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Preliminary Geotechnical Investigation

City of San Diego Task 15GT15 – Manzanita Canyon Water and Storm Drain Group 968

Manzanita Drive & 39th Street,
San Diego, California

Prepared for:

City of San Diego
525 B Street, Suite 750 (MS 908A)
San Diego, CA 92101

September 28, 2018

Project No.: 180325.2

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September 28, 2018
Project No. 180325.2

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Subject: Preliminary Geotechnical Investigation
Task 15GT15 – Manzanita Canyon Water and Storm Drain Group 968
Manzanita Drive & 39th Street
San Diego, California

Dear Mr. Magdedi,

In accordance with your request and authorization, we are presenting the results of our geotechnical engineering evaluation for the above-referenced project in the City Heights area of San Diego, California. The purpose of this investigation was to evaluate the subsurface conditions at the proposed water pipeline locations and to provide geotechnical engineering recommendations for the Manzanita Canyon Water Group 968 project.

Please note that the recommendations presented within the report are based on assumptions stated herein. Should conditions encountered during installation and construction differs from those assumed in our analyses, or should the proposed project change, our recommendations may need to be modified accordingly.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report, or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
TWINING, INC.

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1. INTRODUCTION

This report presents the results of our preliminary geotechnical investigation performed for the Manzanita Canyon Water and Storm Drain Group 968 within the City Heights area of San Diego, California. The approximate locations of the proposed sewer pipelines are shown in Figure 1, Project Location Map. The purpose of this study was to evaluate the subsurface conditions at the project site and evaluate the feasibility of using trenchless methods and to provide geotechnical engineering recommendations for the design and construction of the proposed water main installation.

2. PROJECT DESCRIPTION

According to the information presented in the construction plans prepared by the City of San Diego *Plans for the Construction of Water and Storm Drain Group 968*, (Sheets 38719-01-D, 38719-20-D, and 38719-21-D), undated, the water main portion of the project (this project) consists of the installation of a new 12-inch pipeline between the existing 10-inch water main at the north side of cul-de-sac of Manzanita Drive and the existing 12-inch water main at the end of 39th street. According to the provided plans, the proposed water line will replace an existing 12-inch AC water pipeline between Manzanita Drive and 39th Street using both open trench and trenchless installation methods. The open trench method is proposed for the 12-inch diameter pipe between Stations 6+40.00 and 8+69.36. A trenchless method will be used for a 12 inch diameter pipe between Stations 3+64.97 and 6+40.00. From Stations 1+00.00 to 3+64.97 the Cured-In-Place Pipe (CIPP) method will used to upgrade the existing 12-inch diameter water main. The depth of the proposed waterline installation along the alignment ranges from 2.5 feet to 4.25 feet below ground surface.

3. SITE DESCRIPTION

The Manzanita Canyon Water and Storm Drain Group 968 project is located within the City Heights area of San Diego. The area is characterized by uneven topography and with slopes varying from 1:30 (V:H) to 1:2 (V:H). In general, the project vicinity is adjacent to a residential area with single- family homes and multi-family dwellings, paved streets, sidewalks, and parks with an open-space canyon. Most of the proposed alignment lies on the Manzanita canyon and adjacent slope areas which are densely vegetated and a canyon bottom with a sandy streambed with abundant gravels and cobbles. The alignment has elevation ranges from 181 feet to 277 feet above from mean sea level (MSL). Review of historical aerial photographs indicates that the majority of the pipe alignment is within a previously existing canyon drainage that was subsequently surrounded by development. Latitudes for the site range from 32.7336N to 32.7351N and longitudes ranges from -117.1094W to -117.1108W.

4. SCOPE OF SERVICES

Our scope of services for this project consisted of the following:

- Review of readily available background data, including project plans provided by the City of San Diego, in-house geotechnical data, geotechnical literature, and, geologic and topographic maps relevant to the project (see Section 11, Selected References).
- Discussion with City of San Diego representatives to select locations for 3 borings and 2 hand-excavated trench pits for the subsurface investigation.
- Obtaining Traffic Control Permit from the City of San Diego Traffic Control Department.
- Performance of a site reconnaissance to observe the general surface conditions at the project

site and mark out the boring locations.

