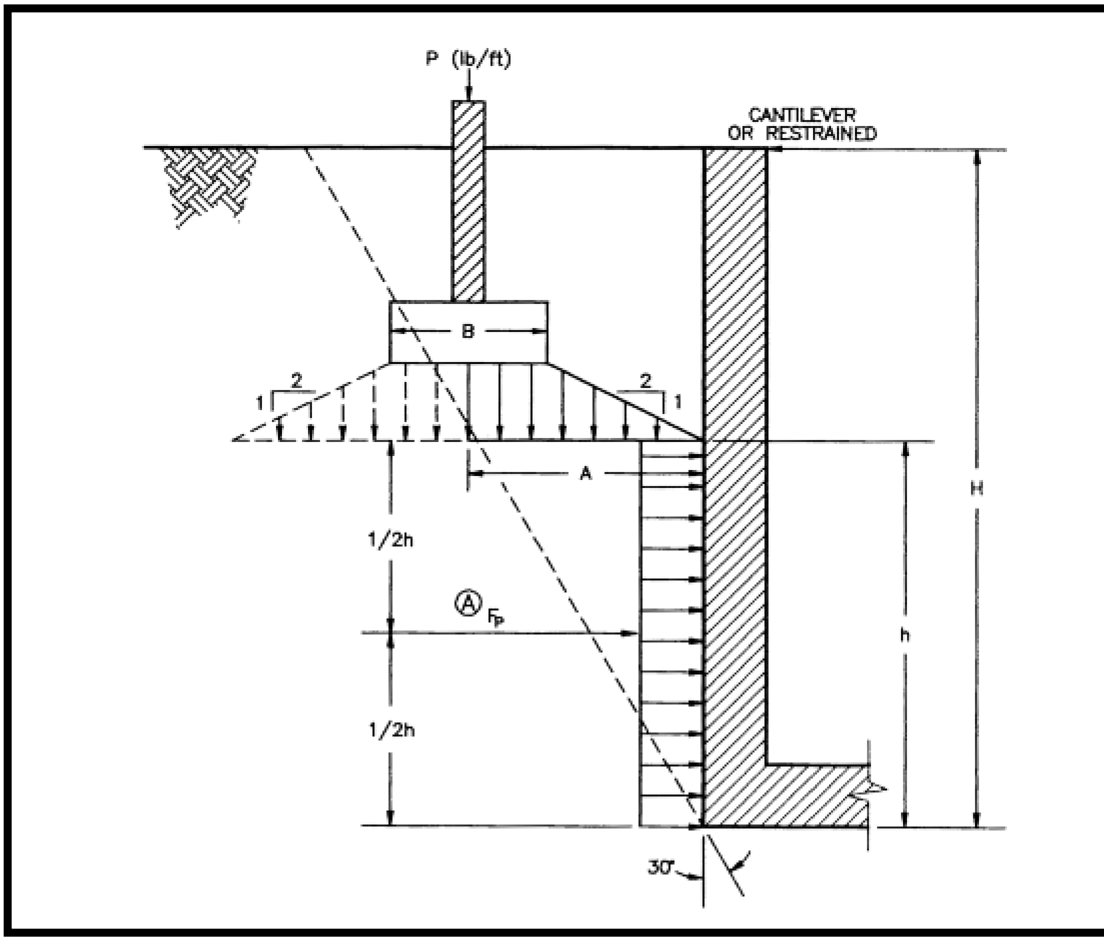
b = 4a = 39.6H

$F_z = 24.75H^2$, lb/ft @ 0.6H (Soil)

$\Delta P_w = 21.6h_w^2$, lb/ft @ $0.5h_w$ (Water)

LATERAL PRESSURES FOR RESTRAINED WALLS
MAPLE CANYON RESTORATION PHASE 1

PROJECT NO. 60C508	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

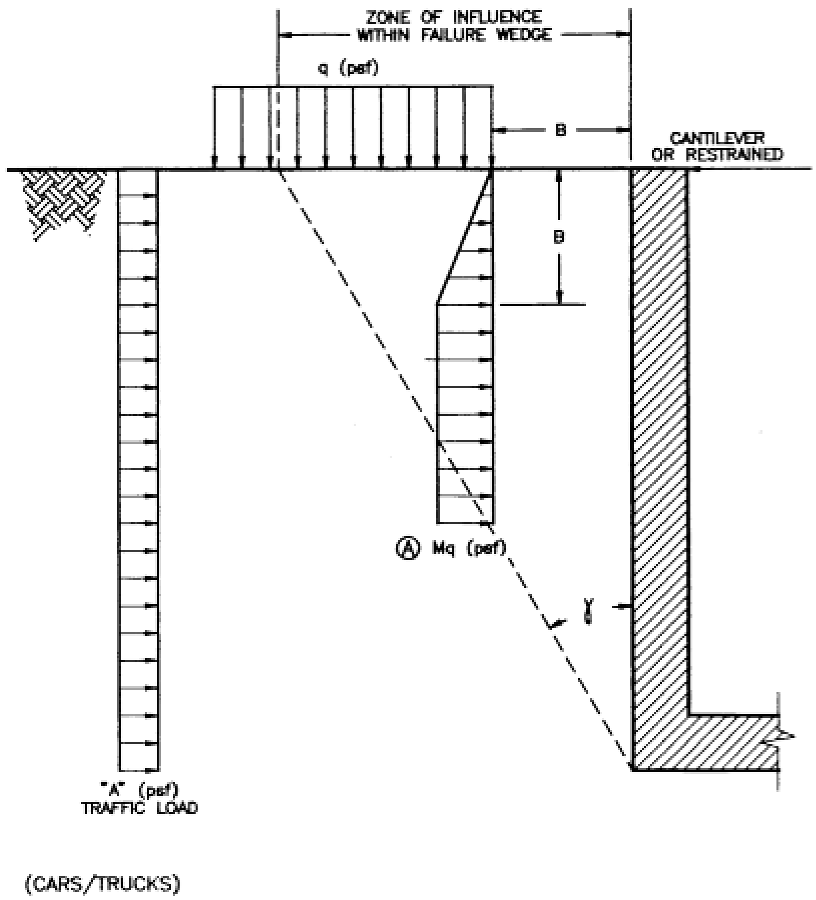
NOTES:

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
MAPLE CANYON RESTORATION PHASE 1

PROJECT NO. 60C508	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
MAPLE CANYON RESTORATION PHASE 1

PROJECT NO. 60C508	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 6

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

The field exploration program for this project was performed during the period between July 7and 15, 2014, and included the excavation of fifteen (15) soil test pits. The pits were hand dug to depths ranging from 6.5 feet to11 feet below the existing ground surface (bgs).

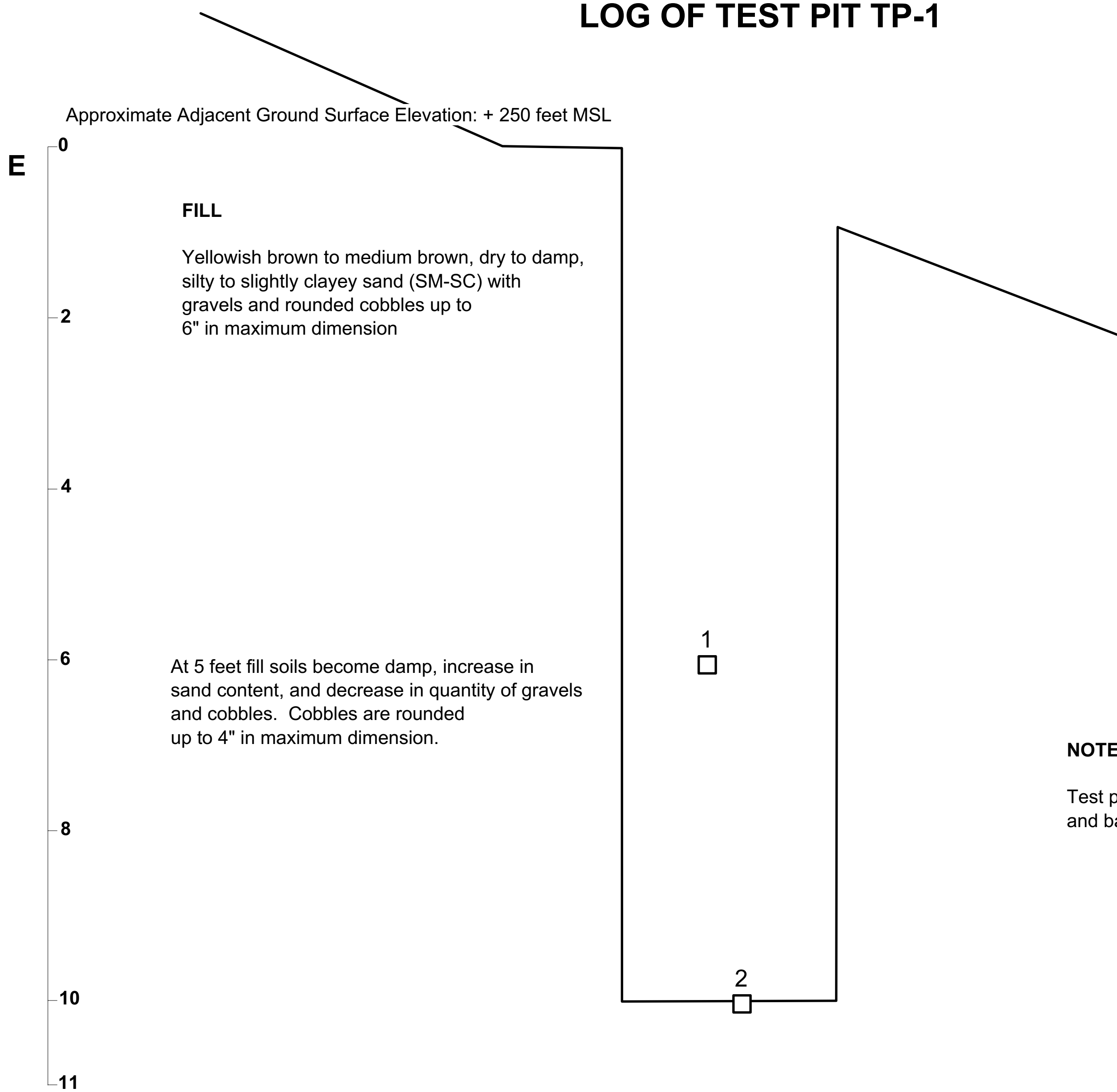
Prior to commencement of the field exploration activities, several site visits were performed to observe existing conditions and to select suitable locations for the test pits. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed test pit locations with respect to existing buried utilities.

The soils encountered in the test pits were visually classified and logged by an experienced field geologist from our firm. Representative samples of the various soil types encountered test pits were collected at selected depths for laboratory testing and analysis. The samples collected include bulk samples and relatively undisturbed chunk samples. In addition, relatively undisturbed samples were also obtained by driving a 3-inch (OD) diameter standard California sampler with a special cutting tip and inside lining of thin brass rings into the soils at the bottom of the test pit. The sampler was driven a distance of 12 inches into the soils at the bottom of the test pit with a slide hammer falling from a height of 30 inches. A 6-inch long sample of soil that was retained in the brass rings was extracted from the sampling tube and transported to our laboratory in close-fitting, waterproof containers.

The test pit logs are presented in Figures A-1 and A-15. Upon completion of the excavation, logging and sampling activities, the test pits were backfilled with excess soil cuttings.

LOG OF TEST PIT TP-1

Approximate Adjacent Ground Surface Elevation: + 250 feet MSL



FILL

Yellowish brown to medium brown, dry to damp, silty to slightly clayey sand (SM-SC) with gravels and rounded cobbles up to 6" in maximum dimension

At 5 feet fill soils become damp, increase in sand content, and decrease in quantity of gravels and cobbles. Cobbles are rounded up to 4" in maximum dimension.

<u>LEGEND</u>:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

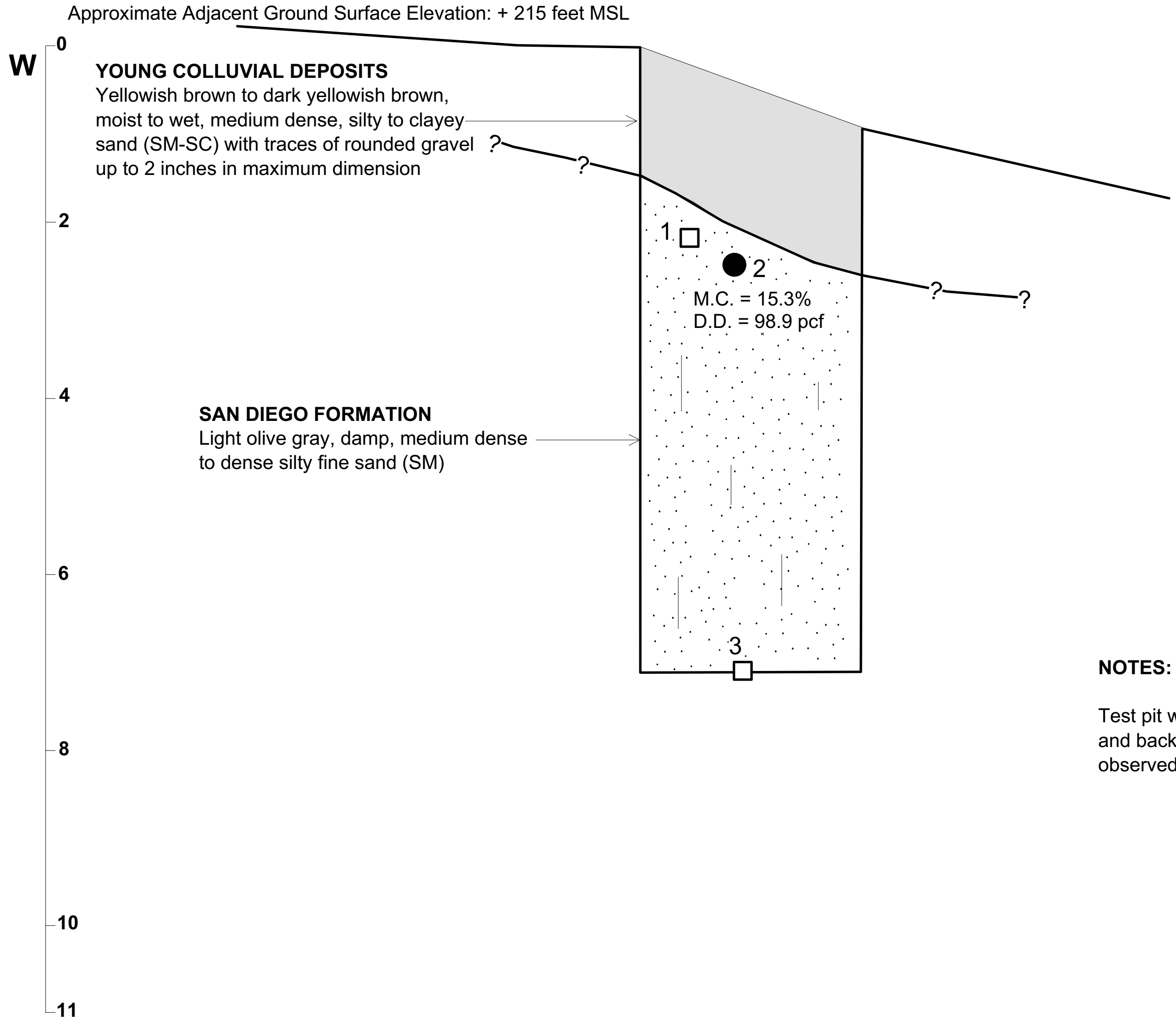
NOTES:

Test pit was excavated to a depth of 10 feet on July 14, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-1**

LOG OF TEST PIT TP-2



LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

NOTES:

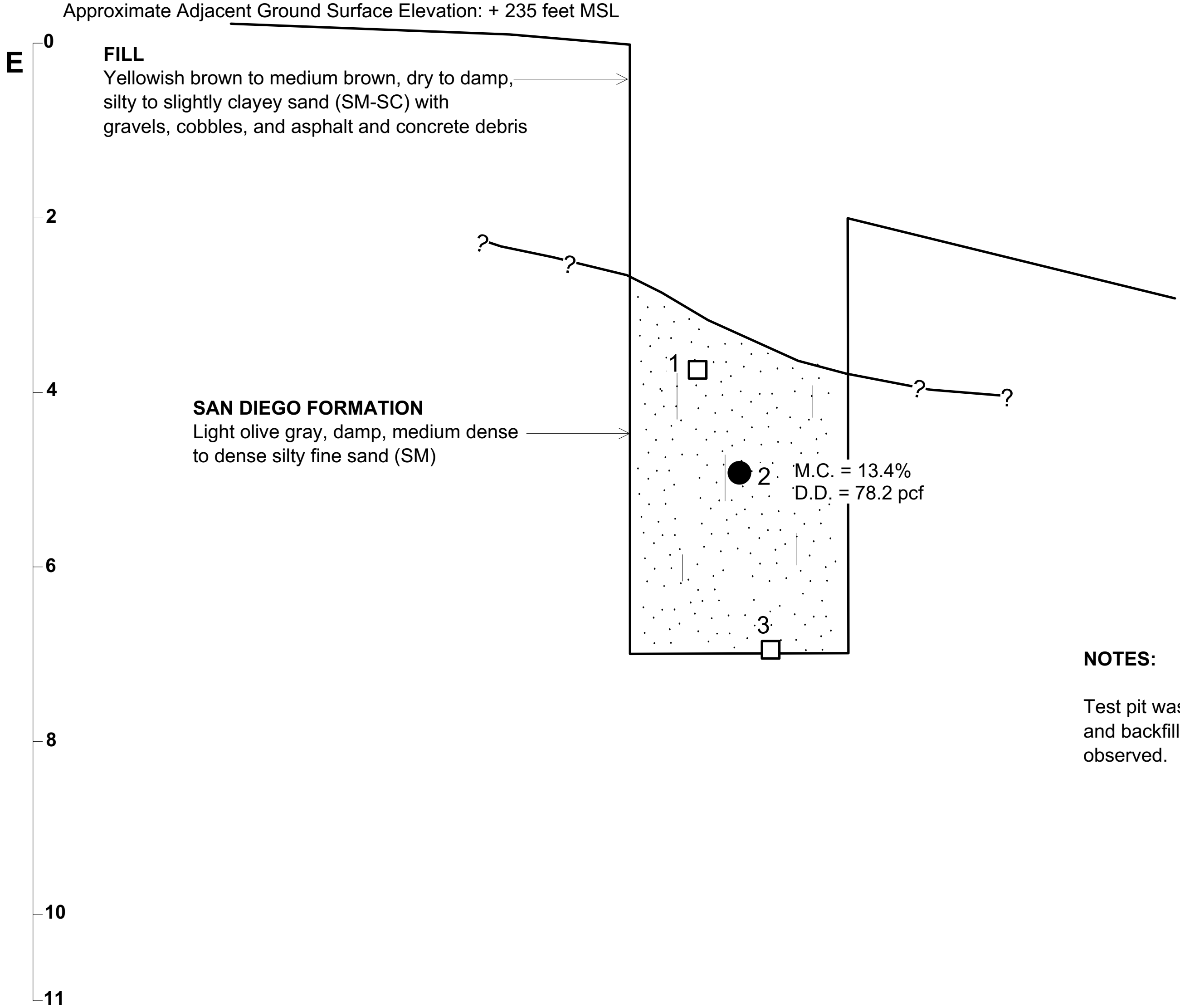
Test pit was excavated to a depth of 7 feet on July 11, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-2**

LOG OF TEST PIT TP-3

Approximate Adjacent Ground Surface Elevation: + 235 feet MSL



LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

NOTES:

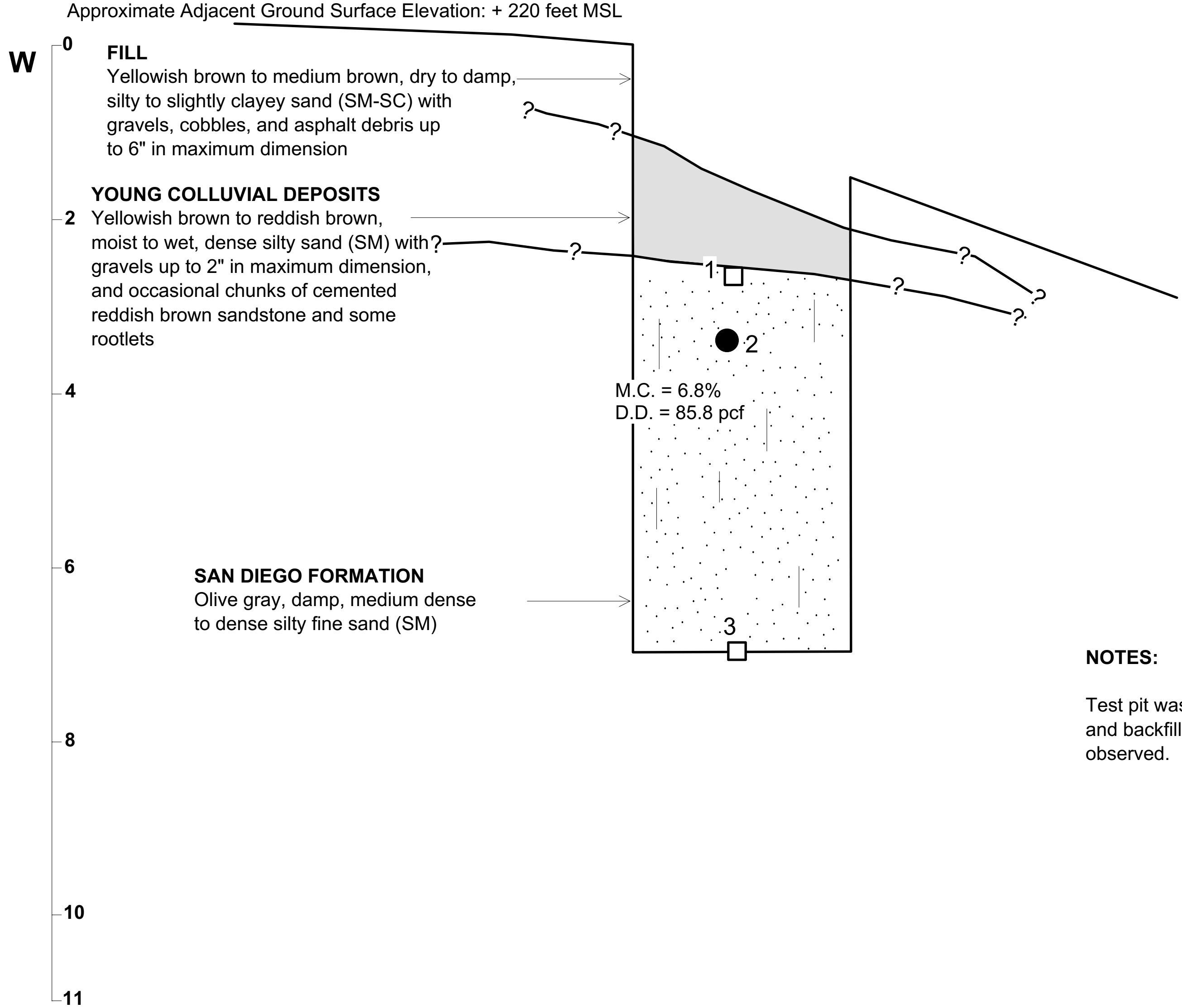
Test pit was excavated to a depth of 7 feet on July 11, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-3**

LOG OF TEST PIT TP-4

Approximate Adjacent Ground Surface Elevation: + 220 feet MSL



LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

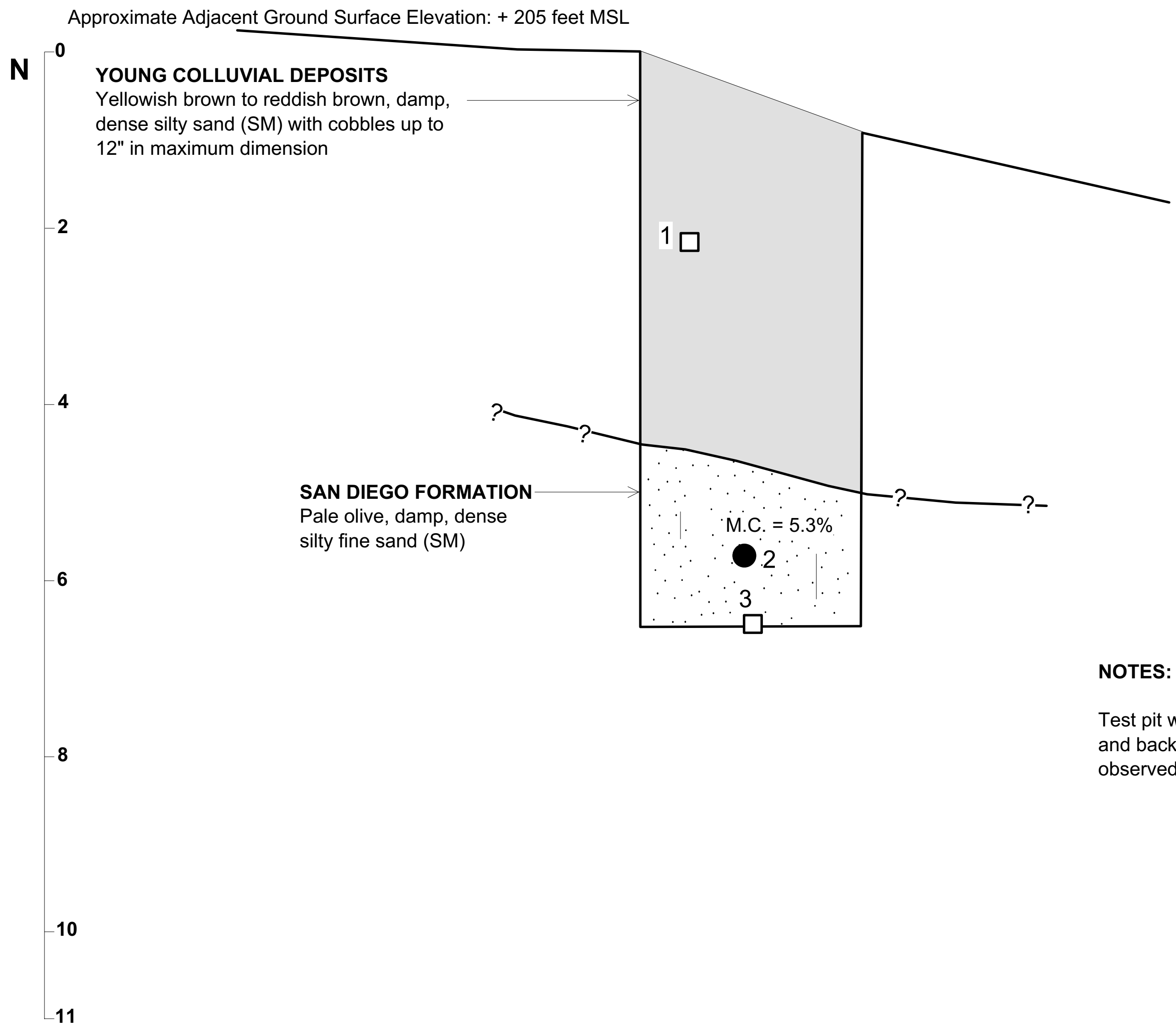
NOTES:

Test pit was excavated to a depth of 7 feet on July 11, 2014 and backfilled on the same day. No active was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-4**

LOG OF TEST PIT TP-5



LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

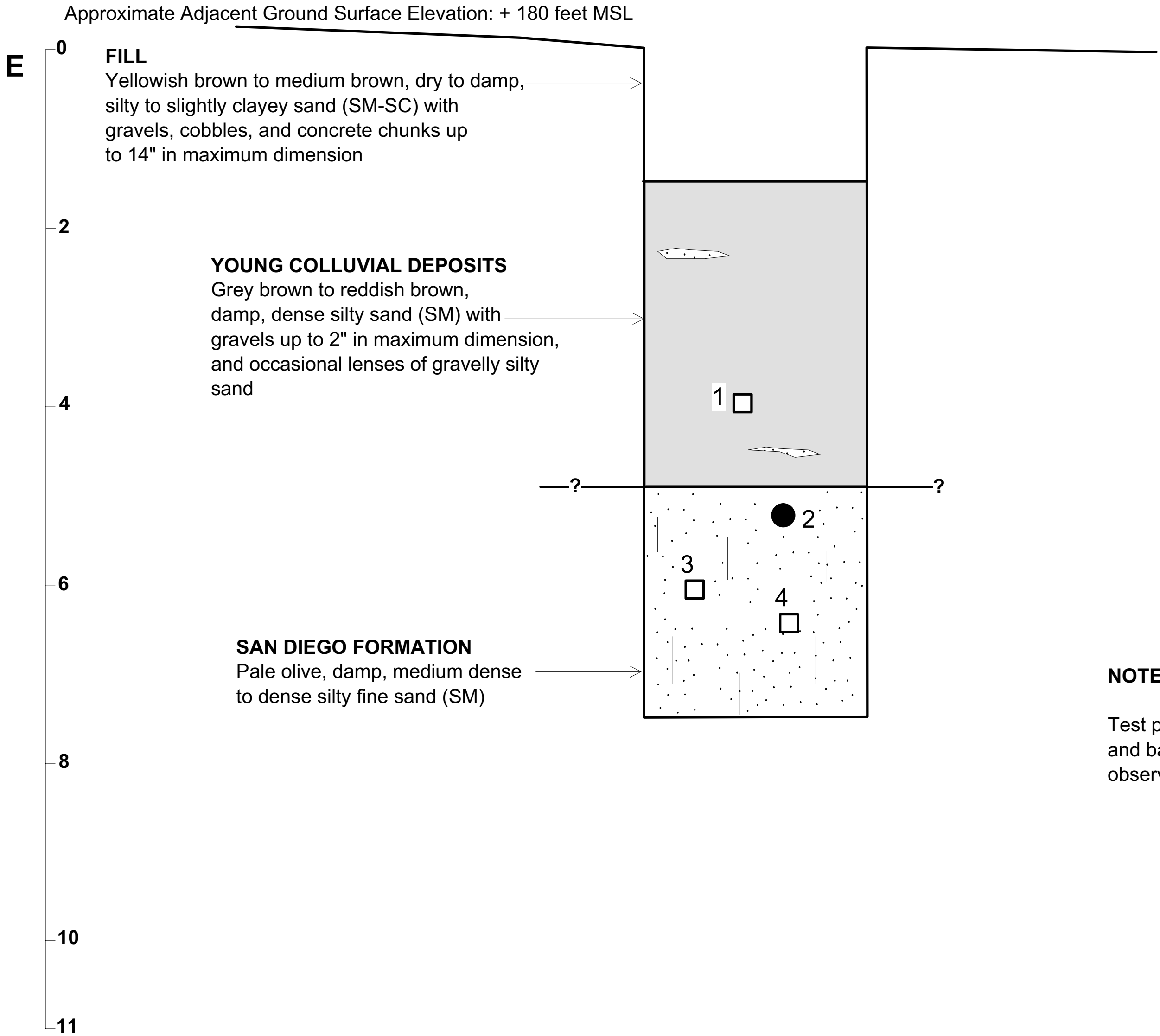
NOTES:

Test pit was excavated to a depth of 6.5 feet on July 10, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-5**

LOG OF TEST PIT TP-6



LEGEND:

☐	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

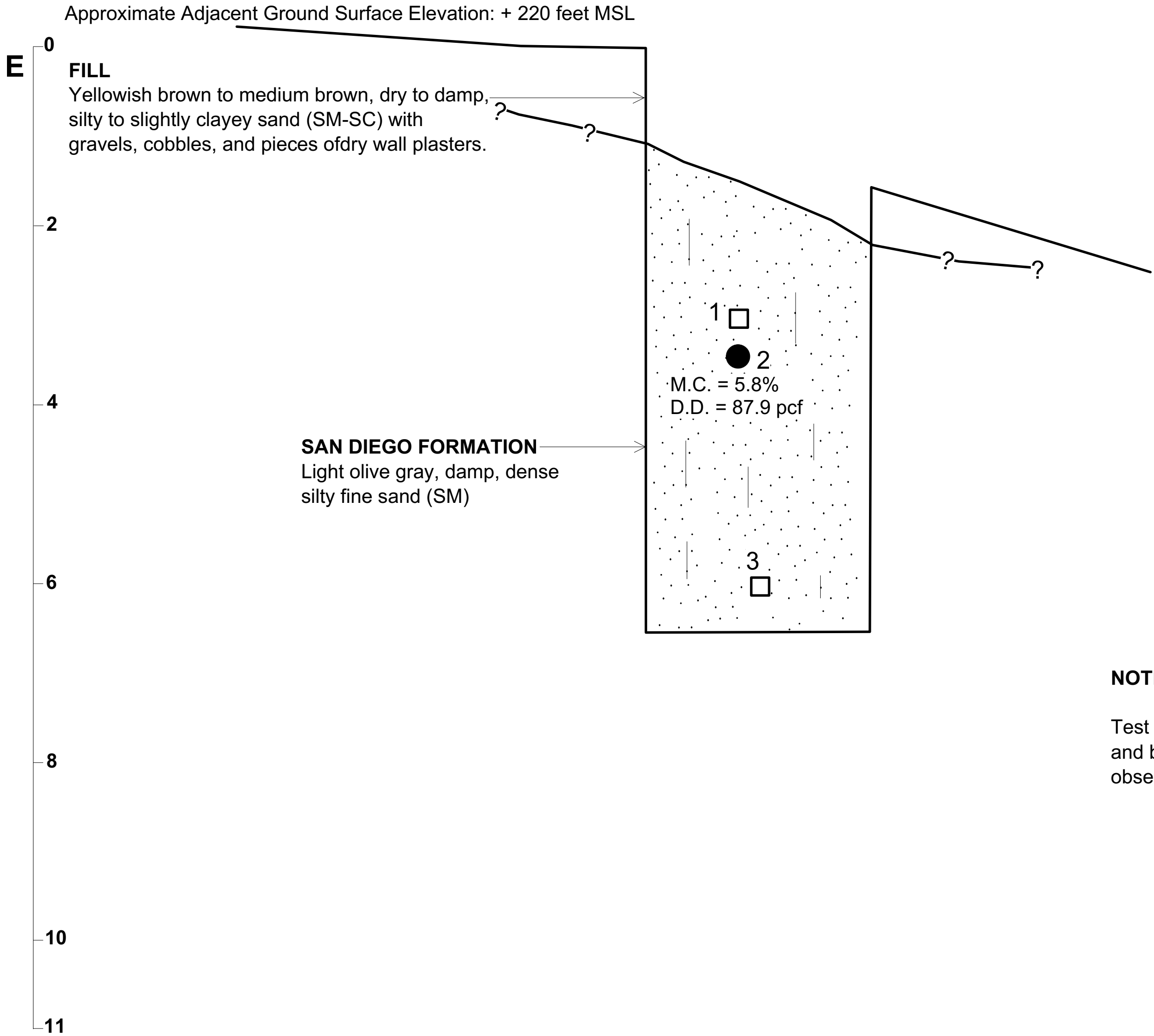
NOTES:

Test pit was excavated to a depth of 7.5 feet on July 10, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-6**

LOG OF TEST PIT TP-7



LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

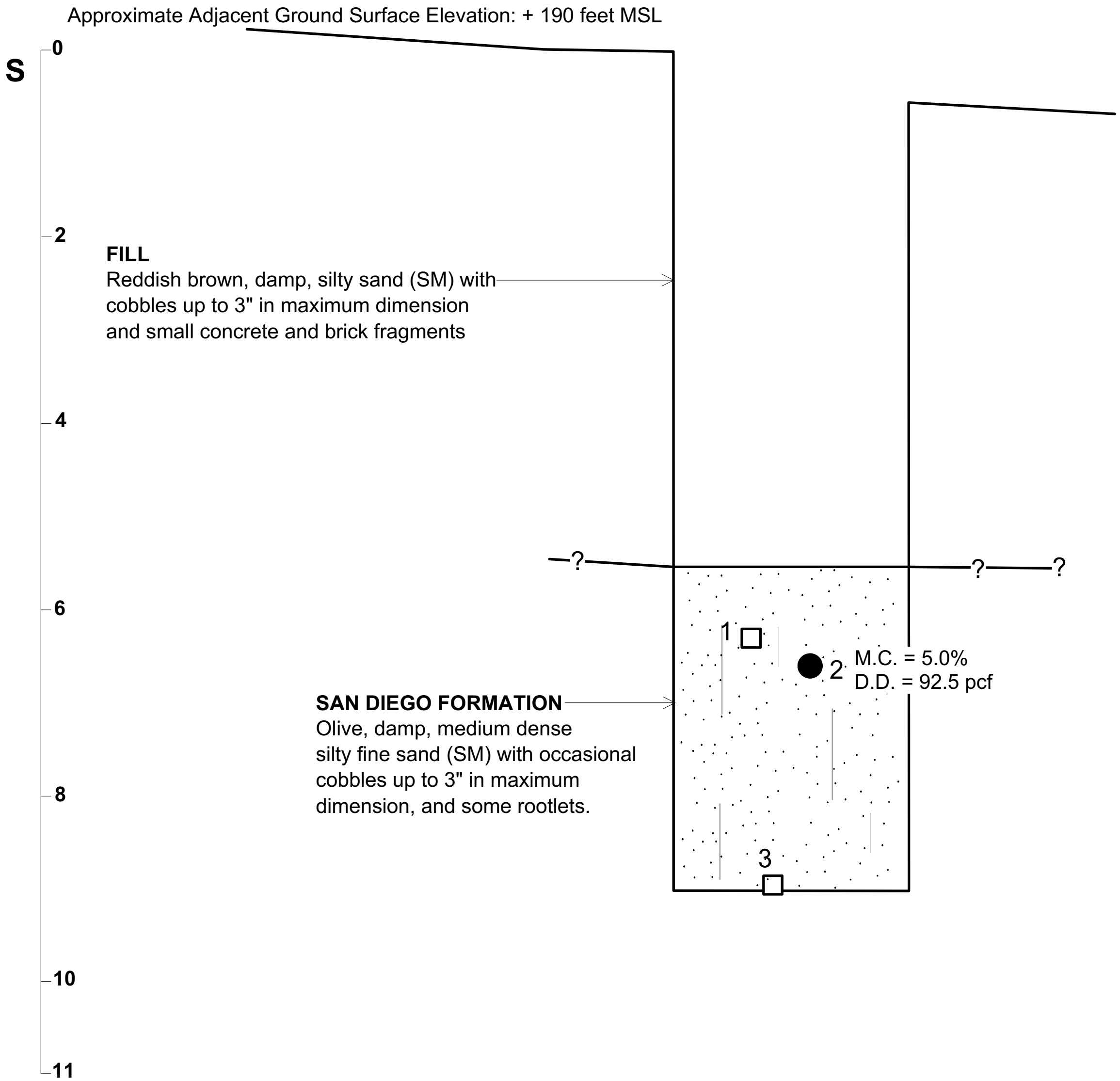
NOTES:

Test pit was excavated to a depth of 6.5 feet on July 10, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-7**