- Notification of Underground Service Alert (USA) a minimum of 72 hours prior to excavation.
- Performance of a subsurface evaluation consisting of drilling and sampling three exploratory borings and two hand-excavated trench pits.
- Laboratory testing on selected bulk and relatively undisturbed samples to evaluate the geotechnical engineering properties of the on-site soils.
- Review and analysis of data collected from our site reconnaissance, subsurface explorations, and laboratory testing. Specifically, our analyses included the following:
  - Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials;
  - Evaluation of current and historical groundwater conditions at the site and potential impact on design and construction;
  - Evaluation of project feasibility and suitability of on-site soils for fill materials;
  - Development of general recommendations for earthwork, including requirements for placement of compacted fill; and,
  - Recommendations for temporary excavations, shoring design and trenchless installation.
- Preparation of this report summarizing the results of our findings and presenting our conclusions and geotechnical recommendations to assist in the design and construction of the proposed improvements.

5. FIELD EXPLORATION AND LABORATORY TESTING

5.1. Field Exploration

The field exploration was performed on August 28th, 2018. The subsurface conditions were evaluated by drilling three borings to approximate depths ranging from 10.8 feet to 19.2 feet below existing ground surface (bgs). The borings were drilled using a CME-75 truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers. Additionally, two test pits were excavated on the toe of the slope on the canyon stream bed edges. The test pits were manually excavated to depths of 4.5 feet below existing ground surface (bgs). The approximate locations of the exploratory borings and test pits are shown on Figure 2, Boring Location Map. The logs of borings are presented in Appendix A, Field Exploration.

Relatively undisturbed samples were obtained using a modified California split spoon sampler. Standard Penetration Tests (SPTs) were performed to obtain disturbed soil samples using a split barrel sampler. The samplers were driven using a 140-pound, automatic-drop hammer falling approximately 30 inches. The blow counts were recorded and the materials encountered in the borings were logged by Twining's geologist who was assisted by a senior staff engineer. The number of blows required to drive the sampler 12 inches was recorded and are presented on the boring logs in Appendix A. After completion, the borings were backfilled in accordance with San Diego County Department of Environmental Health (SDCDEH) requirements and the street borings were capped with rapid-set concrete with black dye. Twining's geologist also mapped the geologic boundaries observed during trench pit excavation. Trench test pits were backfilled with the soil cuttings.

5.2. Laboratory Testing

Laboratory tests were performed on selected samples obtained from the borings in order to aid in the soil classification and to evaluate the engineering properties of the soils. The laboratory tests included: in-situ moisture and dry density, maximum density, Atterberg limits, sieve analyses, direct shear and corrosivity evaluation. In-situ moisture content and density data are presented on the boring logs in Appendix A. A description of the laboratory tests performed as well as the test results are shown in Appendix B.

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6. GEOLOGY AND SUBSURFACE CONDITIONS

6.1. Regional Geologic Setting

The site is located in the Peninsular Ranges Geomorphic Province (PRGP) of California. The Peninsular Range Province is characterized by northwest trending mountain ranges separated by a series of sub-parallel fault zones associated with the San Andreas Fault System. Within the PRGP, the mountain ranges generally consist of Cretaceous igneous rocks of the Peninsular Ranges Batholith and Jurassic metasediments and metavolcanics, and the topographically lower areas in the coastal region typically consist of marine and terrestrial sedimentary rocks (Kennedy and Peterson, 1975). In the coastal region of San Diego County, Quaternary and late Tertiary age folding and tilting has occurred in areas adjacent to the active Rose Canyon fault zone and a few randomly oriented and scattered small scale faults exist throughout the region (Kennedy and Peterson, 1975; Treiman, 1993; Tan and Kennedy, 2008). The site is located within the PRGP coastal region.

6.2. Tectonic Setting

The tectonic setting of the San Diego is influenced by plate boundary interaction between the Pacific and North American lithospheric plates. This crustal interaction occurs along a broad zone of northwest-striking, predominantly right-slip faults that span the width of the Peninsular Ranges and extend offshore into the California Continental Borderland Province. At the latitude of San Diego (project site), this extends from the San Clemente fault zone, located approximately 54 miles southwest offshore of the San Diego coastline, to the San Andreas fault, located about 85 miles northeast of San Diego (California Geological Survey, 2010).

Geologic, geodetic, and seismic data indicate that the faults along the eastern margin of the plate boundary, including the San Andreas, San Jacinto, and Imperial faults, are currently the most active. These active faults are located in the Imperial Valley and are the dominant structures in accommodating the majority of motion between the two adjacent plates. A smaller portion of the relative plate motion is being accommodated by northwest-striking active faults to the west, including the Elsinore, Newport-Inglewood-Rose Canyon, and offshore faults. The offshore faults include the Coronado Bank, San Diego Trough, and San Clemente faults zones.

6.3. Site Geology and Subsurface Conditions

The project site is underlain by artificial fill, Quaternary-aged colluvium and alluvium, and dense sand with abundant gravel/cobble associated with the Quaternary-aged Very Old Paralic Deposits ($Qvop_8$). Dense sand associated with the Tertiary-aged San Diego Formation was also encountered. These materials have been mapped by Kennedy (1975) and Kennedy and Tan (2008). At the exploratory boring locations, the alluvial and formational materials are mantled by artificial fill soils likely associated with residential streets and utility construction. The regional geology is presented in Figure 3. The geologic units observed are described below from youngest to oldest.

6.3.1. Artificial Fill (Unmapped)

Artificial fill was encountered in the upper portions of the borings (B-1 through B-3). At the boring locations the fill soils were generally composed of reddish brown to brown, silty to clayey sand with gravel. The fill encountered was generally moist and dense. The thickness of fill encountered was approximately 3 to 5.75 feet. A portion of the fill is considered suitable for reuse as backfill for trench cut and cover methods provided the fill is screened of over-sized cobbles.

6.3.2. Colluvium and Alluvium (Unmapped)

Colluvial and alluvial (referred herein as alluvium) soils were encountered at trench pits extending to depths ranging from 0 feet to 3. 5 to 4.5 feet bgs. The alluvium generally consisted of light brown to dark brown, damp to moist, silty sand to sandy gravel. The alluvium is generally loose to medium dense, with few to abundant gravels and cobbles. The alluvium is underlain by formational sedimentary units (San Diego Formation), as noted below. The depth of alluvial soils in the center of the canyon bottom is unknown. Note that cobbles in the area of the borings were up to 8 inches in diameter.

6.3.3. Very Old Paralic Deposits ($Qvop_8$)

The geologic map prepared by Kennedy and Tan (2008) indicate that the borings on the streets area are underlain by very old paralic deposits. These sediments were previously assigned to the more extensive Lindavista Formation (Kennedy, 1975). The Lindavista Formation is distinguished from other similar sedimentary units by its orange brown color and cemented resistant nature. Some of the other features of this formation are as follows: poorly sorted, moderately permeable, interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate. Materials typically associated with very old paralic deposits (Lindavista Formation) were observed in our borings B-1 through B-3. These deposits consisted of dense to very dense, yellowish-brown to reddish-brown, silty sand with abundant gravel and cobble. This unit was difficult to drill. Due to the sampler size, the size of the cobble could not be determined but the fractured (broken) gravel indicates that the cobble size may range from 4 to 8 inches. The cobbles of the Lindavista Formation were also observed on the exposed slopes surrounding the borings.

6.3.4. San Diego Formation (Tsd)

The materials encountered at the canyon portion of the site are described by Kennedy (1975) and Kennedy and Tan (2008) as the San Diego Formation. In their study they described this formation as predominantly yellowish brown and gray, fine- to medium-grained, poorly indurated fossiliferous marine sandstone (Tsdss) and reddish-brown, transitional marine and nonmarine pebble and cobble conglomerate (Tsdcg). In part of the area the sandstone and conglomerate are undivided (Tsd).

At the trench pit locations (TP-1 and TP-2), this sedimentary unit was composed of dense, moist, dark orange brown, silty sand with gravel. The San Diego Formation was weakly cemented. We encountered difficult excavation conditions in the trench pits.

6.4. Groundwater

No groundwater or seepage was encountered in the borings at the time of field exploration. The depth of the regional groundwater table beneath the project site is unknown but may be assumed to be in excess of 50 feet bgs. However, localized shallow perched water conditions may occur, particularly during the wet (rainy) season. Perching would most likely be encountered in fill materials or alluvium above the contact with the relatively impermeable formational materials. Pipe leaks, overflows, and landscape irrigation could also potentially contribute to groundwater perching.

It should be noted that any required construction operations could change surface drainage patterns and/or reduce permeability due to the densification of compacted soils. Such changes of surface and subsurface hydrologic conditions, plus irrigation of landscaping or significant increases in rainfall, may result in the appearance of surface or near-surface water at locations where none existed previously. The damage from such water is expected to be localized and cosmetic in nature, if good positive drainage is implemented during and at the completion of construction.

It must be understood that unless discovered during site exploration or encountered during site construction operations, it is extremely difficult to predict if or where perched or true groundwater conditions may appear in the future. When site fill or formational soils are fine-grained and of low permeability, water problems may not become apparent for extended periods of time.

6.5. Geologic Hazards

Geologic hazards at the site are essentially related to those caused by earthquakes. The major cause of damage from earthquakes is fault rupture and strong shaking from seismic waves. Potential geologic hazards that could affect the project site are discussed below.