LOG OF TEST PIT TP-8



LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

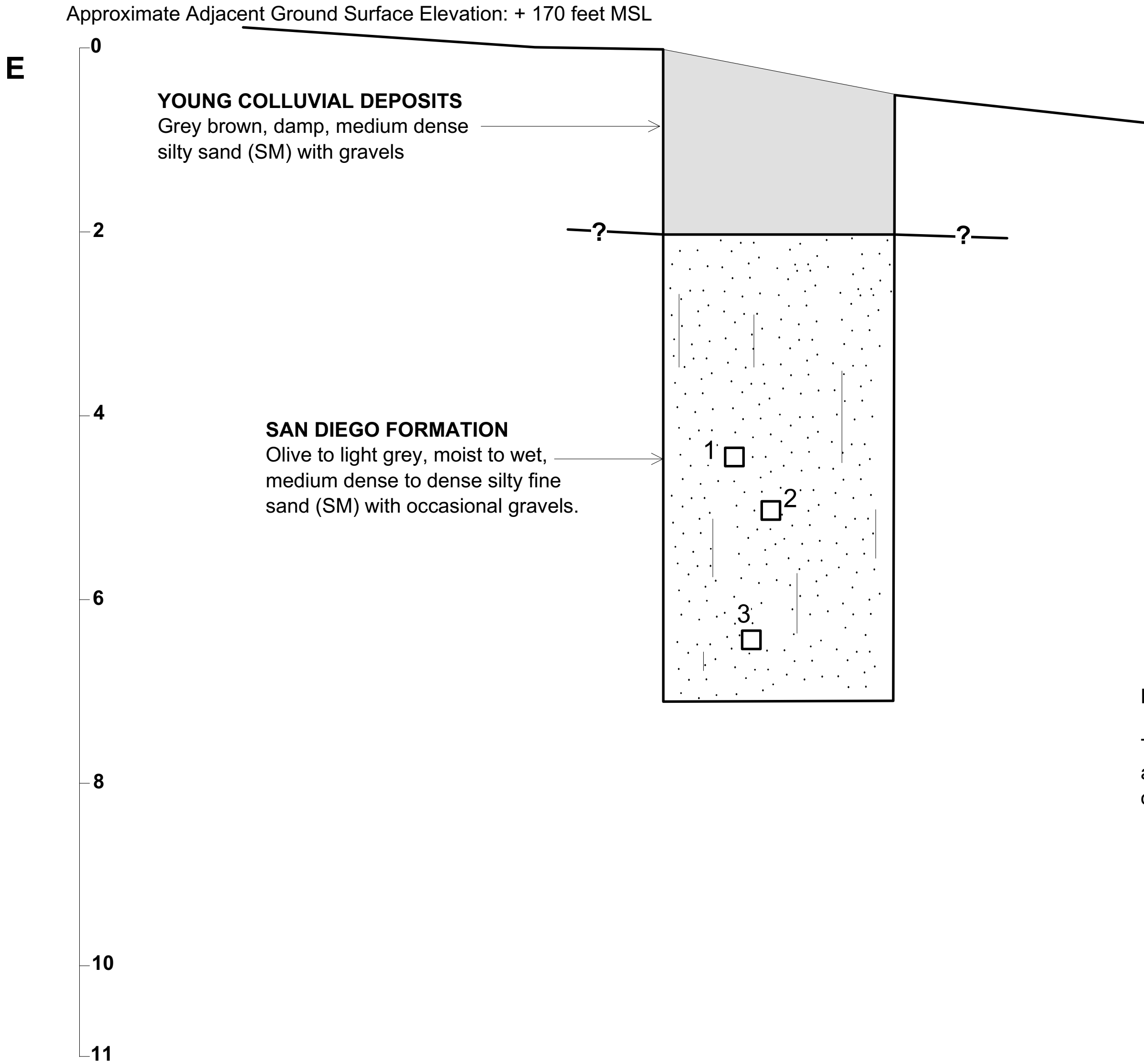
NOTES:

Test pit was excavated to a depth of 9 feet on July 9, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-8**

LOG OF TEST PIT TP-9



LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

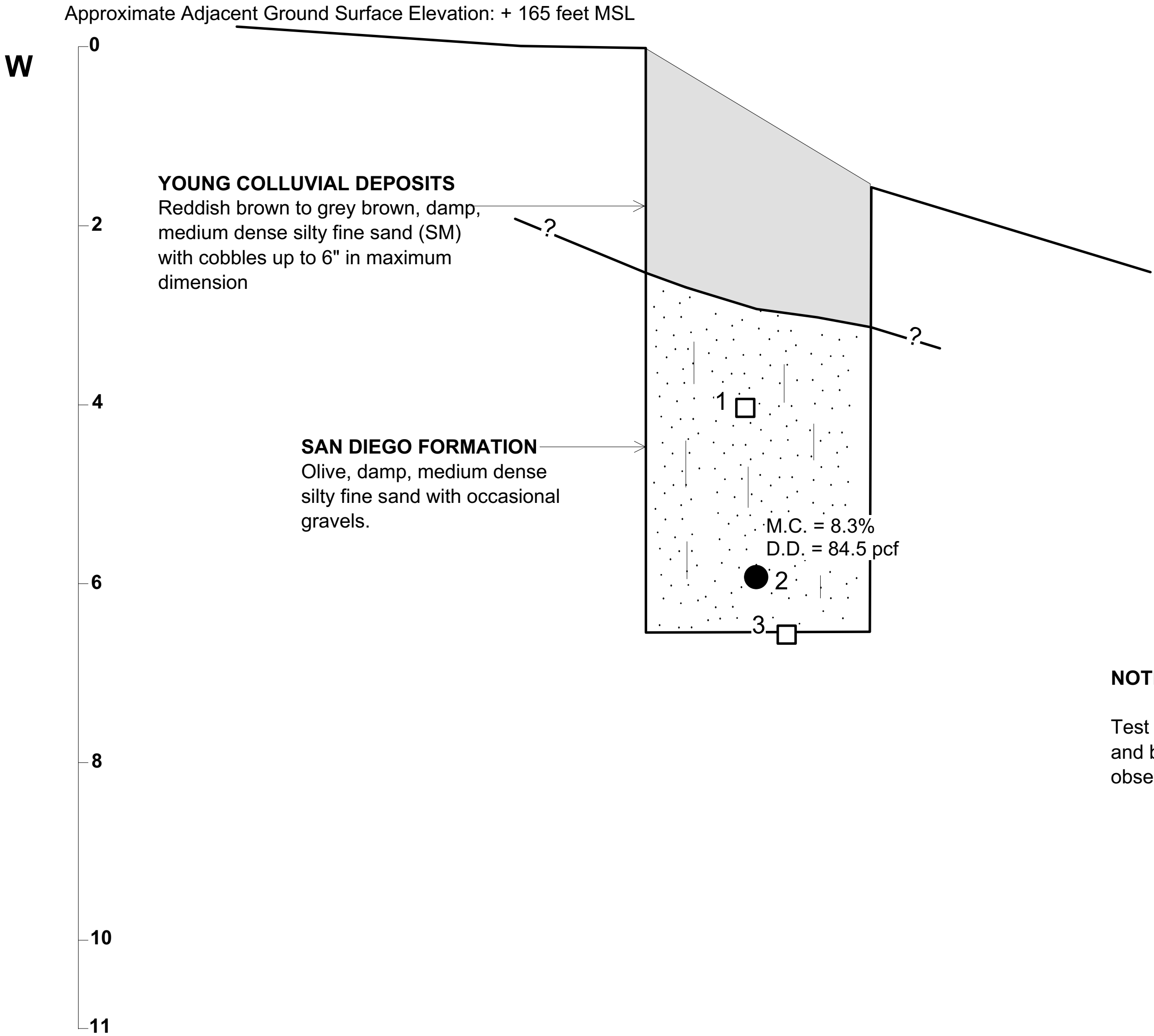
NOTES:

Test pit was excavated to a depth of 7 feet on July 8, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-9**

LOG OF TEST PIT TP-10



LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

NOTES:

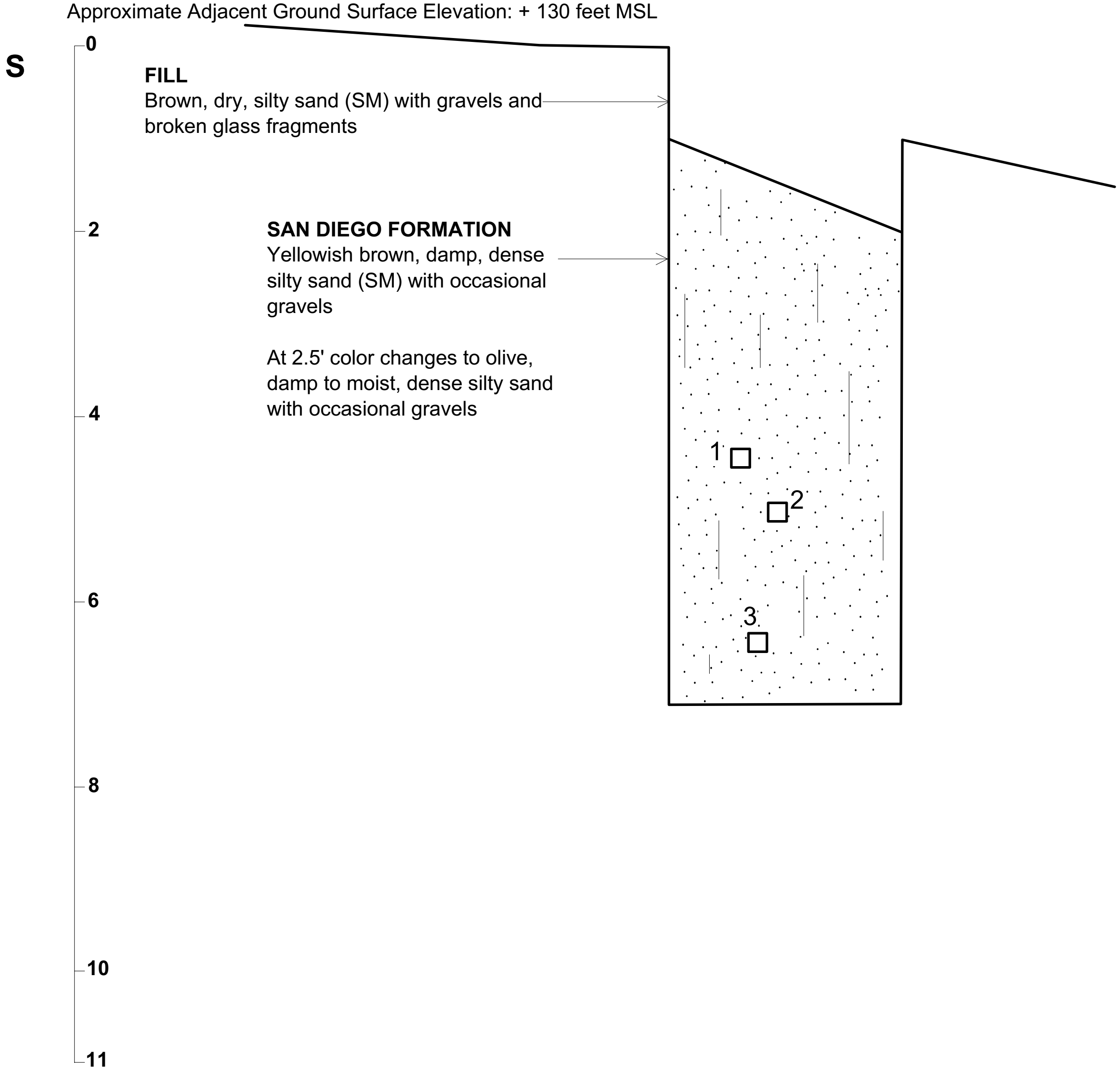
Test pit was excavated to a depth of 6.5 feet on July 8, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-10**

LOG OF TEST PIT TP-11

Approximate Adjacent Ground Surface Elevation: + 130 feet MSL



FILL
Brown, dry, silty sand (SM) with gravels and broken glass fragments

SAN DIEGO FORMATION
Yellowish brown, damp, dense silty sand (SM) with occasional gravels

At 2.5' color changes to olive, damp to moist, dense silty sand with occasional gravels

LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

NOTES:

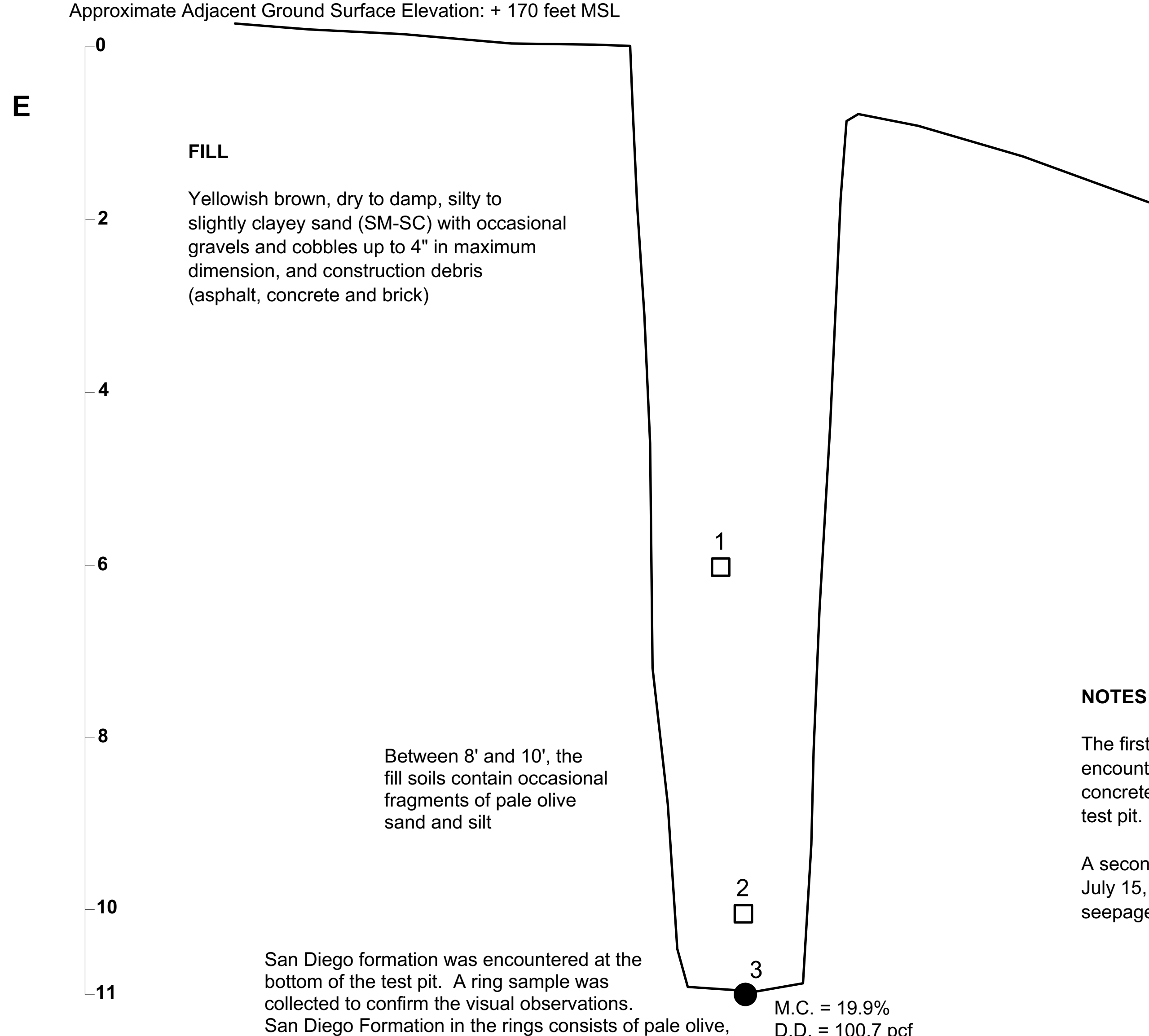
Test pit was excavated to a depth of 7 feet on July 7, 2014 and backfilled on July 8, 2014. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-11**

LOG OF TEST PIT TP-12

Approximate Adjacent Ground Surface Elevation: + 170 feet MSL



FILL

Yellowish brown, dry to damp, silty to slightly clayey sand (SM-SC) with occasional gravels and cobbles up to 4" in maximum dimension, and construction debris (asphalt, concrete and brick)

Between 8' and 10', the fill soils contain occasional fragments of pale olive sand and silt

San Diego formation was encountered at the bottom of the test pit. A ring sample was collected to confirm the visual observations. San Diego Formation in the rings consists of pale olive, wet, stiff sandy silt (ML)

<u>LEGEND</u>:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

NOTES:

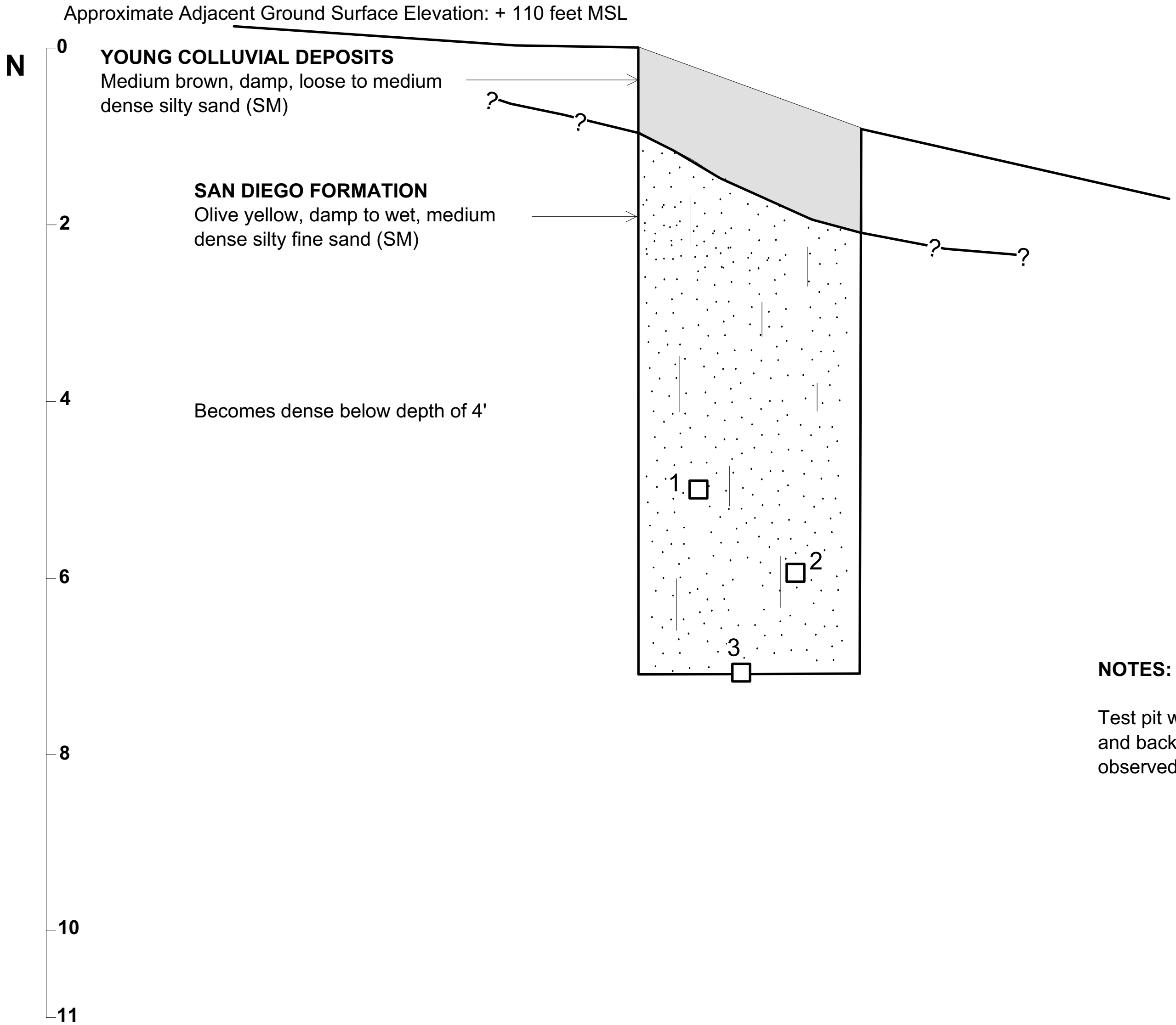
The first test pit was excavated on July 9, 2014 and encountered refusal at depth of 5.5 feet on a large concrete piece which covered the entire bottom of the test pit.

A second test pit was excavated to a depth of 11 feet on July 15, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-12**

LOG OF TEST PIT TP-13



LEGEND:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

NOTES:

Test pit was excavated to a depth of 7 feet on July 7, 2014 and backfilled on the same day. No seepage was observed.