6.5.1. Faulting

The southern California region has long been recognized as being seismically active. Seismic activity results from a number of active faults that cross the region, all of which are related to the San Andreas transform system which covers a broad zone of right lateral faults that extend from Cape Mendocino to Baja California. Faults in Southern California are classified according to their activity as active, potentially active, and inactive faults. Active faults are those faults that have had surface displacement within Holocene time (approximately the last 11,700 years). Faults are considered potentially active if they show evidence of surface displacement since the beginning of Quaternary time (about 1.6 million years ago), but not since Holocene time.

The site is not within a currently established State of California Alquist-Priolo Earthquake Fault Zone for fault rupture hazard (formerly Special Studies Zones for fault rupture hazard). Based on a review of geologic literature, no active or potentially active faults are known to occur beneath the project site. Accordingly, it appears that there is little probability of surface rupture due to faulting beneath the site. There are, however, several faults located in sufficiently close proximity that movement associated with them could cause significant ground motion at the site as shown in Figure 4, Fault Location Map.

Regional active faults that occur near the City Heights area include the Rose Canyon fault zone, the offshore Coronado Bank and San Diego Trough fault zones to the west, the Elsinore and San Jacinto fault zones to the east, and the San Miguel-Vallecitos and Agua Blanca fault zones to the south in Mexico. Locally, the Rose Canyon fault zone trends north-northwest through downtown San Diego and the San Diego Bay. The closest known active faults to the site are the Rose Canyon fault zone located approximately 3.5 miles to the west, the Coronado Bank fault zone located 15 miles to the southwest and the Newport-Inglewood fault zone located 4.75 miles northwest. The closest known potentially active faults to the site include the Texas Street fault approximately 1.2 miles to the west and the La Nacion fault system approximately 1.8 miles to the east.

6.5.2. Earthquake Ground Motion

The project area may be subject to strong ground shaking in the event of an earthquake; however this hazard is common to Southern California and the effects on the proposed project can be mitigated if the improvements are designed and constructed in accordance with current engineering practice and building codes.

6.5.3. Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The potential for seismically induced liquefaction is greatest where shallow groundwater and poorly consolidated, well sorted, fine grained sands and silts are present. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and grain size characteristics, relative density of the soil, initial confining pressure, and intensity of duration of ground shaking.

Fill soils that are about 2 to 5.75 feet in thickness cover the project site within the street areas. These materials are composed of medium dense to dense, silty sand and clayey sand with some gravel and cobble. Beneath the fill, the formational materials consist of dense to very dense sands with gravel and cobble. Groundwater was not encountered within the depths drilled. Accordingly, the potential for liquefaction in the event of a strong to moderate earthquake on a nearby fault is considered low, with the exception of the canyon bottom containing young sandy alluvium where the potential is considered moderate.

6.5.4. Seismic Settlement

Seismic settlement occurs when dry to saturated, loose to medium dense granular soils densify during ground shaking. Due to lithologic variations, such settlement can differ across a site. Differential settlement may also be induced by ground failures, such as liquefaction, flow slides, and surface ruptures. The potential for seismic settlement in the fill and alluvial materials is considered low to moderate, respectively. The potential for seismic settlement in dense formational materials is very low.

6.5.5. Landslides and Slope Stability

No evidence indicating the presence of deep seated landslides was observed on or in the immediate vicinity of the site. The sedimentary units exposed within the vicinity of the project area appeared to exhibit nearly horizontal bedding (Kennedy and Tan, 2008). The potential for deep seated slope stability problems at the site is considered low. There is, however, the potential for shallow sloughing and slumping of surficial slope materials such as colluvium exposed on the canyon slopes; pipes should be imbedded below these materials, as discussed in the report. In addition, the site is mapped in Landslide Susceptibility Area “3-1” – Generally Susceptible (Tan, 1995). Per Tan, although most slopes within subarea 3-1 do not currently contain landslide deposits, they can be expected to fail, locally, when the slopes are steep or adversely modified.

6.5.6. Seismic Safety Study

The City of San Diego Seismic Safety Study designates the project area as "Zone 53: Level or sloping terrain, unfavorable geologic structure. Low to moderate risk."' as shown in Figure 5, Seismic Safety Map.

6.6. Seismic Design Parameter

The project area is located approximately between latitudes 32.7336N to 32.7351N and longitudes -117.1094W to -117.1108W. The materials beneath the site consist of dense fill; loose to medium dense colluvium/alluvium; and underlain in some portions, dense to very dense formational materials.

Based on the results of our field investigation, the applicable Site Class is D, consisting of a stiff soil profile with average SPT N values between 15 and 50 blows per foot. Table 2 presents seismic design parameters for the site in accordance with 2016 CBC and mapped spectral acceleration parameters (United States Geological Survey, 2016).

Table 1
2013 California Building Code Design Parameters

Design Parameter	Value
Site Class	D
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s	1.044g
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1	0.399g
Site Coefficient, F_a	1.083
Site Coefficient, F_v	1.603
Adjusted MCE_R[1] Spectral Response Acceleration Parameter at Short Period, S_{MS}	1.130g
1-Second Period Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1}	0.639g
Short Period Design Spectral Response Acceleration Parameter, S_{DS}	0.753g
1-Second Period Design Spectral Response Acceleration Parameter, S_{D1}	0.426g
Peak Ground Acceleration, PGA_M[2]	0.468g
Seismic Design Category	D

Notes: [1] Risk-Targeted Maximum Considered Earthquake
[2] Peak Ground Acceleration adjusted for site effects

7. CONCLUSIONS

Based on the results of our subsurface evaluation, laboratory testing, and data analysis, construction of the proposed improvements is feasible from a geotechnical standpoint, provided the recommendations of this report are incorporated in the design and construction of the project. Geotechnical considerations include the following:

- The site is locally underlain by 3 to 5 ½ feet of dense fill soils overlying formational deposits. The canyon area is underlain by 2 to 3 feet of loose to medium dense colluvial/alluvial soil, underlain by dense sands with gravels and cobbles associated with San Diego Formation. Refusal on cobbles was encountered in boring B-2 at a depth of 12', and 4 ½ feet in test pit TP-2. Depth of alluvial soils in the center of the canyon bottom (near the middle of the project) is unknown.

- The majority of the fill and alluvium is suitable for re-use as compacted fill, however, oversize materials within all of the sediments encountered, will need to be screened.

- On-site materials are considered generally excavatable with conventional heavy-duty earth moving construction equipment. Difficult excavation is anticipated within locally strongly cemented formational materials and cobble zones. The strongly cemented zones, although not encountered, are characteristics of the formation materials. The installation systems and drilling equipment used should be designed for the anticipated subsurface conditions.

- Implementation of appropriate method of trenchless system is vital as the subsurface condition is not suitable for all trenchless technology.

- Groundwater was not encountered within the boring locations. Transitory localized seepage may occur at the geologic contacts due to season, rainfall, irrigation practices, and other factors.

- Sieve analysis presented in this report is solely dependent on the material captured in the sampler but abundance of cobble up to 8" was visible all through the alignment. Considering the size of cobble and hardness of cobble, a larger fraction of coarse fragment during construction should be anticipated than that of testing results.

- Based on review of readily available geologic literature, active or potentially active faults do not cross the subject site. Accordingly, the possibility of surface rupture at the site due to faulting is considered low.

- The potential for seismically induced seismic settlement is moderate to low in the fill and alluvial soils and very low in formational materials.

- Based on Caltrans (2015) corrosion criteria, the project site would be classified as a non-corrosive site for concrete.

8. RECOMMENDATIONS

8.1. General

Based on the results of our field exploration, laboratory testing, and engineering analyses, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and are implemented during construction. The following sections present our conclusions and recommendations pertaining to the geotechnical engineering design for this project.

8.2. Site Preparation

All exposed temporary excavation bottoms (for cut and cover, or pit excavation construction) should be observed and accepted by the geotechnical engineer or engineering geologist prior to construction of the sewer and water lines and prior to any fill placement. Unstable excavation bottoms such as loose fill, colluvial and alluvial soils, may require additional removal to expose competent, non-yielding earth materials.

Vegetation, debris, organics and oversized materials greater than 6 inches in maximum dimension should be separated from on-site soil and legally disposed of off-site prior to placement of any compacted fill. If imported fill materials are needed on the site, they should have a very low expansion potential (expansion index not greater than 20). Proposed import materials should be evaluated and approved by the geotechnical engineer prior to use at the site. For further information about fill materials please see section 8.4 Materials for Fill. Alternatively, gravel and geotextile fabrics may be used to stabilize the bottom of excavations when saturated or unstable materials are exposed within the excavation depth.

8.3. Excavation Characteristics

The results of our field exploration indicate that the project alignment is underlain by undocumented fill and alluvium, and gravel/cobble conglomerate with silt/clay sand matrix associated with the Very Old Paralic Deposits and San Diego Formation. Areas of difficult drilling and refusal were encountered at depths of 16.5', 10.8' and 4.5' in borings B-1 and B-2, and test pit TP-2, respectively.