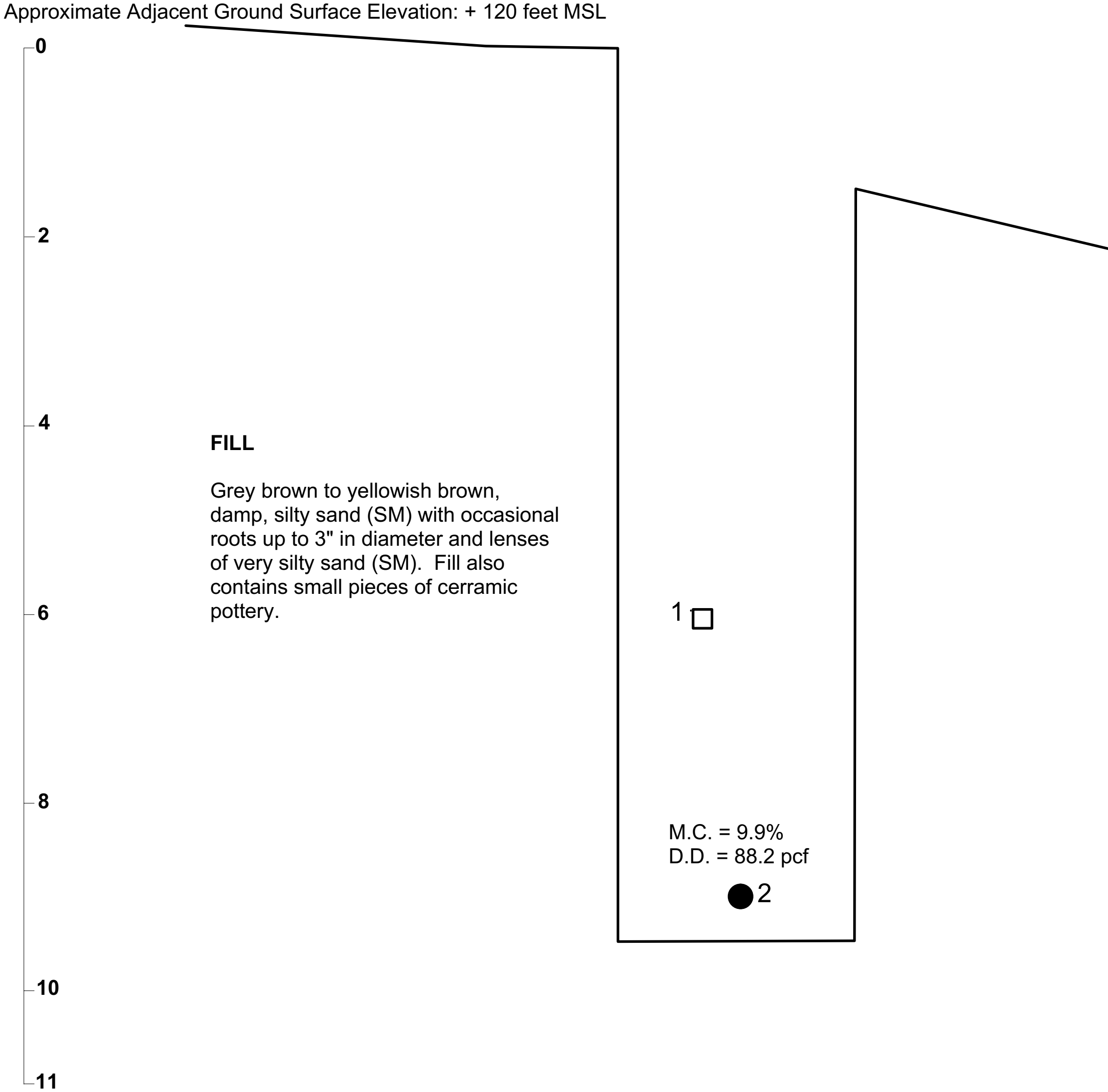
SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-13**

LOG OF TEST PIT TP-14

Approximate Adjacent Ground Surface Elevation: + 120 feet MSL

S N



FILL

Grey brown to yellowish brown, damp, silty sand (SM) with occasional roots up to 3" in diameter and lenses of very silty sand (SM). Fill also contains small pieces of cerramic pottery.

<u>LEGEND</u>:

□	**BULK SAMPLE**
●	**RING SAMPLE**
M.C.	**MOISTURE CONTENT**
D.D.	**DRY DENSITY**

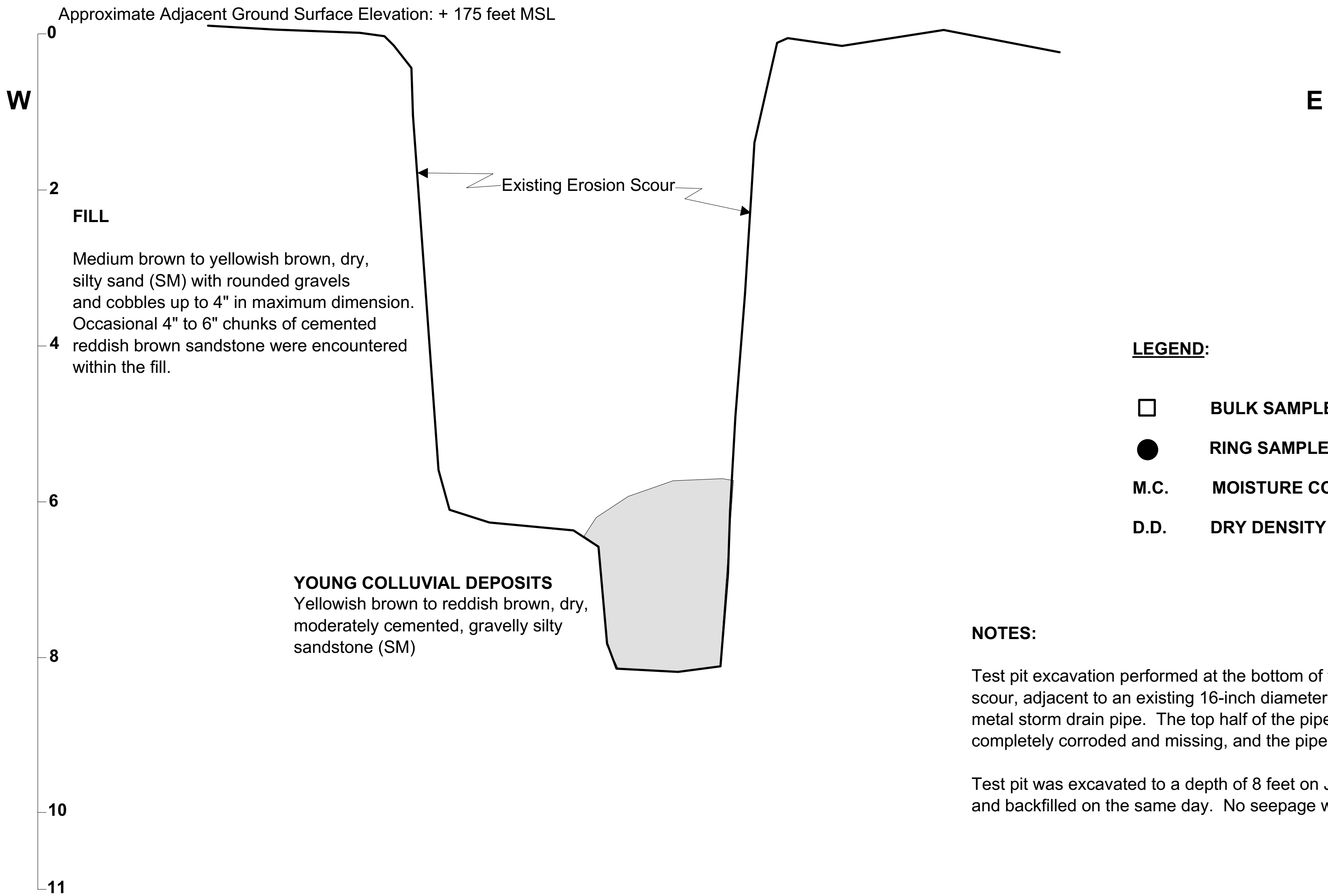
NOTES:

Test pit was excavated to a depth of 9.5 feet on July 7, 2014 and backfilled on July 8, 2014. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-14**

LOG OF TEST PIT TP-15



LEGEND:

☐	BULK SAMPLE
●	RING SAMPLE
M.C.	MOISTURE CONTENT
D.D.	DRY DENSITY

NOTES:

Test pit excavation performed at the bottom of the existing scour, adjacent to an existing 16-inch diameter corrugated metal storm drain pipe. The top half of the pipe was completely corroded and missing, and the pipe was filled with soil.

Test pit was excavated to a depth of 8 feet on July 15, 2014 and backfilled on the same day. No seepage was observed.

SCALE: 1 INCH = 2 FEET (HORIZONTAL AND VERTICAL)

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-15**

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:

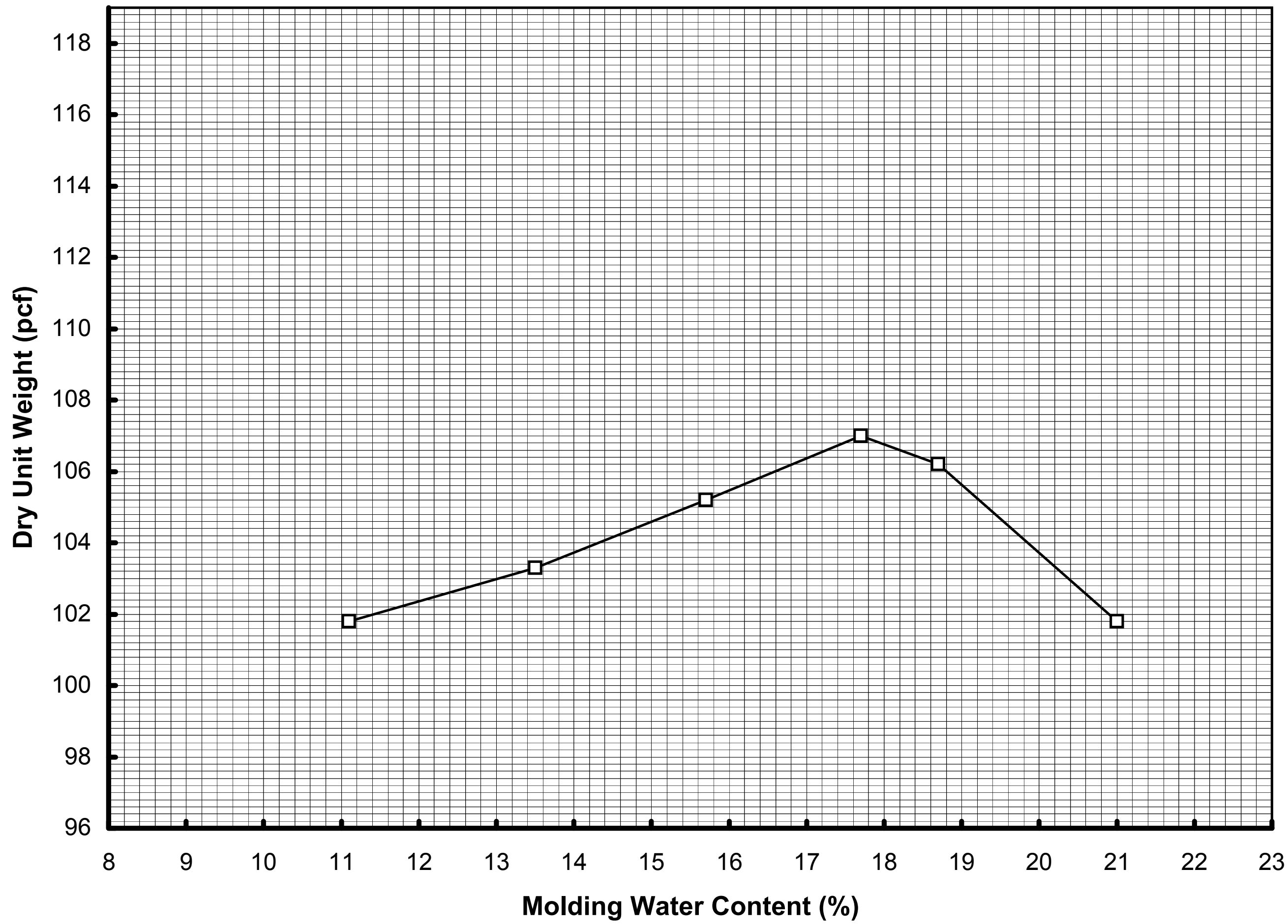
- Determination of in-place moisture content (ASTM D2216). The final test results are presented on the test pit logs;
- Determination of in-place dry density and moisture content (ASTM D2937) based on relatively undisturbed drive samples. The final test results are presented on the test pit logs;
- Maximum density and optimum moisture content (ASTM D1557). The final test results are presented on Figures B-1 through B-10;
- Sieve analyses (ASTM D422), and the final test results are plotted as gradation curves on Figures B-11 and B-12;
- Direct shear test (ASTM D3080). The test results are presented on Figures B-13 through B-20;

In addition, representative samples of the onsite soil materials were delivered to Clarkson Laboratory and Supply, Inc. for analytical (chemical) testing to determine soil pH and resistivity, soluble sulfate and chloride concentrations, and bicarbonate content. Copies of Clarkson's laboratory test data reports are included herein.

COMPACTION CURVE

Test Method: ASTM D 1557

Compaction Procedure: A Specimen Preparation Method: X Moist or Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
2	Bulk	2.5-3.5	17.7	107.0			Pale olive, fine to very fine silty sand (SM)

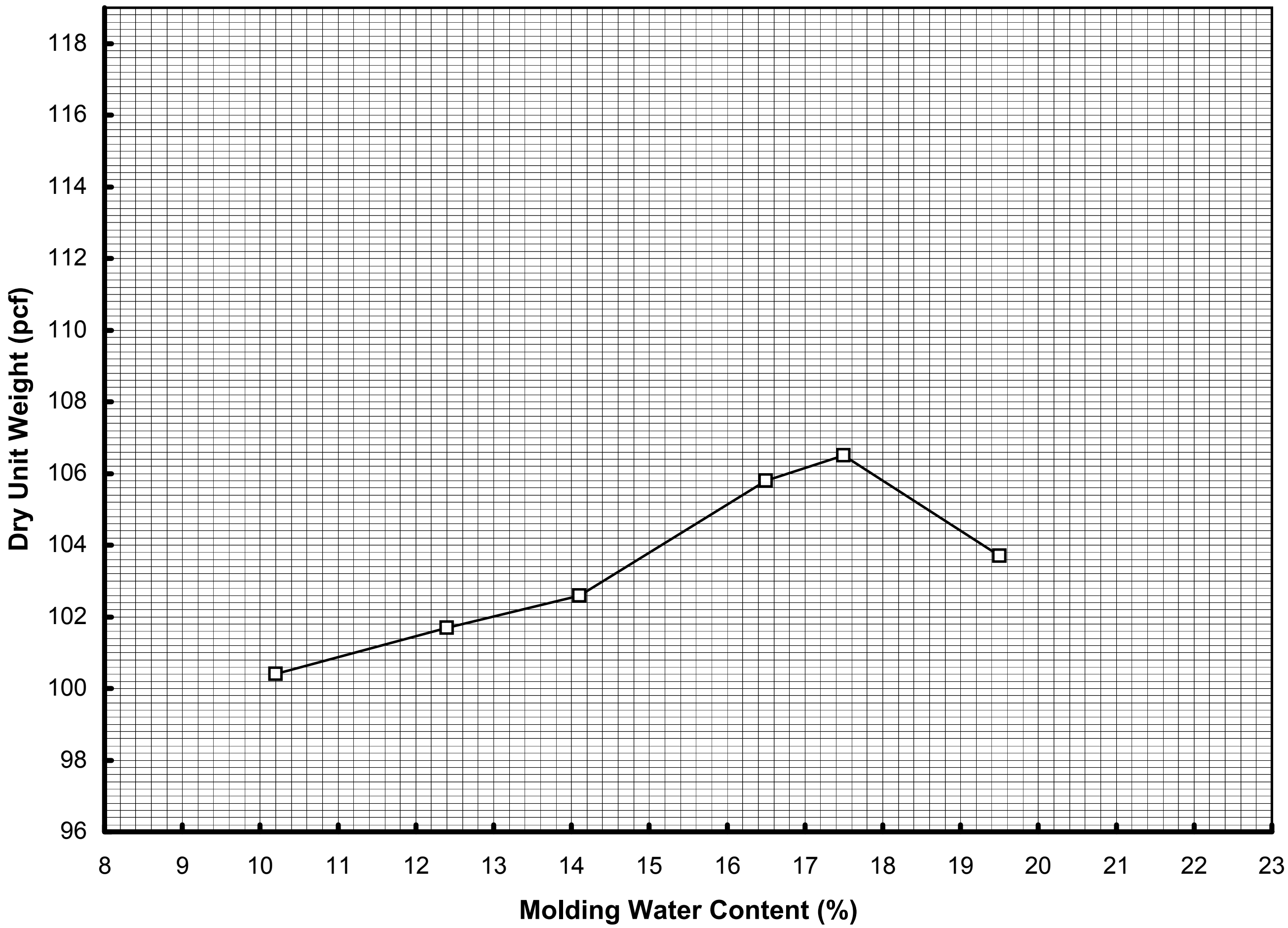
MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 60C508	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE B-1

COMPACTION CURVE

Test Method: ASTM D 1557

Compaction Procedure: A Specimen Preparation Method: X Moist or Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
4	Bulk	2.5-3.5	17.5	106.5			Pale olive, fine to very fine silty sand (SM)

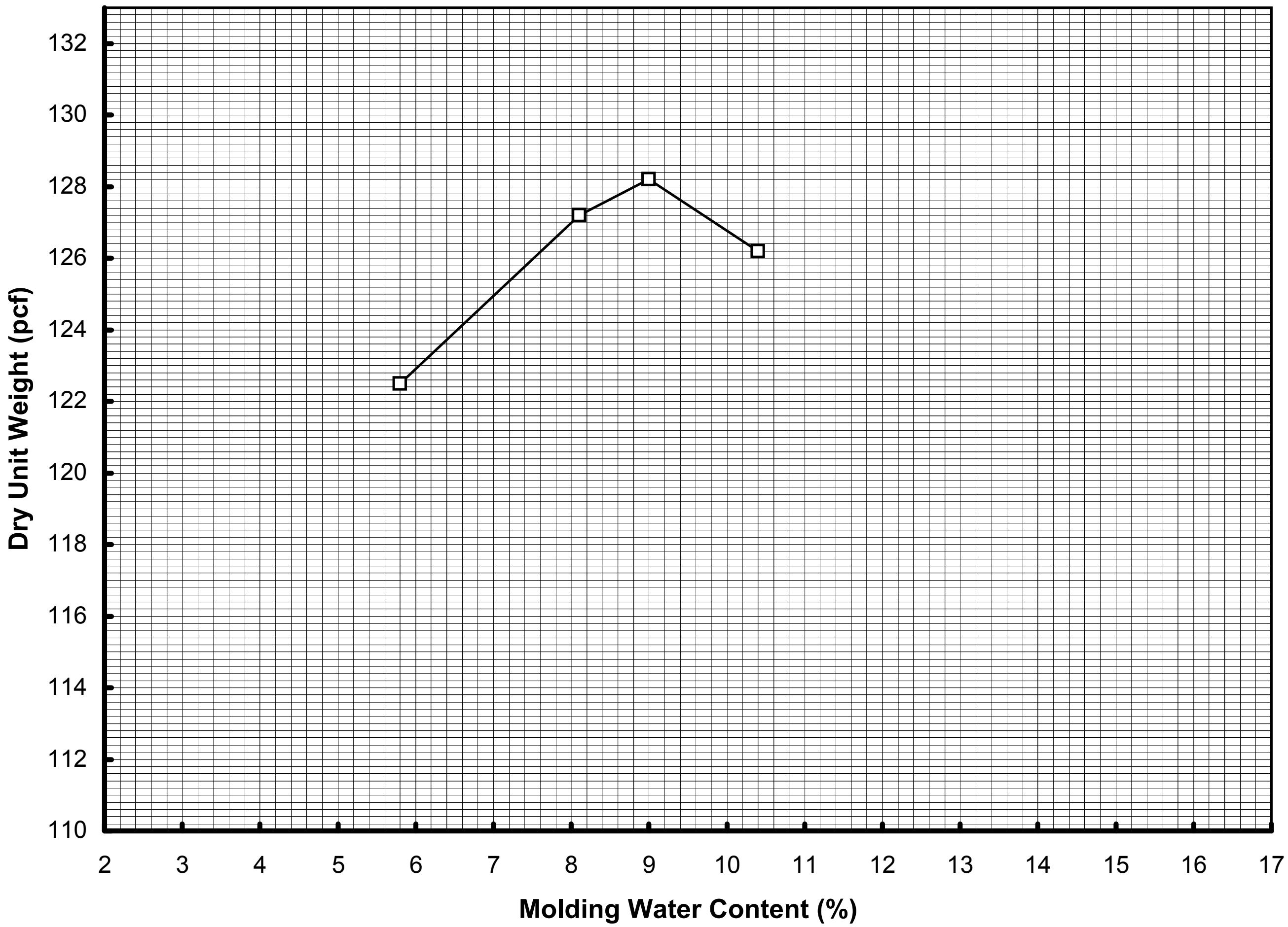
MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 60C508	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE B-2

COMPACTION CURVE

Test Method: ASTM D 1557

Compaction Procedure: C Specimen Preparation Method: X Moist or Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
5	Bulk	3-4	9.0	128.2			Yellowish brown, silty sand (SM) with gravels and cobbles