Excavations in fill and weakly cemented formational materials should generally be feasible using heavy-duty earth moving equipment in good working condition. Construction debris, loose soils, caving and/or sloughing conditions may occur when excavating within undocumented fill and loose portions of alluvium. Difficult excavation is anticipated within gravels and cobbles of the underlying formational materials, when encountered. Excavations in these materials may entail the use of heavy ripping or rock breakers.

8.4. Materials for Fill

On-site soils with "low" expansion potential (expansion index of 50 or less) and organic content of less than 3 percent by volume (or 1 percent by weight) are suitable for use as fill. Fill soil should not contain contaminated materials, rocks, lumps over 4 inches in largest dimension, or more than 40 percent larger than 0.75 inch. Utility trench backfill material should not contain rocks or lumps over 3 inches in largest dimension. Larger chunks, if encountered during excavation, may be broken into acceptably sized pieces or may be disposed offsite. Any imported fill material should consist of "very low" expansion index (expansion index of 20 or less) granular soil. Import material should also have low corrosion potential (Chloride content less than 500 parts per million [ppm], soluble sulfate content of less than 0.1 percent, and pH of 5.5 or higher). Materials to be used as fill should be evaluated by a Twining representative prior to importing or filling. Cuttings generated from drilling operations will not be suitable as fill below any structure, pavements and should be exported offsite.

8.5. Temporary Excavations

The upper portion of on-site materials are loose to medium dense. Temporary un-surcharged excavation sides may be sloped back at an inclination of 1½:1 (horizontal to vertical). Personnel from Twining, Inc. should observe the excavations so that any necessary modifications based on the encountered soil conditions can be recommended. Localized sections of open trench, such as in the canyon bottom, will likely require flatter inclinations for side slopes.

Barricades should be placed around temporary excavations so that vehicles and storage loads do not encroach within 10 feet of the top of excavated slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Twining, Inc. should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If temporary construction slopes are to be maintained during the rainy season, we recommend that berms be graded along the top of slopes in order to prevent runoff water from entering the excavation and eroding slope faces.

All excavations should be performed in accordance with CalOSHA requirements. Vertical excavations will require temporary shoring/shielding. Design recommendations for temporary shoring are presented in the following section.

8.6. Temporary Shoring

Temporary excavations to maximum depths of 6 feet are anticipated for jacking pit and shoring pit for Jack and Bore method. Shoring will be necessary for vertical excavations that are greater than 4 feet in depth, where there is the potential for caving soils or for support of adjacent buried utilities. Shoring should be maintained throughout the installation. When supporting adjacent improvements, sheeting and/or shoring should be installed to prevent loss of support and/or significant settlement.

For design of cantilevered shoring with heights of 15 feet or less a triangular distribution of lateral earth pressure may be used. If the soils behind the shoring are level and groundwater is below the bottom of the excavation, an equivalent fluid pressure of 44 pounds per cubic foot may be assumed for design. Where movement is not acceptable, we recommend that the shoring be designed for an "at rest" pressure of 66 pounds per cubic foot. Some surface settlement should be anticipated during shoring installation especially within the loose to medium dense fill soils.

Surcharge from live loads including traffic and dead loads including adjacent structures that are located within a 1:1 (horizontal to vertical) plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures. The lateral contribution of uniform surcharge loads located immediately behind the temporary shoring may be calculated by multiplying the vertical surcharge pressure by 0.35. Lateral load contributions of surcharge loads behind the shored wall may be provided once the load configurations and layouts are known. As a minimum, 250 pounds per square foot vertical uniform surcharge is recommended to account for nominal construction and/or traffic loads.

8.7. Trenchless Installation

According to our construction plans provided by City, we understand that the existing 12-inch diameter water main will be replaced with 12-inch pipe using appropriate trenchless methods. The selection of the installation method will depend on the length of the reach, the surface and subsurface conditions, and the alignment tolerances for the pipes to be installed. Our recommendations are based on our understanding of the proposed project, the results of the site reconnaissance, field explorations and laboratory testing completed for this investigation.

8.7.1. Microtunneling

This method uses a remote controlled microtunnel boring machine that provides continuous support to the tunnel face. Sections of pipe are jacked behind the tunneling machine which is used as casing during pipeline installation. Soil cuttings are removed through the casing pipe to the sending pit using augers or conveyors. While microtunneling provides control of alignment, large set-up areas are required. The greatest concern using microtunneling is the presence of obstructions such as cobbles and debris. Typically a 36-inch microtunnel boring machine is limited to a maximum material size of 9 to 12 inches, depending on the machine.