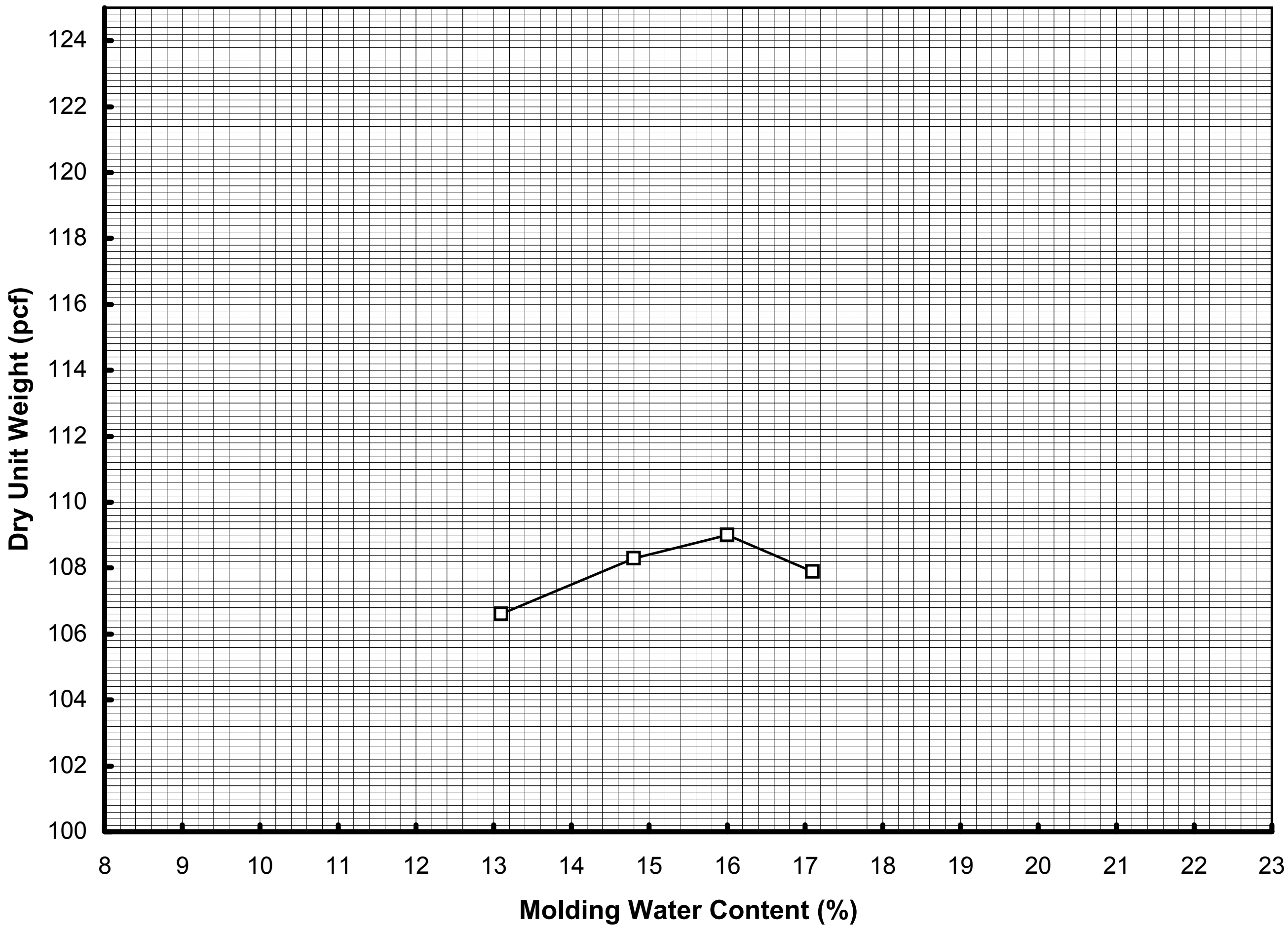
MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 60C508	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE B-3

COMPACTION CURVE

Test Method: ASTM D 1557

Compaction Procedure: A Specimen Preparation Method: X Moist or Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
7	Bulk	3-4	16.0	109.0			Olive brown, very fine to fine silty sand (SM)

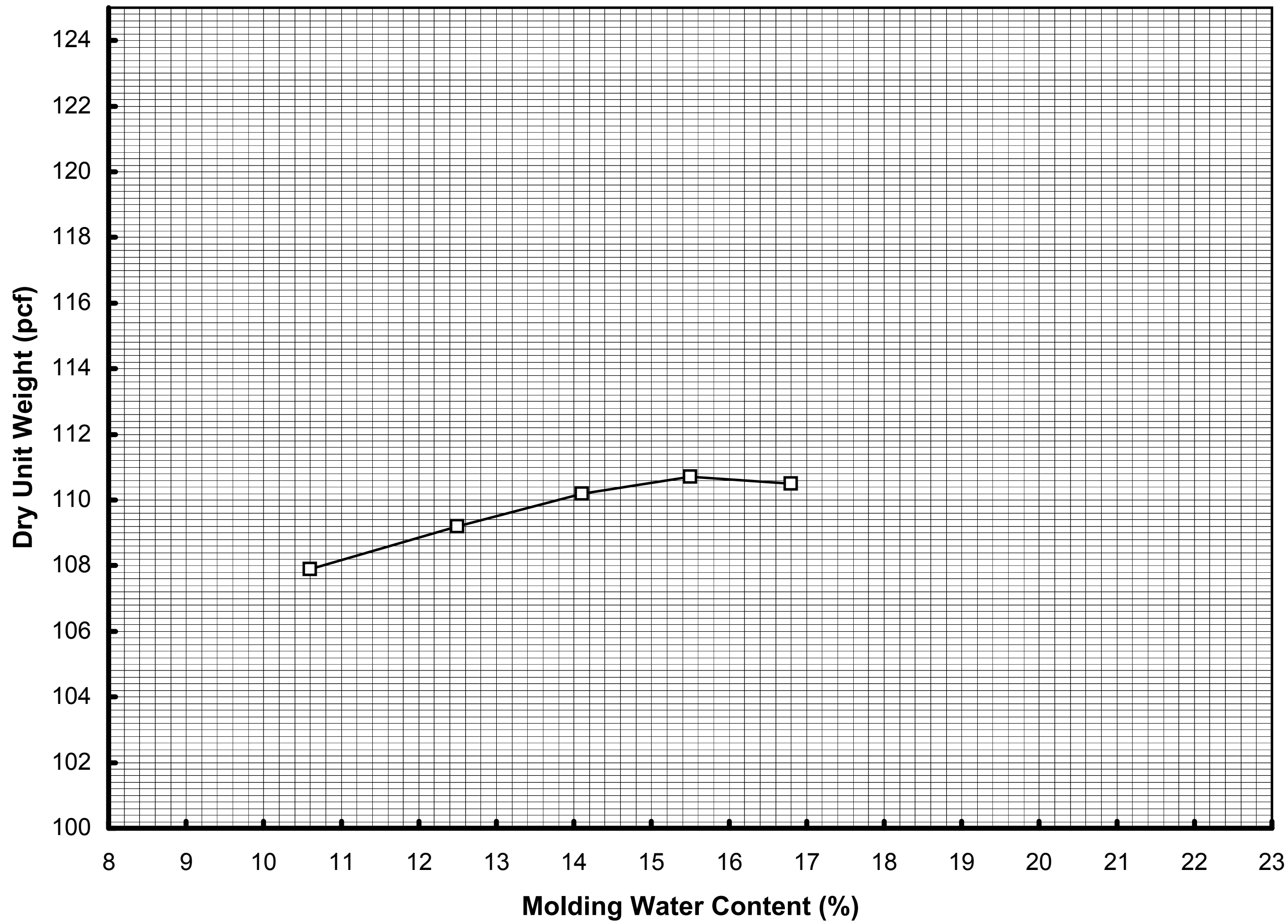
MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 60C508	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE B-4

COMPACTION CURVE

Test Method: ASTM D 1557

Compaction Procedure: A Specimen Preparation Method: X Moist or Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
9	Bulk	5-6	15.5	110.7			Pale olive sandy silt (ML)

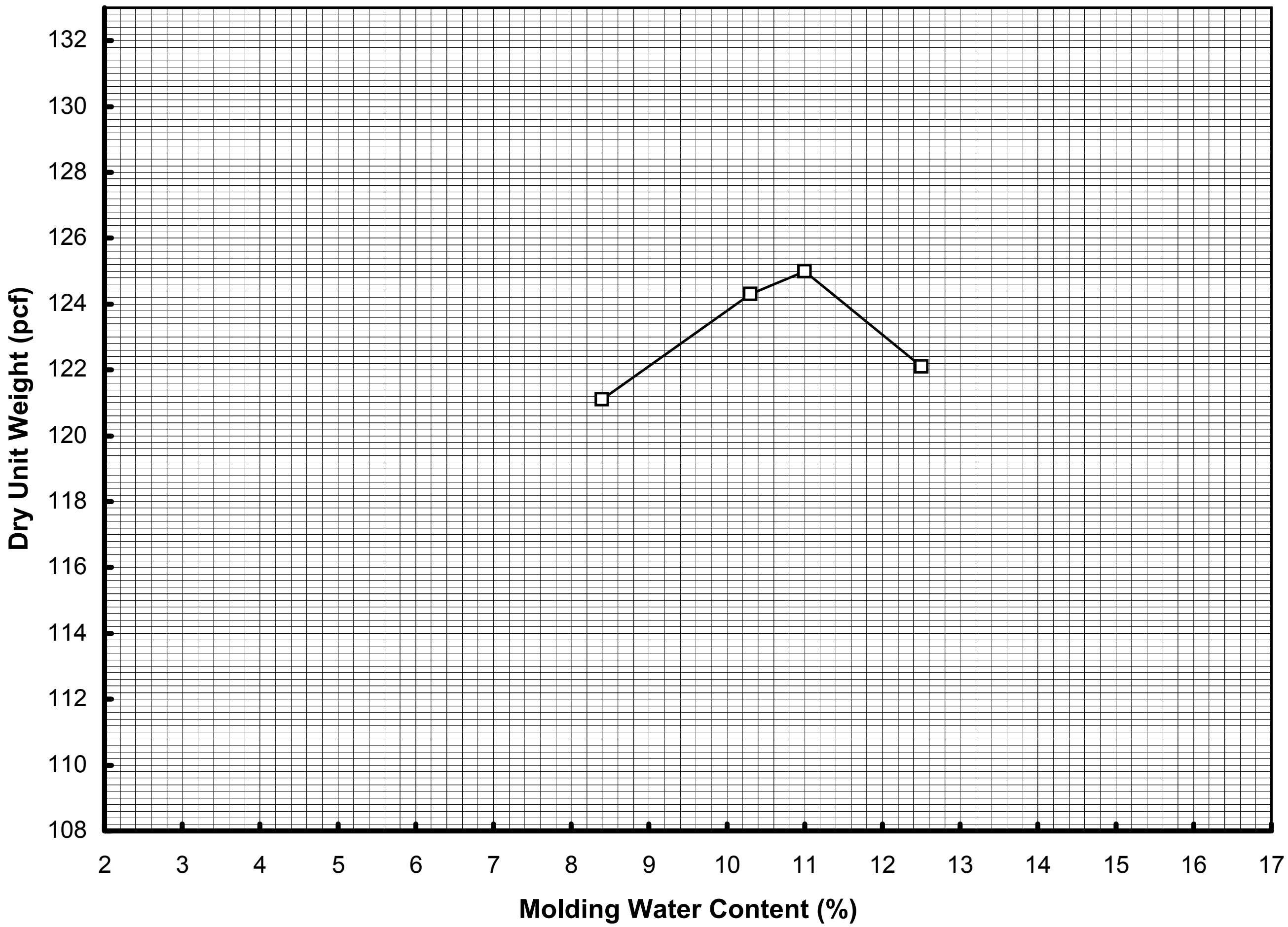
MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE B-5**

COMPACTION CURVE

Test Method: ASTM D 1557

Compaction Procedure: A Specimen Preparation Method: X Moist or ☐ Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
11	Bulk	5-6	11.0	125.0			Olive brown silty to clayey sand (SM-SC)

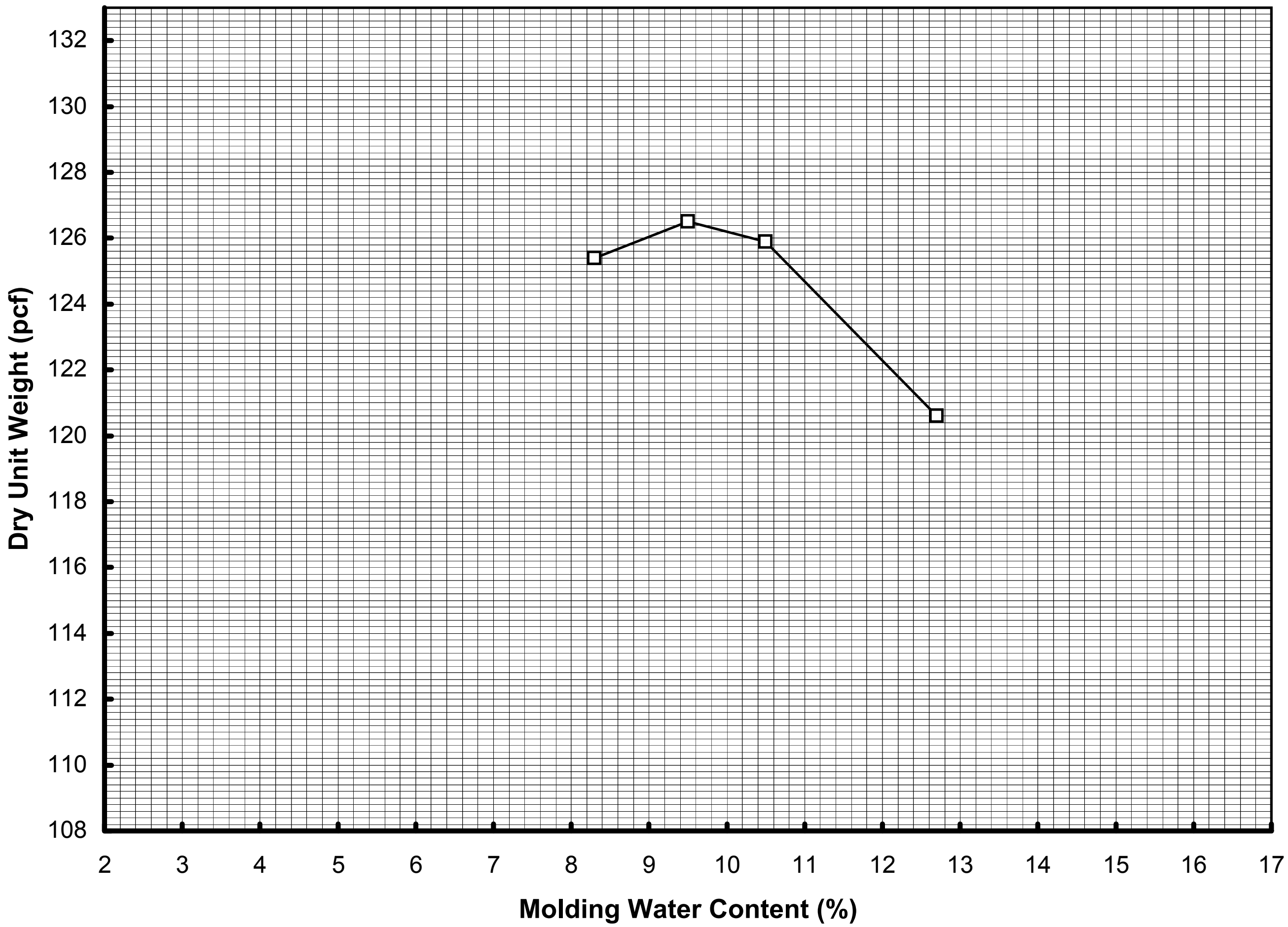
MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 60C508	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE B-6

COMPACTION CURVE

Test Method: ASTM D 1557

Compaction Procedure: A Specimen Preparation Method: X Moist or Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
12	Bulk	9-10	9.5	126.5			Dark yellowish brown, silty sand (SM) with gravel

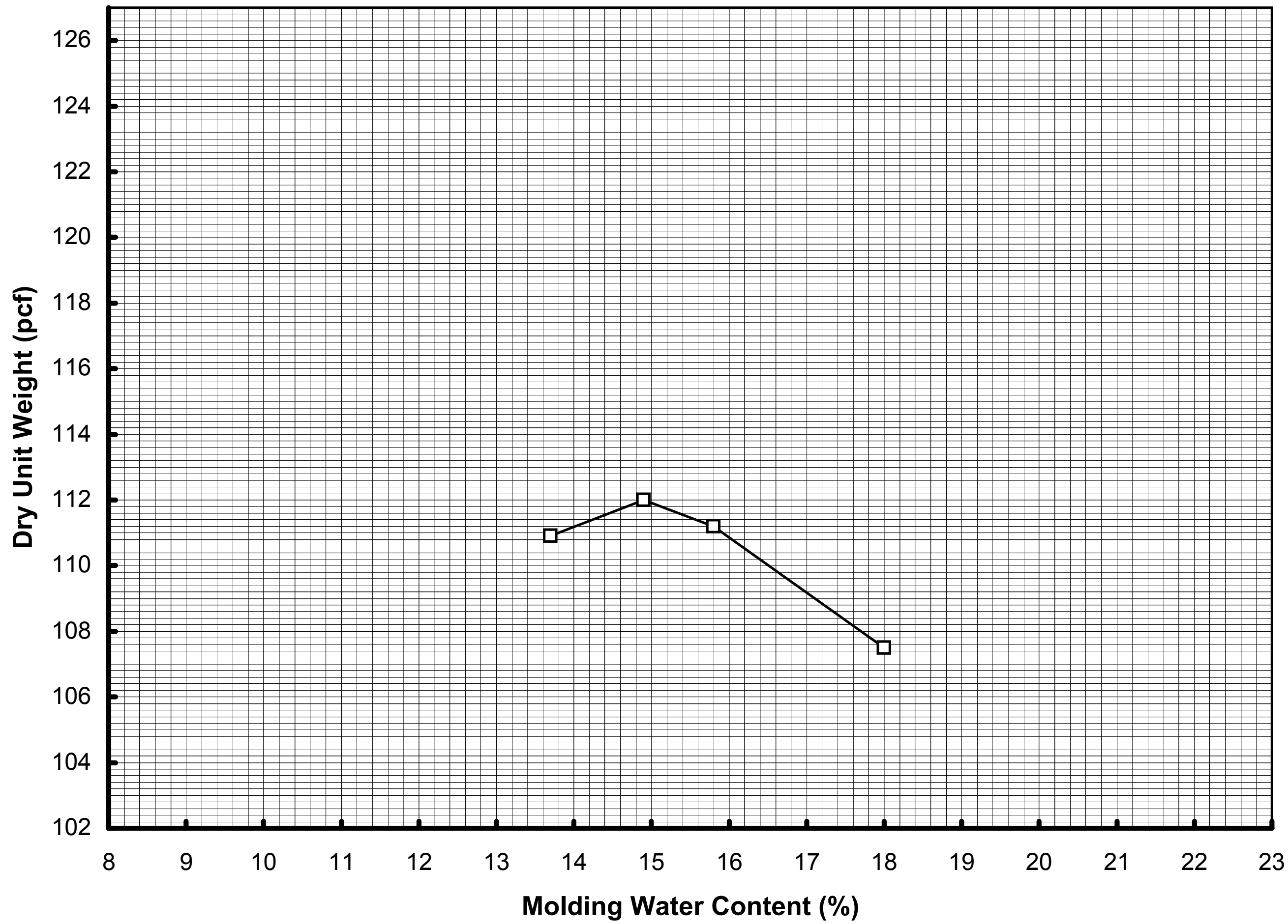
MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 60C508	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE B-7

COMPACTION CURVE

Test Method: ASTM D 1557

Compaction Procedure: A Specimen Preparation Method: X Moist or Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
13	Bulk	5-6	14.9	112.0			Light olive brown, very fine to fine silty sand (SM)