The weakly cemented and medium dense soils encountered at the site (at the anticipated pipe depths) are anticipated to exhibit firm to moderately fast raveling behavior in accordance with the Tunnelman's Ground Classification. Firm to slow raveling is anticipated in the very dense formational cobble silt matrix. And very slow raveling is anticipated in the weathered rock layer. It is likely that over-sized microtunneling machines on the order of 6 feet in diameter would be needed due to the power required to advance the machine in the harder formational layer. Bedrock and conglomerate layers are associated with San Diego Formation. High blow counts and refusal were noted in exploratory borings. Due to the size of the sampling equipment and the drilling methods, it was not possible to determine the maximum size of the materials (gravel, cobbles or debris) encountered. Additional subsurface exploration may be performed at this location to characterize the materials maximum size within the pipeline alignment. Tunneling equipment should be designed for the anticipated site conditions.

8.7.2. Horizontal Directional Drilling

Horizontal directional drilling (HDD) methods involve steerable tunneling systems for installation of small- and large-diameter pipelines. In most cases, it is a two stage process. The first stage consists of drilling a small diameter pilot hole along the desired centerline of the proposed line. The second stage consists of enlarging the pilot hole to the desired diameter and pulling the utility line through the enlarged hole. This method allows to track the location of the drill bit and steer it during the drilling process. The result is greater degree of precision in placing utilities. Since HDD does not require shafts to advance the bore, it requires a long laydown area as the pipe to be pulled into the bore hole must be laid out its full length prior to installation. Since pressurized drilling fluids are present within the bore hole, care must be taken to avoid inadvertent fluid releases to the surface during drilling. The entry and exit angles for HDD bore should be between about 8 and 12 degrees from the horizontal. The minimum bending radius for the pipe (in feet) should be about 100 times the diameter of the pipe (in inches). Based on our subsurface exploration, the site is underlain by dense to very dense sandy gravel/cobble matrix with some silt, therefore HDD installation using HDPE pipe may be considered as an appropriate material. Necessary action should be taken to avoid the violation of entry and exit angle limit if this method would be selected.

8.7.3. Jack and Bore or Auger Boring

The jack and bore (also known as auger boring) method uses a rotating cutting head to create a borehole from a drive shaft to a reception shaft. The most common type of jack and bore used for pipe installation is the track system. Spoils are transported back to the drive shaft by the auger rotating inside a casing that is being jacked in place during augering. Hydraulic jacks at the boring machine are used to advance the casing. A properly constructed drive shaft is important for the success of a track type auger boring project. The shaft requires a stable foundation and an adequate thrust block. The thrust block transmits the horizontal jacking forces from the tracks to the ground at the rear drive shaft. It must be designed to distribute the jacking force over sufficient area so that the allowable compressive strength of the soil is not exceeded. The typical pipe material is steel because the pipe must resist abrasion caused by the rotating augers, although concrete pipe may also be used designed for jack and bore method. Pipes with a diameter of 8 to 60 inch and drive lengths of 40 up to 500 feet can be used. This method is unguided and thus provides very limited tracking. This techniques has limited steering ability, which can affect the line and grade accuracy. Jack and bore should not be used below the groundwater table, in running sands, or in soils with large boulders. Another drawback associated with this method is surface subsidence and heaving during construction. Subsidence occurs when over-excavation is permitted, and heaving occurs when excessive force is applied to the excavation force. Considering all these disadvantages. Twining does not recommend Jack and Bore as a method for trenchless installation.

8.7.4. Trenchless Installation Recommendations

We recommend that trenchless pipe installation for this project be performed by contractors with experience in similar projects using installation methods and equipment compatible with local soil conditions. The risk of impacting adjacent structures, utilities, ground heave, vibrations, settlement and refusal of the excavation tools should be considered. Surface settlements are anticipated to be greater where pipe installations occur at shallower depths. Monitoring of surface settlement should be provided during installation. Even though significant settlement is not anticipated, mitigation measures may be required if surface settlement exceeds ½-inch. The estimated load on 12-inch pipelines installed at depths ranging from 3 to 5 feet is 120 pounds per linear feet based on Marston's formula. Loads for different pipe sizes and depths would need to be evaluated.

8.8. Open Cut Installation

Twining understands that the City wants to install the proposed pipelines by means of trenchless installation system. Due to subsurface conditions present on the site, we have also provided open cut installation recommendations in case of deviation from the original proposal. Trenching and excavation should be performed in accordance with CalOSHA guidelines. Recommendations for temporary excavations were presented in sections 8.5 and 8.6 of this report.