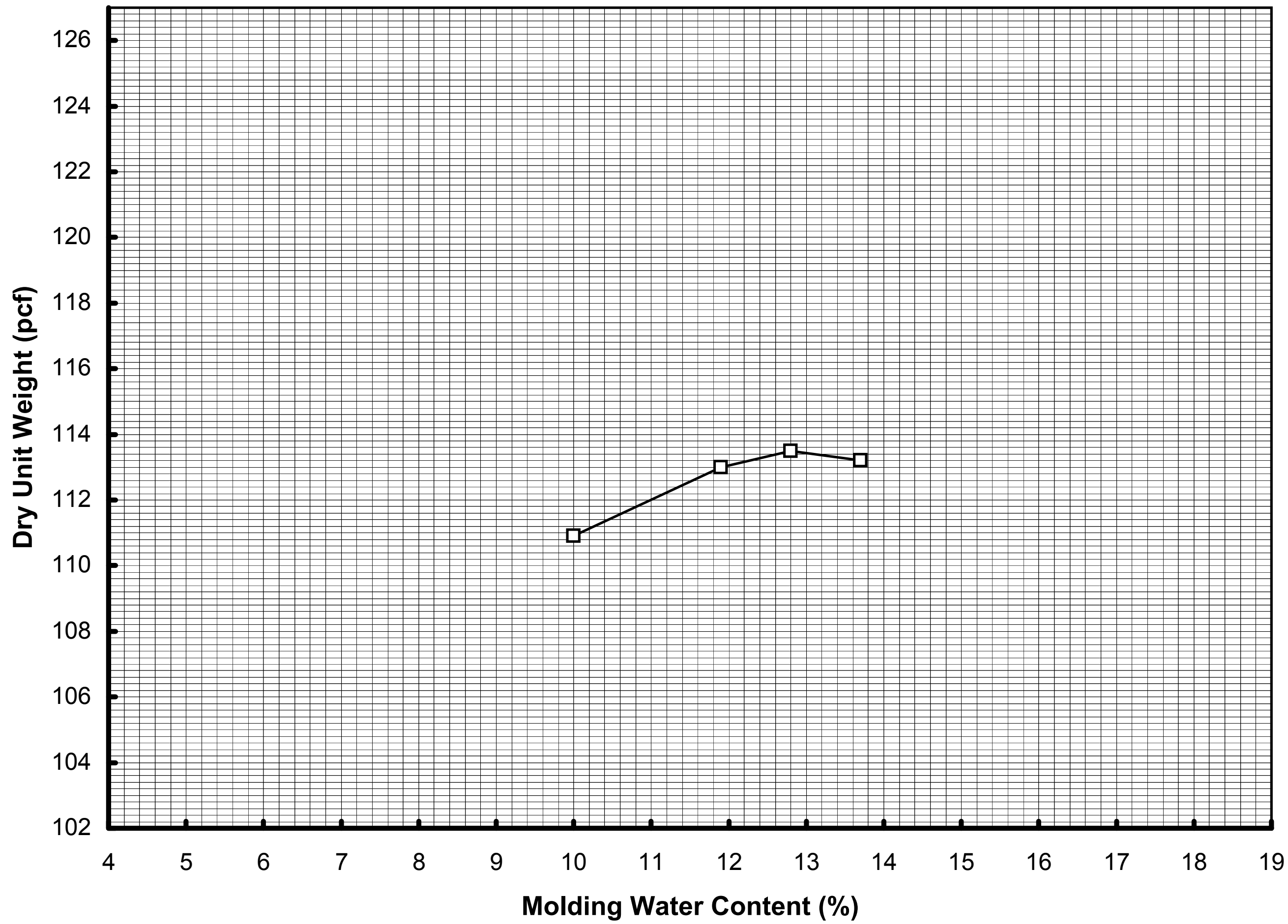
MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 60C508	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE B-8

COMPACTION CURVE

Test Method: ASTM D 1557

Compaction Procedure: A Specimen Preparation Method: X Moist or Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
14	Bulk	6-7	12.8	113.5			Pale olive sandy silt

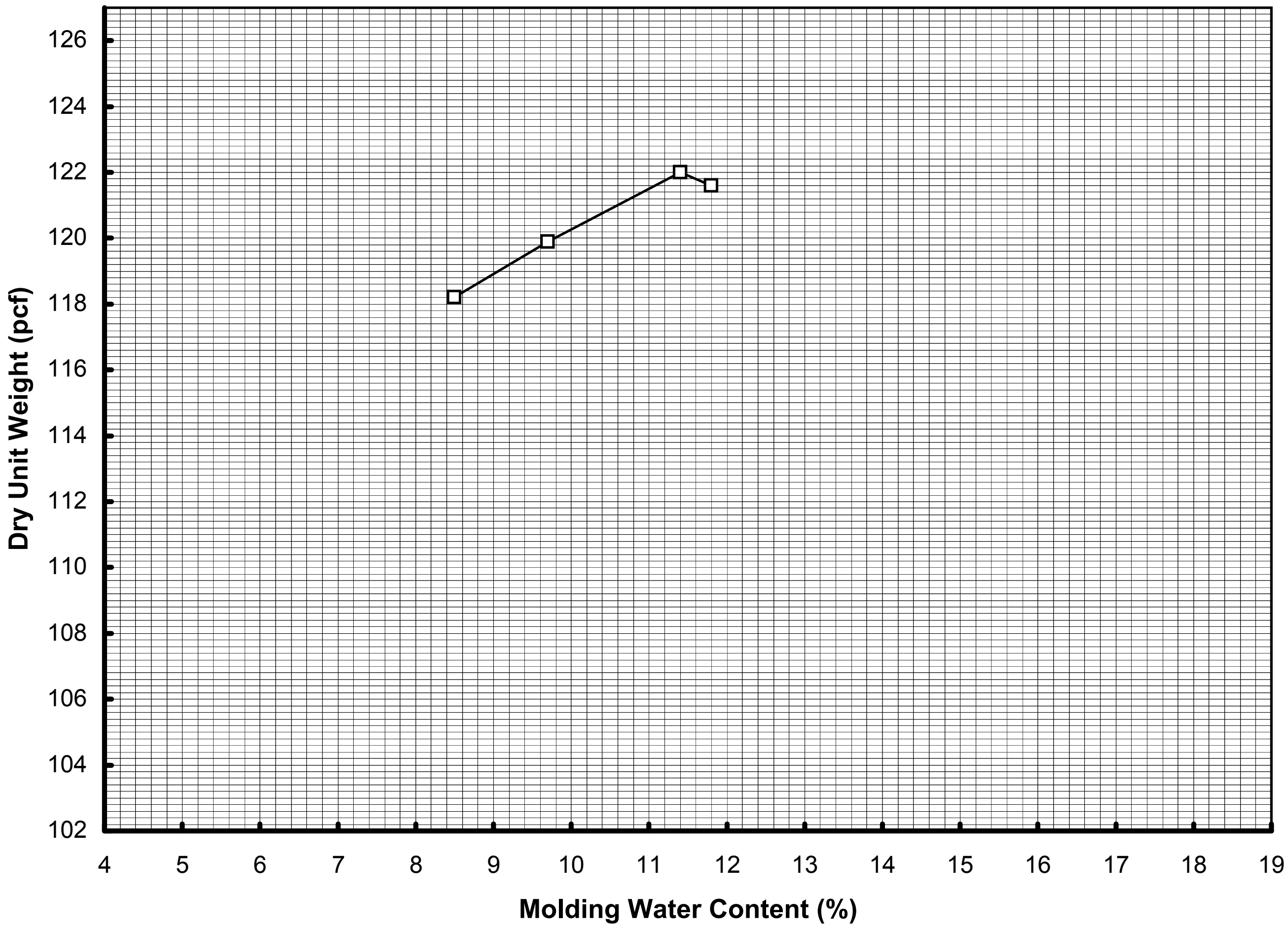
MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 60C508	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE B-9

COMPACTION CURVE

Test Method: ASTM D 1557

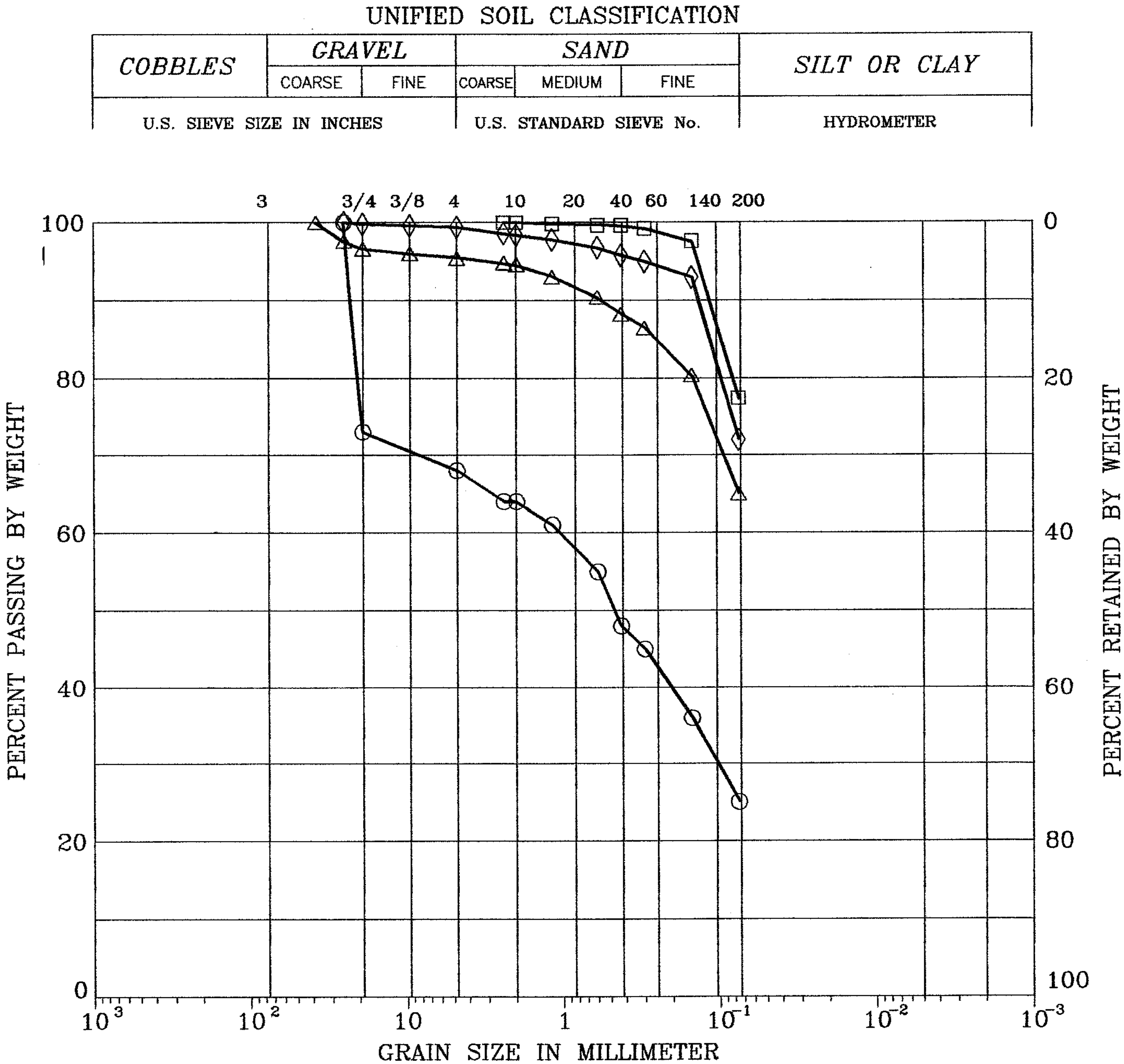
Compaction Procedure: A Specimen Preparation Method: X Moist or Dry



Test Pit No.	Sample Type	Depth (ft)	OPT. WC (%)	MAX. DUW (pcf)	LL	PI	Description and/or Classification
14	Bulk	6-7	12.8	113.5			Pale olive sandy silt

MAPLE CANYON RESTORATION PHASE 1
SAN DIEGO, CALIFORNIA

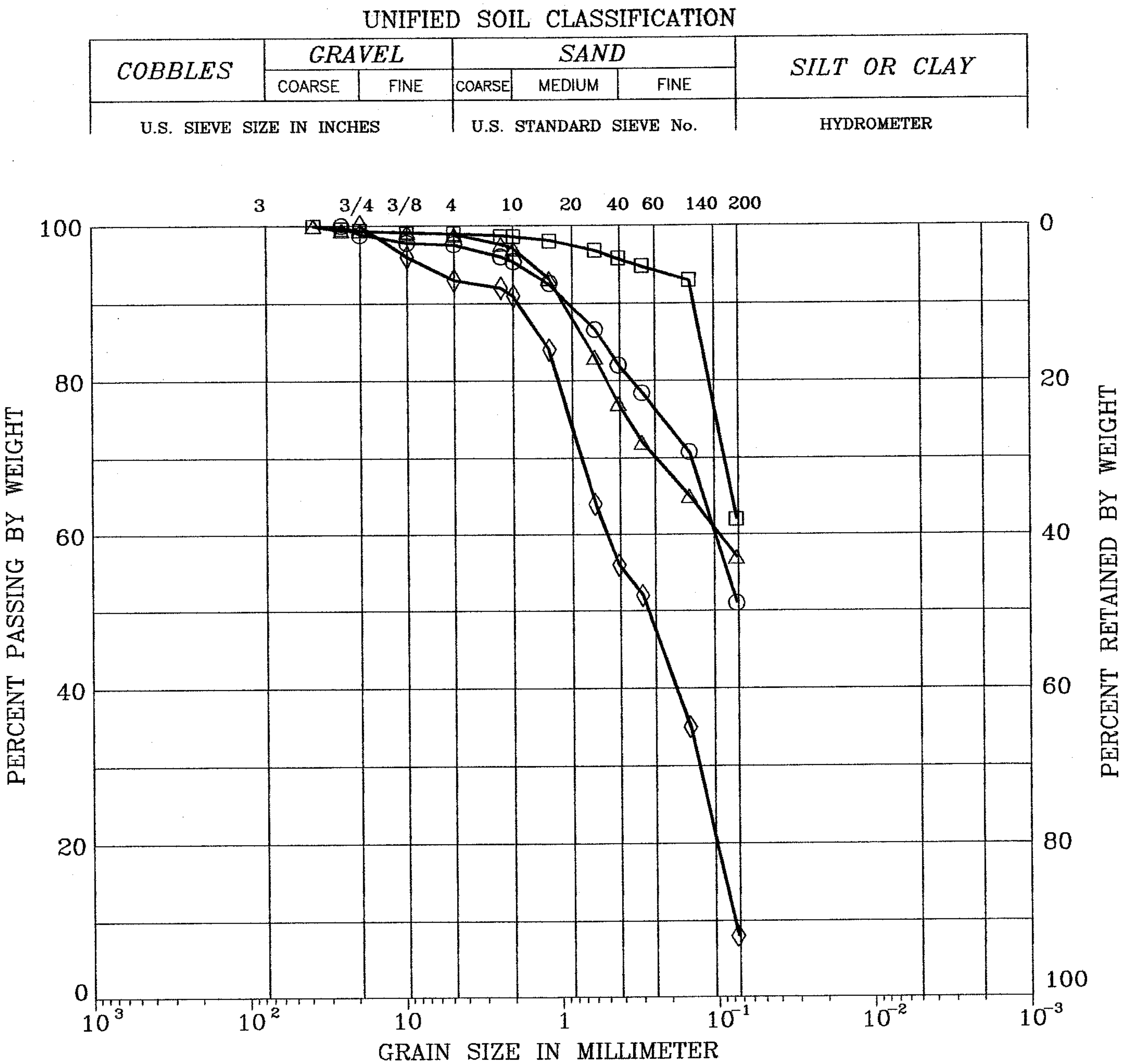
PROJECT NO. 60C508	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE B-10**



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	TP-5	3-4			SILTY SAND (SM)
□	TP-6	7-7.5			SANDY SILT (ML)
△	TP-8	5.5-6.5			SANDY SILT (ML)
◇	TP-9	5-6			SANDY SILT (ML)

Remark :

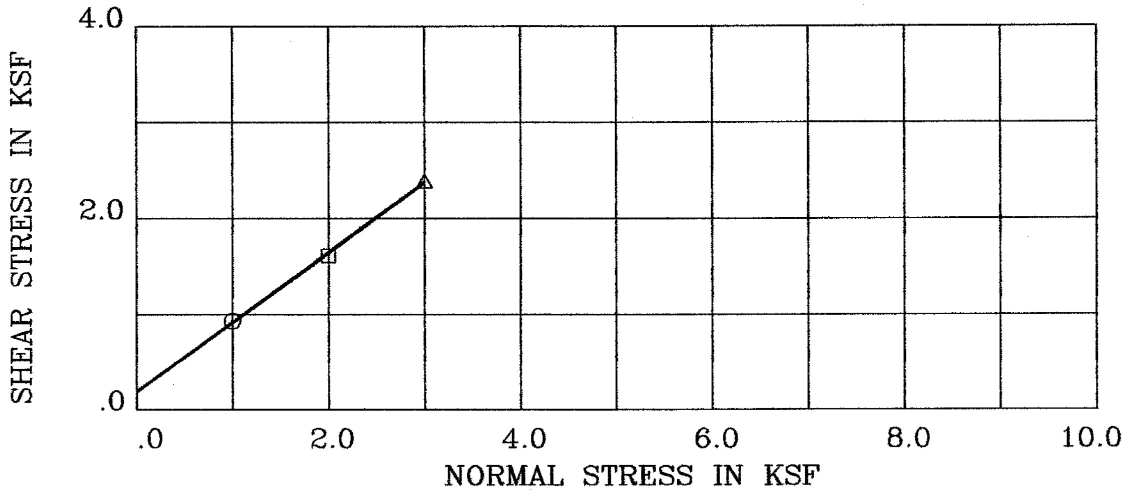
Project 60C508	MAPLE CANYON RESTORATION PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	GRAIN SIZE DISTRIBUTION Figure B-11

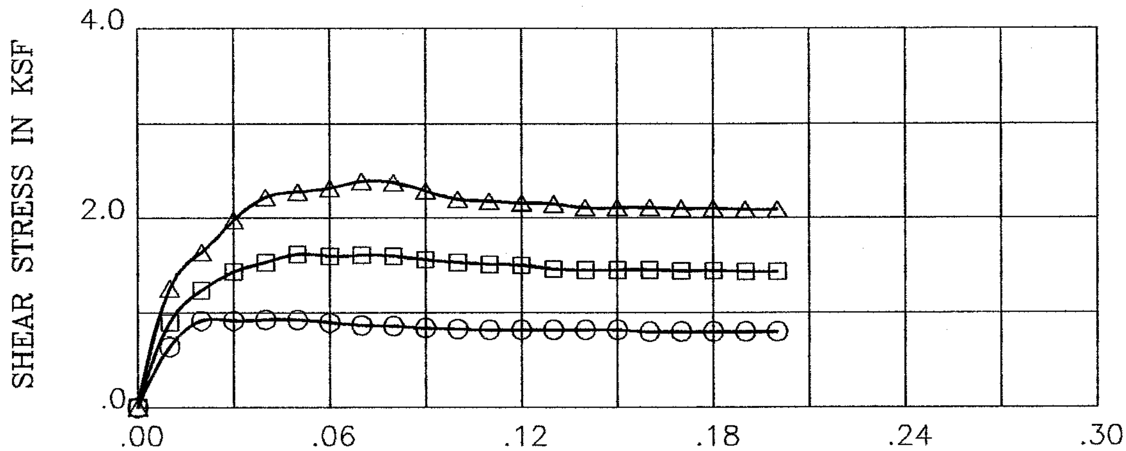


SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	TP-10	4-5			SANDY SILT (ML)
□	TP-13	5-6			SANDY SILT (ML)
△	TP-14	6-7			SANDY SILT (ML)
◇	TP-15	7-8			POORLY GRADED SAND (SP)

Remark :

Project 60C508	MAPLE CANYON RESTORATION PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	GRAIN SIZE DISTRIBUTION Figure B-12





BORING/SAMPLE : TP-2 DEPTH (ft) : 4-4.5

DESCRIPTION :

STRENGTH INTERCEPT (C) : .186 KSF

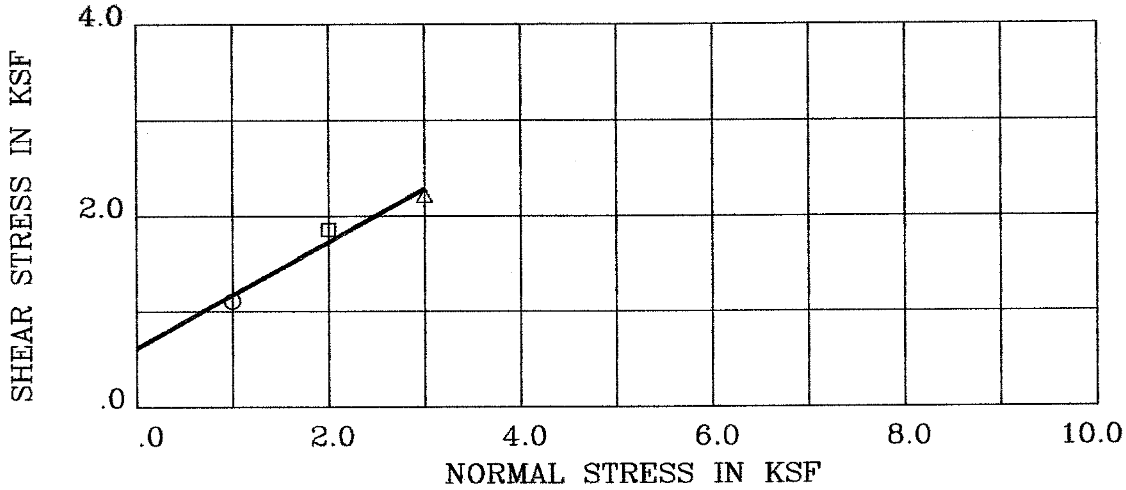
FRICTION ANGLE (PHI) : 36.2 DEG

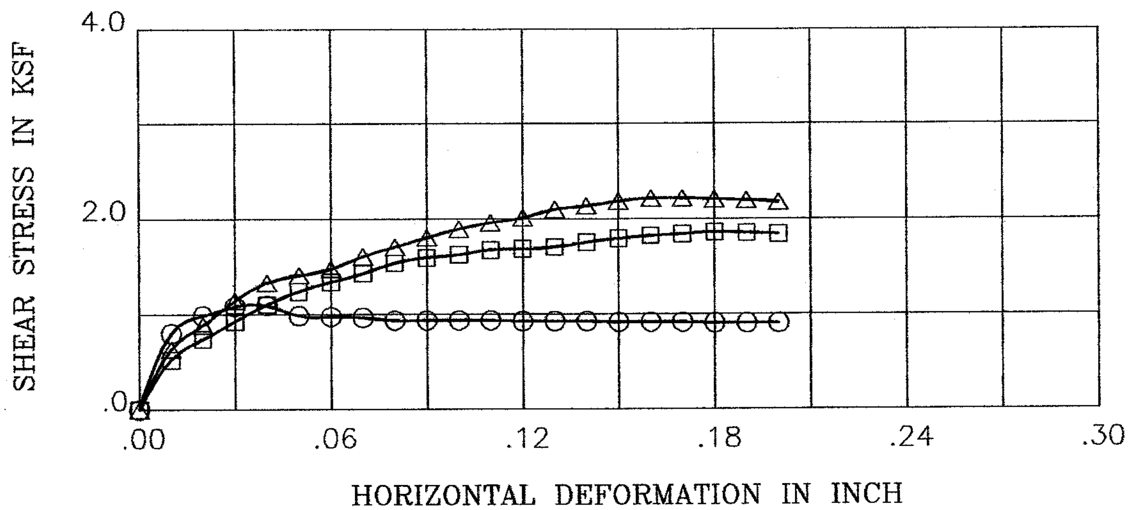
(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	16.7	94.2	.721	1.00	.93	.80
□	16.8	94.9	.709	2.00	1.62	1.43
△	16.6	96.7	.677	3.00	2.39	2.09

Remark :

Project No. 60C508	MAPLE CANYON RESTORATION PROJECT PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	**DIRECT SHEAR TEST** Figure B-13





BORING/SAMPLE : TP-3 DEPTH (ft) : 4.5-5

DESCRIPTION :

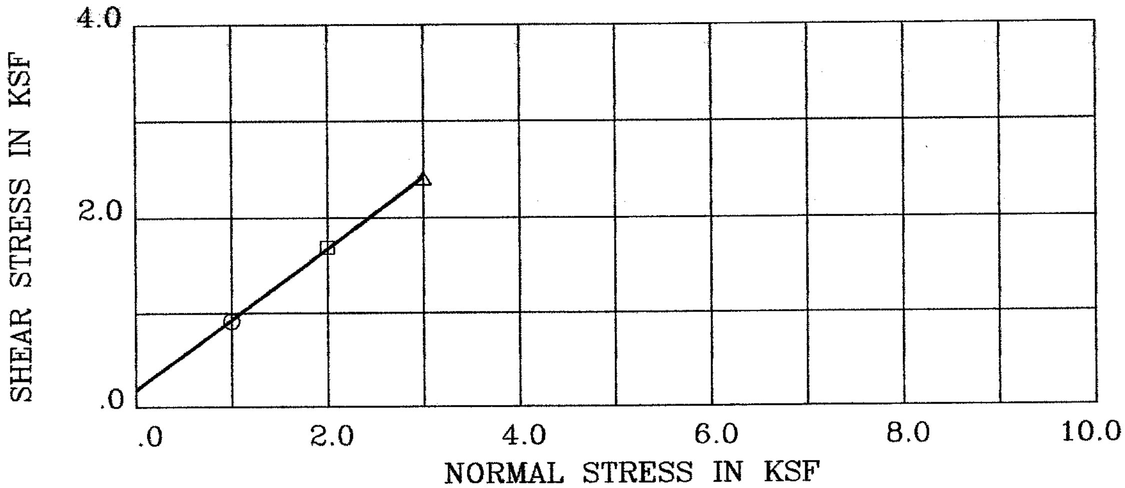
STRENGTH INTERCEPT (C) : .619 KSF (PEAK STRENGTH)

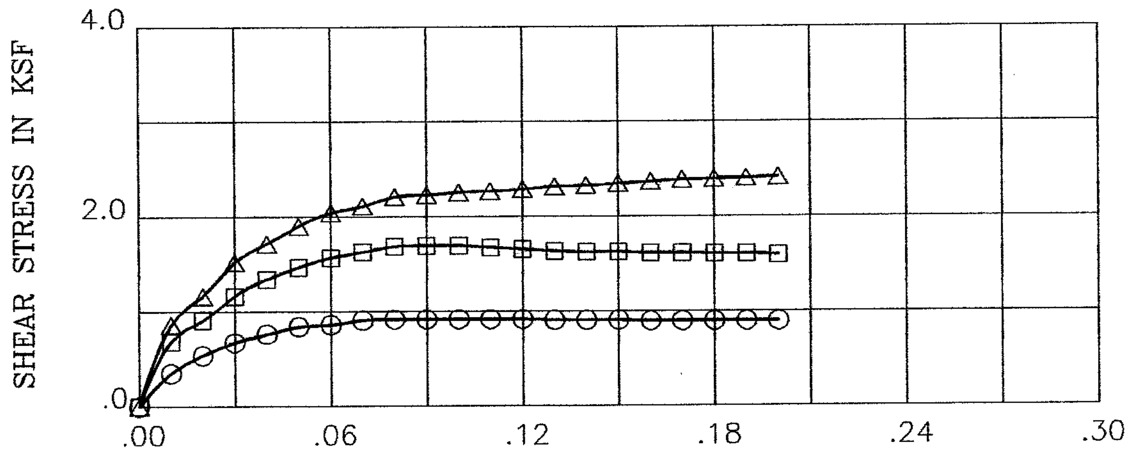
FRICTION ANGLE (PHI) : 29.1 DEG

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	-12.4	116.1	.398	1.00	1.11	.91
□	16.0	76.8	1.113	2.00	1.86	1.84
△	15.4	73.8	1.199	3.00	2.22	2.18

Remark :

Project No. 60C508	MAPLE CANYON RESTORATION PROJECT PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	DIRECT SHEAR TEST Figure B-14





BORING/SAMPLE : TP-4 DEPTH (ft) : 4-4.5

DESCRIPTION :

STRENGTH INTERCEPT (C) : .179 KSF

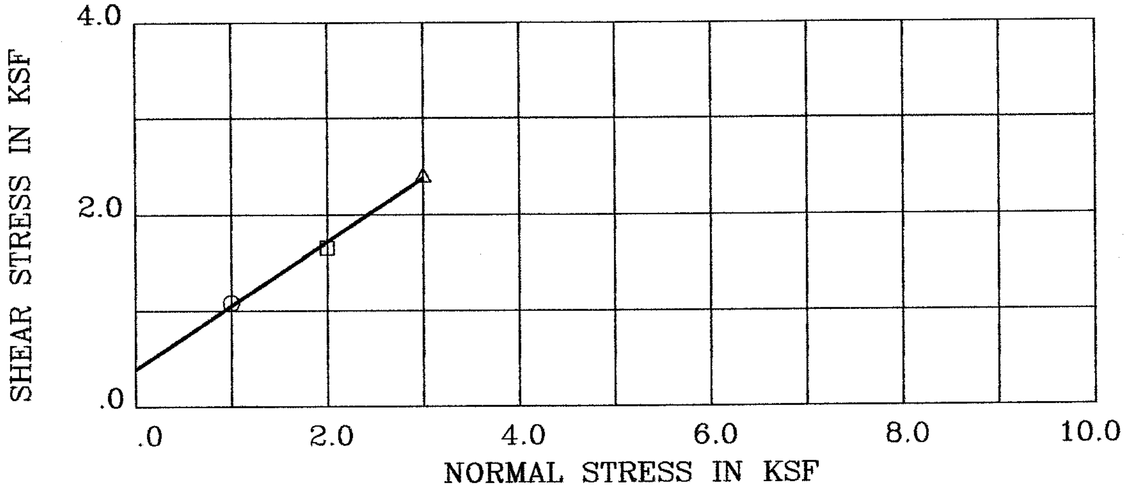
FRICTION ANGLE (PHI) : 36.8 DEG

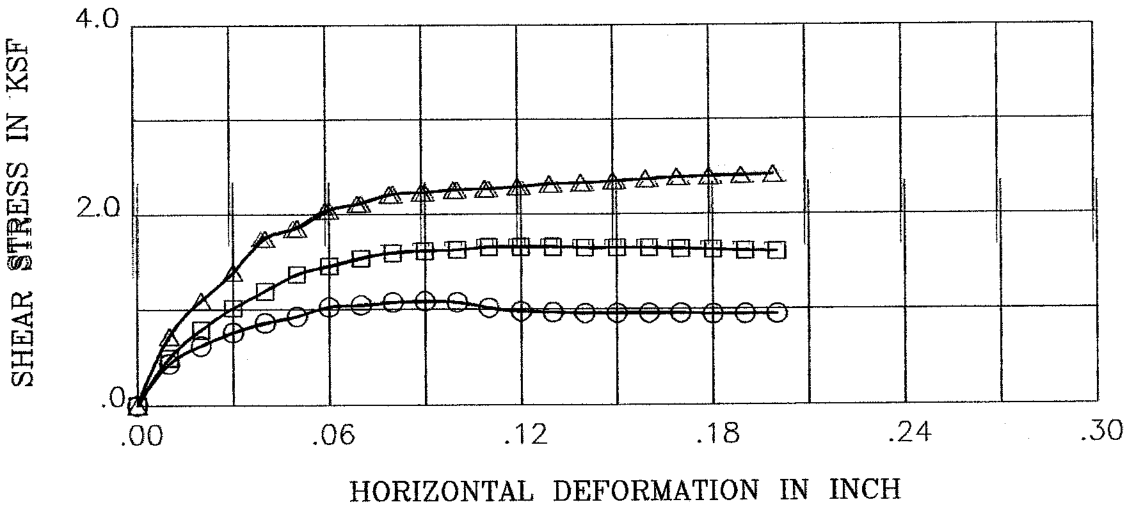
(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	8.9	83.3	.947	1.00	.92	.90
□	9.1	86.2	.883	2.00	1.70	1.60
△	10.4	81.9	.982	3.00	2.41	2.41

Remark :

Project No. 60C508	MAPLE CANYON RESTORATION PROJECT PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	DIRECT SHEAR TEST Figure B-15





BORING/SAMPLE : TP−7 DEPTH (ft) : 4−4.5

DESCRIPTION :

STRENGTH INTERCEPT (C) : .389 KSF

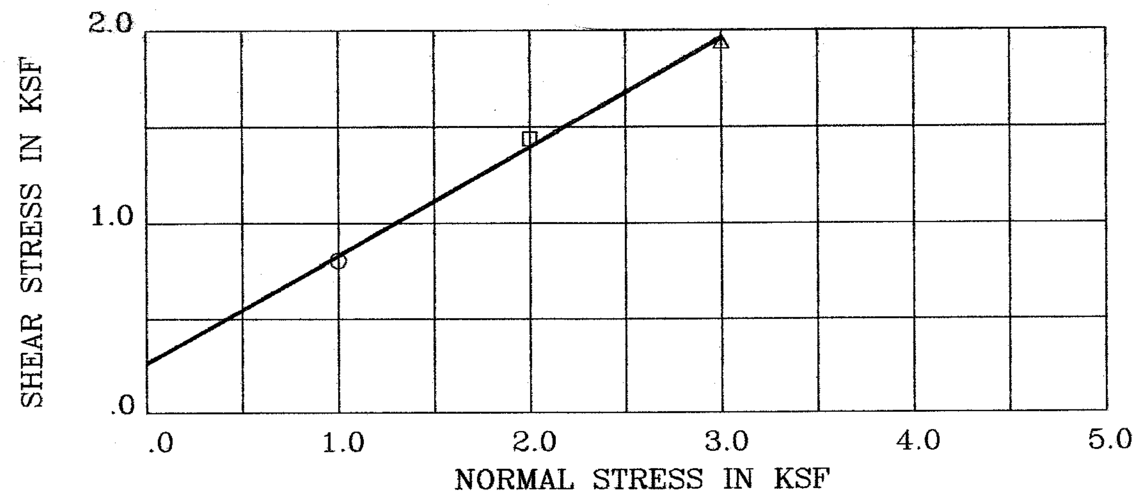
FRICTION ANGLE (PHI) : 33.6 DEG

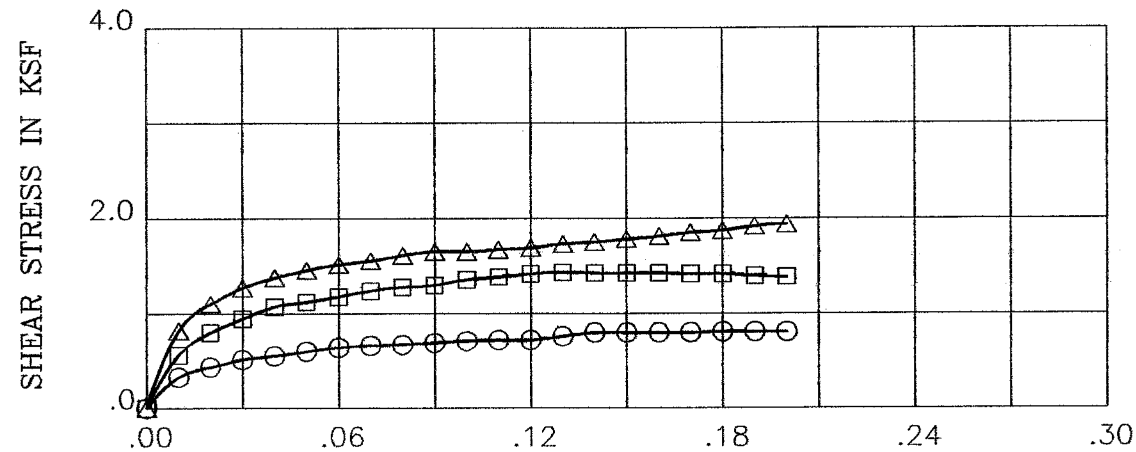
(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	6.5	84.6	.917	1.00	1.08	.95
□	7.5	82.3	.971	2.00	1.66	1.61
△	6.9	85.6	.894	3.00	2.41	2.41

Remark :

Project No. 60C508	MAPLE CANYON RESTORATION PROJECT PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	DIRECT SHEAR TEST Figure B−16





BORING/SAMPLE : TP-8 DEPTH (ft) : 6.5-7

DESCRIPTION :

STRENGTH INTERCEPT (C) : .264 KSF

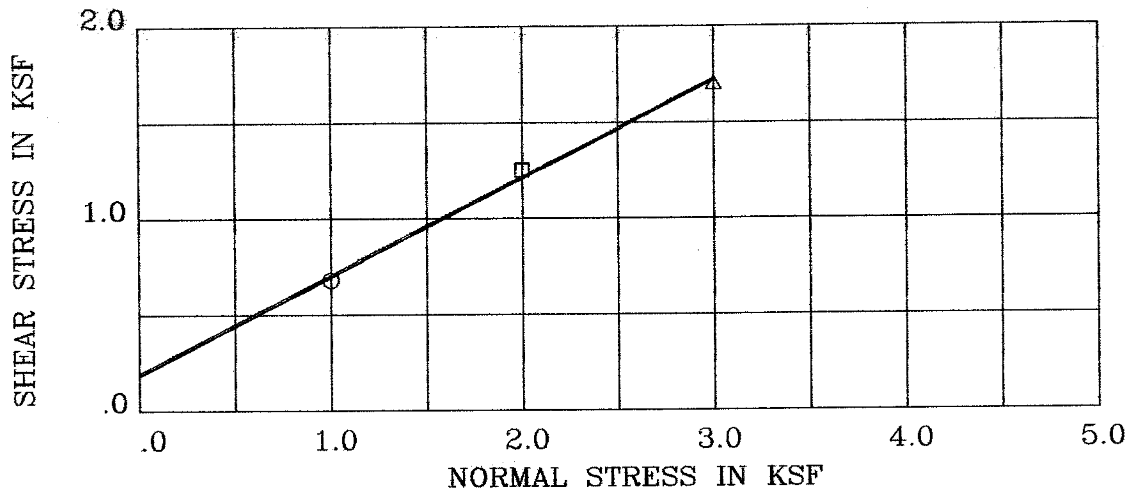
FRICTION ANGLE (PHI) : 29.5 DEG

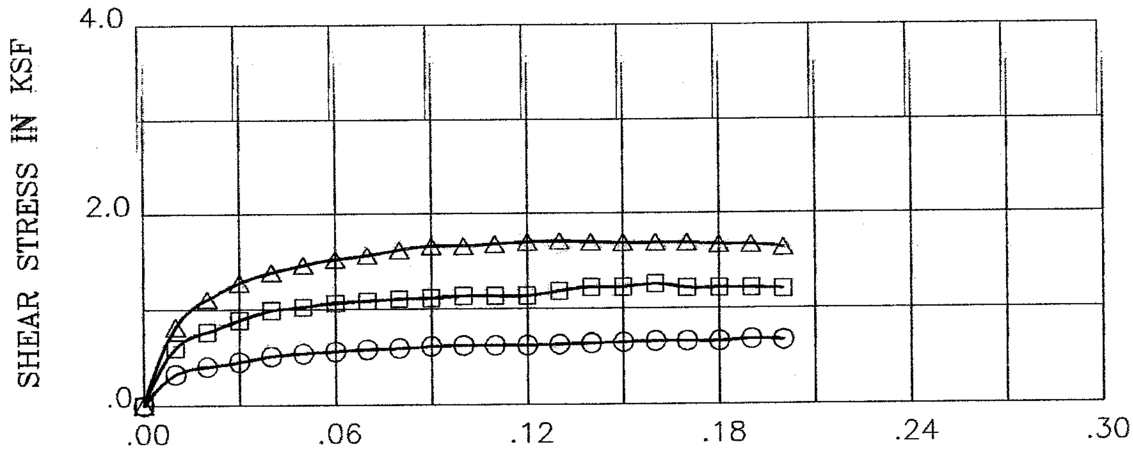
(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	7.5	88.1	.842	1.00	.81	.81
□	7.8	87.3	.859	2.00	1.44	1.39
△	6.8	89.7	.809	3.00	1.94	1.94

Remark :

Project No. 60C508	MAPLE CANYON RESTORATION PROJECT PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	DIRECT SHEAR TEST Figure B-17



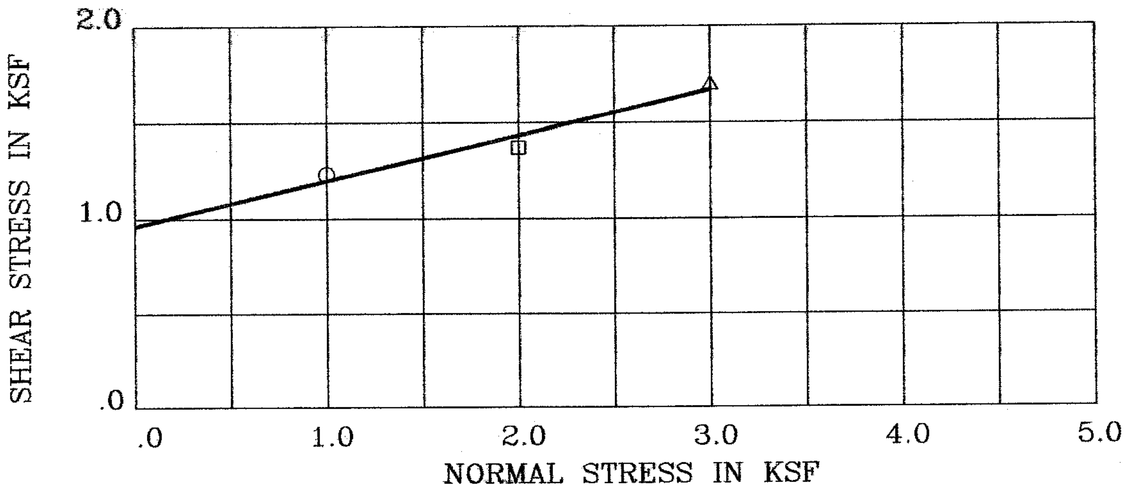


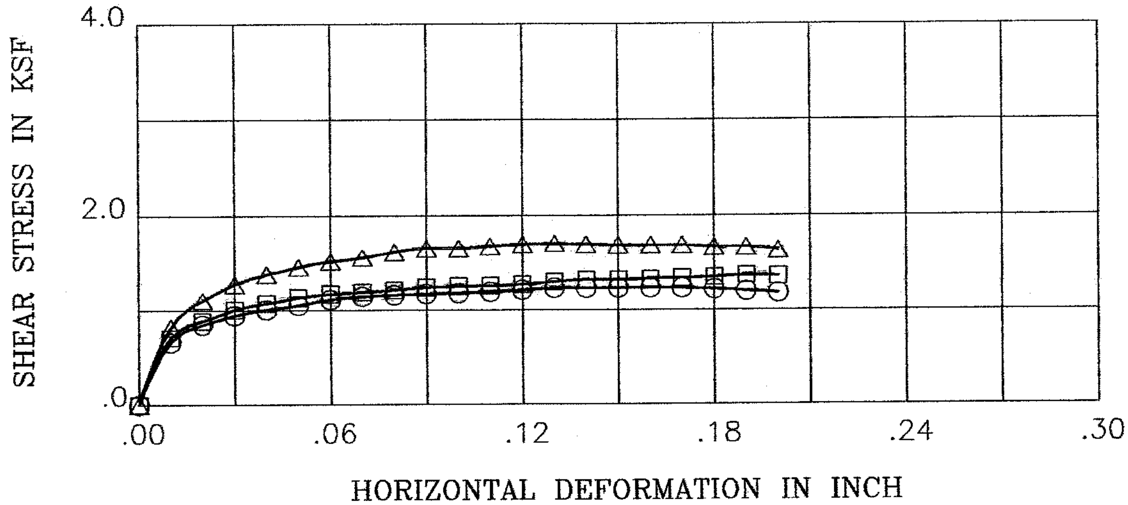
BORING/SAMPLE : TP-10 DEPTH (ft) : 5.5
DESCRIPTION :
STRENGTH INTERCEPT (C) : .189 KSF
FRICTION ANGLE (PHI) : 27.1 DEG
(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	8.8	80.1	1.024	1.00	.68	.67
□	9.0	79.7	1.035	2.00	1.25	1.20
△	11.9	79.0	1.053	3.00	1.70	1.64

Remark :

Project No. 60C508	MAPLE CANYON RESTORATION PROJECT PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	DIRECT SHEAR TEST Figure B-18





BORING/SAMPLE : TP-12 DEPTH (ft) : 10.5-11

DESCRIPTION :

STRENGTH INTERCEPT (C) : .965 KSF

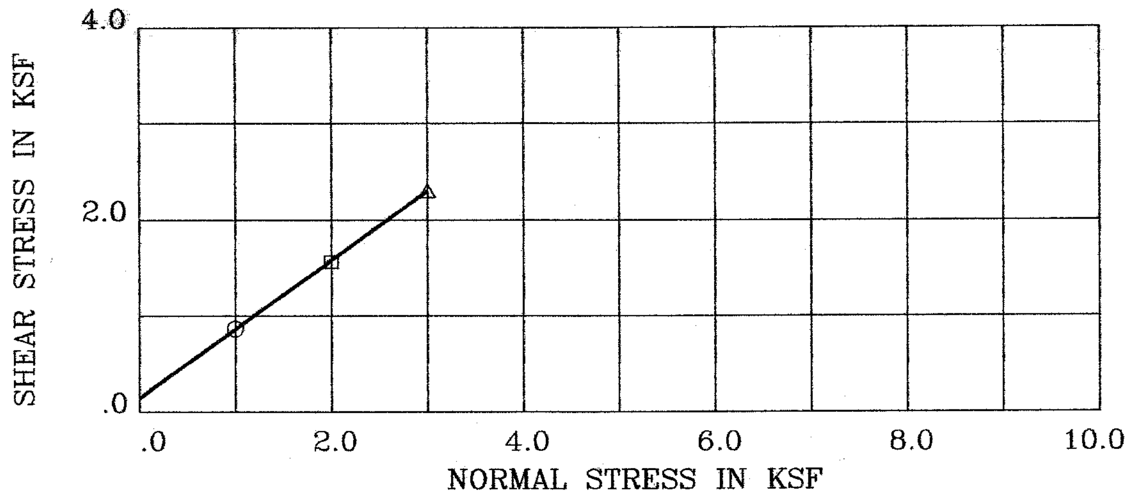
FRICTION ANGLE (PHI) : 13.3 DEG

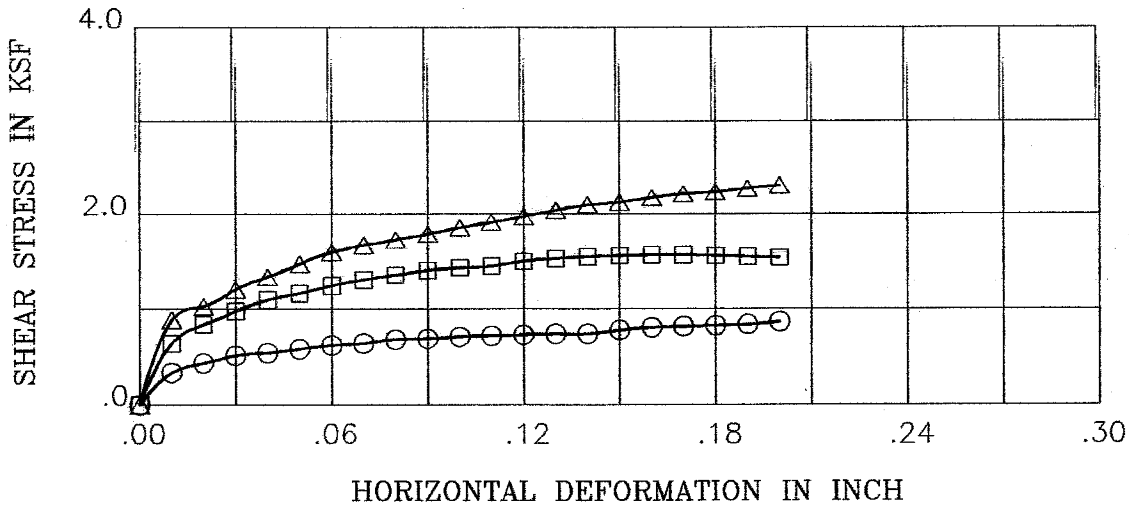
(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	19.6	103.4	.568	1.00	1.23	1.18
□	20.2	96.8	.676	2.00	1.37	1.36
△	11.9	79.0	1.053	3.00	1.70	1.64

Remark :

Project No. 60C508	MAPLE CANYON RESTORATION PROJECT PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	DIRECT SHEAR TEST Figure B-19





BORING/SAMPLE : TP-14 DEPTH (ft) : 9

DESCRIPTION :

STRENGTH INTERCEPT (C) : .142 KSF

FRICTION ANGLE (PHI) : 35.7 DEG

(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	9.6	88.9	.825	1.00	.87	.87
□	10.4	84.7	.915	2.00	1.57	1.54
△	8.1	94.8	.712	3.00	2.31	2.31

Remark :

Project No. 60C508	MAPLE CANYON RESTORATION PROJECT PHASE 1
ALLIED GEOTECHNICAL ENGINEERS, INC.	DIRECT SHEAR TEST Figure B-20

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Date: July 28, 2014
Purchase Order Number: PROJECT# 60C508
Sales Order Number: 23172
Account Number: ALLG
To:
--
Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-1   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
--
One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14 from Maple Canyon Restoration Project# 60C508 marked as TP-1 @ 6'-7'.

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

pH 9.0

Water Added (ml)	Resistivity (ohm-cm)
10	1300
5	1100
5	1200
5	1300
5	1300

32 years to perforation for a 16 gauge metal culvert.
41 years to perforation for a 14 gauge metal culvert.
57 years to perforation for a 12 gauge metal culvert.
73 years to perforation for a 10 gauge metal culvert.
89 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.007% (72 ppm)
Water Soluble Chloride Calif. Test 422	0.006% (64 ppm)
Bicarbonate (as $CaCO_3$) (In a saturated soil paste extract)	49 ppm

Laura Torres

Laura Torres
LT/ram

LABORATORY REPORT

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Sales Order Number: 23172
Account Number: ALLG
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Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-2 Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
--
One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14from Maple Canyon Restoration Project# 60C508 marked as TP-2 @ 6.5'-7'.

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

pH 9.1

Water Added (ml)	Resistivity (ohm-cm)
10	5800
5	3700
5	2500
5	2000
5	1700
5	1300
5	1100
5	1200
5	1300

32 years to perforation for a 16 gauge metal culvert.
41 years to perforation for a 14 gauge metal culvert.
57 years to perforation for a 12 gauge metal culvert.
73 years to perforation for a 10 gauge metal culvert.
89 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.014% (140 ppm)
Water Soluble Chloride Calif. Test 422	0.007% (75 ppm)
Bicarbonate (as $CaCO_3$) (In a saturated soil paste extract)	56 ppm

Laura Torres

Laura Torres
LT/ram

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El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-3   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
--
One soil sample received on 07/17/14 at 9:00am,
taken on 07/16/14 from Maple Canyon Restoration
Project# 60C508 marked as TP-3 @ 6.5'-7'.

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

pH 7.9

Water Added (ml)	Resistivity (ohm-cm)
10	1600
5	920
5	560
5	430
5	300
5	290
5	300
5	320

18 years to perforation for a 16 gauge metal culvert.
24 years to perforation for a 14 gauge metal culvert.
33 years to perforation for a 12 gauge metal culvert.
42 years to perforation for a 10 gauge metal culvert.
52 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 — 0.189% (1890 ppm)

Water Soluble Chloride Calif. Test 422 — 0.047% (470 ppm)

Bicarbonate (as $CaCO_3$) — 44 ppm
(In a saturated soil paste extract)

Laura Torres

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LT/ram

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Sales Order Number: 23172
Account Number: ALLG
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El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-4   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
--
One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14 from Maple Canyon Restoration Project# 60C508 marked as TP-4 @ 6.5'-7'.

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

pH 8.2

Water Added (ml)	Resistivity (ohm-cm)
15	9100
5	8100
5	6200
5	5200
5	4300
5	3500
5	3500
5	3600
5	3800

51 years to perforation for a 16 gauge metal culvert.
66 years to perforation for a 14 gauge metal culvert.
92 years to perforation for a 12 gauge metal culvert.
118 years to perforation for a 10 gauge metal culvert.
143 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.002% (15 ppm)
Water Soluble Chloride	Calif. Test 422	0.002% (21 ppm)

Bicarbonate (as $CaCO_3$) 34 ppm
(In a saturated soil paste extract)

Laura Torres

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Attention: Sani Sutanto

Laboratory Number: SO5359-5   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
--
One soil sample received on 07/17/14 at 9:00am,
taken on 07/16/14 from Maple Canyon Restoration
Project# 60C508 marked as TP-5 @ 6'-6.5'.

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

pH 7.6

Water Added (ml)	Resistivity (ohm-cm)
15	9500
5	7600
5	7100
5	6000
5	4600
5	4200
5	4200
5	4400
5	4500

55 years to perforation for a 16 gauge metal culvert.
72 years to perforation for a 14 gauge metal culvert.
99 years to perforation for a 12 gauge metal culvert.
127 years to perforation for a 10 gauge metal culvert.
154 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	<0.001% (<11 ppm)
Water Soluble Chloride Calif. Test 422	0.002% (21 ppm)
Bicarbonate (as $CaCO_3$) (In a saturated soil paste extract)	26 ppm

Laura Torres

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El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-6   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:

One soil sample received on 07/17/14 at 9:00am,
taken on 07/16/14 from Maple Canyon Restoration
Project# 60C508 marked as TP-6 @ 6.5-7'.

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

pH 7.9

Water Added (ml)	Resistivity (ohm-cm)
15	3500
5	2800
5	2000
5	1500
5	1400
5	1400
5	1300
5	1400
5	1600

34 years to perforation for a 16 gauge metal culvert.
44 years to perforation for a 14 gauge metal culvert.
61 years to perforation for a 12 gauge metal culvert.
78 years to perforation for a 10 gauge metal culvert.
95 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.005% (54 ppm)
Water Soluble Chloride Calif. Test 422	0.010% (96 ppm)
Bicarbonate (as $CaCO_3$) (In a saturated soil paste extract)	23 ppm

Laura Torres

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El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-7 Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:

One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14 from Maple Canyon Restoration Project# 60C508 marked as TP-7 @ 6'-6.5'.

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

pH 8.1

Water Added (ml)	Resistivity (ohm-cm)
30	660
5	510
5	450
5	400
5	400
5	380
5	370
5	400
5	420

20 years to perforation for a 16 gauge metal culvert.
26 years to perforation for a 14 gauge metal culvert.
37 years to perforation for a 12 gauge metal culvert.
47 years to perforation for a 10 gauge metal culvert.
57 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.067% (670 ppm)

Water Soluble Chloride Calif. Test 422 0.053% (530 ppm)

Bicarbonate (as $CaCO_3$) 36 ppm
(In a saturated soil paste extract)

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El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-8   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
--
One soil sample received on 07/17/14 at 9:00am,
taken on 07/16/14 from Maple Canyon Restoration
Project# 60C508 marked as TP-8 @ 8.5'-9'.

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

pH 8.5

Water Added (ml)	Resistivity (ohm-cm)
10	3100
5	1700
5	1200
5	1100
5	1000
5	1100
5	1300

31 years to perforation for a 16 gauge metal culvert.
40 years to perforation for a 14 gauge metal culvert.
55 years to perforation for a 12 gauge metal culvert.
70 years to perforation for a 10 gauge metal culvert.
86 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.010% (100 ppm)
Water Soluble Chloride Calif. Test 422	0.007% (75 ppm)
Bicarbonate (as $CaCO_3$) (In a saturated soil paste extract)	38 ppm

Laura Torres

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LT/ram

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Date: July 28, 2014
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Account Number: ALLG
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Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-9 Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
--
One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14from Maple Canyon Restoration Project# 60C508 marked as TP-9 @ 6'-6.5'.

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

pH 7.7

Water Added (ml)	Resistivity (ohm-cm)
10	1100
5	860
5	630
5	540
5	430
5	370
5	310
5	320
5	330

19 years to perforation for a 16 gauge metal culvert.
25 years to perforation for a 14 gauge metal culvert.
34 years to perforation for a 12 gauge metal culvert.
43 years to perforation for a 10 gauge metal culvert.
53 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.017% (170 ppm)

Water Soluble Chloride Calif. Test 422 0.107% (1070 ppm)

Bicarbonate (as $CaCO_3$) 20 ppm
(In a saturated soil paste extract)

Laura Torres

Laura Torres
LT/ram

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Sales Order Number: 23172
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1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-10   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
--
One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14 from Maple Canyon Restoration Project# 60C508 marked as TP-10 @ 6'-6.5'.

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

pH 8.8

Water Added (ml)	Resistivity (ohm-cm)
10	2000
5	1100
5	920
5	640
5	550
5	470
5	490
5	520
5	540

22 years to perforation for a 16 gauge metal culvert.
29 years to perforation for a 14 gauge metal culvert.
40 years to perforation for a 12 gauge metal culvert.
52 years to perforation for a 10 gauge metal culvert.
63 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.015% (150 ppm)

Water Soluble Chloride Calif. Test 422   0.043% (430 ppm)

Bicarbonate (as $CaCO_3$)   38 ppm
(In a saturated soil paste extract)

Laura Torres

Laura Torres
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To:

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1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-11   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:

One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14 from Maple Canyon Restoration Project# 60C508 marked as TP-11 @ 6.5'-7'.

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

pH 7.8

Water Added (ml)	Resistivity (ohm-cm)
10	1800
5	1200
5	690
5	500
5	500
5	450
5	440
5	470
5	490

22 years to perforation for a 16 gauge metal culvert.
28 years to perforation for a 14 gauge metal culvert.
39 years to perforation for a 12 gauge metal culvert.
50 years to perforation for a 10 gauge metal culvert.
61 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.023% (230 ppm)
Water Soluble Chloride Calif. Test 422	0.069% (690 ppm)
Bicarbonate (as $CaCO_3$) (In a saturated soil paste extract)	31 ppm

Laura Torres

Laura Torres
LT/ram

LABORATORY REPORT

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

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

Date: July 28, 2014
Purchase Order Number: PROJECT# 60C508
Sales Order Number: 23172
Account Number: ALLG
To:
--
Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-12 Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
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One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14 from Maple Canyon Restoration Project# 60C508 marked as TP-12 @ 6'-6.5'.

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

pH 8.5

Water Added (ml)	Resistivity (ohm-cm)
10	1200
5	750
5	500
5	450
5	430
5	410
5	420
5	430
5	450

21 years to perforation for a 16 gauge metal culvert.
28 years to perforation for a 14 gauge metal culvert.
38 years to perforation for a 12 gauge metal culvert.
49 years to perforation for a 10 gauge metal culvert.
59 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.042% (420 ppm)
Water Soluble Chloride Calif. Test 422	0.030% (300 ppm)
Bicarbonate (as $CaCO_3$) (In a saturated soil paste extract)	40 ppm

Laura Torres

Laura Torres
LT/ram

L A B O R A T O R Y   R E P O R T

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C L A R K S O N   L A B O R A T O R Y   A N D   S U P P L Y   I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L   A N D   C O N S U L T I N G   C H E M I S T S

Date: July 28, 2014
Purchase Order Number: PROJECT# 60C508
Sales Order Number: 23172
Account Number: ALLG
To:
--
Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-13   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
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One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14 from Maple Canyon Restoration Project# 60C508 marked as TP-13 @ 6'-7'.

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

pH 8.2

Water Added (ml)	Resistivity (ohm-cm)
10	8000
5	3600
5	3200
5	2800
5	1900
5	3500
5	4100

40 years to perforation for a 16 gauge metal culvert.
52 years to perforation for a 14 gauge metal culvert.
72 years to perforation for a 12 gauge metal culvert.
91 years to perforation for a 10 gauge metal culvert.
111 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.011% (110 ppm)

Water Soluble Chloride Calif. Test 422   0.012% (120 ppm)

Bicarbonate (as $CaCO_3$)   50 ppm
(In a saturated soil paste extract)

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Date: July 28, 2014
Purchase Order Number: PROJECT# 60C508
Sales Order Number: 23172
Account Number: ALLG
To:

Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-14 Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:

One soil sample received on 07/17/14 at 9:00am, taken on 07/16/14 from Maple Canyon Restoration Project# 60C508 marked as TP-14 @ 7.5'-8'.

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

pH 8.5

Water Added (ml)	Resistivity (ohm-cm)
10	2900
5	1500
5	770
5	540
5	530
5	490
5	480
5	530
5	550

23 years to perforation for a 16 gauge metal culvert.
29 years to perforation for a 14 gauge metal culvert.
41 years to perforation for a 12 gauge metal culvert.
52 years to perforation for a 10 gauge metal culvert.
63 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.042% (420 ppm)

Water Soluble Chloride Calif. Test 422 0.053% (530 ppm)

Bicarbonate (as $CaCO_3$) 130 ppm
(In a saturated soil paste extract)

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Date: July 28, 2014
Purchase Order Number: PROJECT# 60C508
Sales Order Number: 23172
Account Number: ALLG
To:
--
Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO5359-15 Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
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One soil sample received on 07/17/14 at 9:00am,
taken on 07/16/14 from Maple Canyon Restoration
Project# 60C508 marked as TP-15 @ 6'-6.5'.

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

pH 6.8

Water Added (ml)	Resistivity (ohm-cm)
10	4900
5	3100
5	2300
5	1700
5	1700
5	1500
5	1600
5	4800

21 years to perforation for a 16 gauge metal culvert.
27 years to perforation for a 14 gauge metal culvert.
38 years to perforation for a 12 gauge metal culvert.
48 years to perforation for a 10 gauge metal culvert.
59 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.004% (36 ppm)

Water Soluble Chloride Calif. Test 422 0.003% (32 ppm)

Bicarbonate (as $CaCO_3$) 20 ppm
(In a saturated soil paste extract)

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