8.8.1. Installation Recommendations

We recommend that pipe installation for this project be performed by contractors with experience in similar projects and local soil conditions. Due to existing improvements in the areas surrounding the proposed alignments and subsurface conditions, difficulties during installation may occur. The excavation and pipeline installation methods and equipment used should be compatible with the project requirements and anticipated subsurface conditions. The effects of excavation of formational materials on adjacent structures and utilities due to vibrations and settlement should be considered.

8.8.2. Difficult Rippability

Bedrock encountered along the pipeline alignment predominantly includes dense to very dense, to locally cemented gravel and cobble conglomerates, with a sandy matrix. The majority of bedrock (conglomerate) formations are anticipated to be rippable to marginally rippable but will likely contain isolated cemented zones that are very hard and difficult to excavate. Several cemented conglomerate zones were observed near the alignment.

8.8.3. Pipeline Loads

The loads imposed by backfill soils on the buried pipelines may be determined using the Marston-Spangler equation:

$$W_c = C_d w B_c B_d$$

where, W_c = load, in pounds per foot
C_d = Marston load coefficient, defined as:

$$C_d = \frac{1 - e^{-2K\mu' \frac{H}{Bd}}}{2K\mu'}$$

w = density of backfill materials, in pounds per cubic foot
B_d = width of the trench at top of pipe, in feet
B_c = outside width of flexible pipe, in feet

The Martson-Spangler load factors recommended for this project are presented in Table 2. The resulting loads are applicable for project design provided that pipe installation, trench dimensions, placement and compaction of trench backfill materials are performed in accordance with City of San Diego standard plans and specifications and Section 306 of the Standard Specifications for Public Works Construction (SSPWC - Greenbook).

Table 2
Marston-Spangler Load Factors

Unit Weight of Backfill	Coefficient of Friction (μ')	Rankine's Ratio (K)	Maximum Kμ'
132 pcf	0.35	0.33	0.165

8.8.4. Pipe Bedding

Pipe bedding as specified in the "Greenbook" Standard Specifications for Public Works Construction can be used. Bedding material should consist of clean sand having a sand equivalent not less than 30 and should extend to at least 12 inches above the top of pipe. Alternative materials meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The onsite materials are not expected meet "Greenbook" bedding specification. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No void or uncompacted areas should be left beneath the pipe haunches. Ponding or jetting the pipe bedding should not be allowed.

8.8.5. Monitoring

Buildings, structures, sidewalks, pavements and other improvements that are adjacent to the proposed water alignment should be surveyed and photographed prior to excavation. Pre- and post-construction video-documentation should be conducted in adjacent storm and sanitary sewer systems. The initial relative positions and elevations of adjacent improvements should be recorded.

An appropriate number of survey points should be provided by a licensed surveyor so that the Project Engineer may formulate a professional opinion regarding movement. Survey points should be monitored once each week until the installation and backfilling is completed. Additional surveying may be required by the Project Engineer. Visual observations of the excavation and adjacent areas should be made on a daily basis by Twining during installation of the pipeline.

8.8.6. Trench Bottoms

At locations where the trench bottom is yielding or otherwise unstable, pipe support may be improved by placing 12 inches of ¾-inch crushed rock as defined in SSPWC Section 200-1.2. Remedial earthwork at the trench bottom should be performed where oversize materials (rocks or clods greater than 3 inches) are present. Removal of oversize materials to a depth of 6 inches below the bottom of the pipeline and replacement with fill compacted to at least 90% relative compaction is recommended. Alternatively, ¾-inch crushed rock may be used.

8.8.7. Trench Backfill

Pipe trench backfill should conform to the recommendations presented in this report, City of San Diego standard plans and specifications, and SSPWC Section 306.

8.9. Lateral Pressures for Thrust Blocks

Thrust restraint for buried pipelines may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks should be backfilled with granular backfill material, compacted as outlined in this report. Thrust blocks may be designed using lateral passive earth pressure according to the equation presented below:

$$P_p = 150\ (D^2 - d^2)\ \text{lb/ft}$$

where, P_p is the passive soil resistance per foot of width
d is the depth to the top of the thrust block
D is the depth to the bottom of the thrust block.

8.10. Pavement Reconstruction

Trench excavations in existing streets or paved areas will involve replacement of pavement sections at the completion of work. In general, pavement repair should conform to the material thicknesses and compaction requirements of the adjacent pavement section (i.e., match existing unless otherwise directed by the City). Subgrade and aggregate base materials should be compacted to 95 percent relative compaction as evaluated using ASTM D1557. Asphalt concrete (AC) should be compacted to 95 percent relative compaction as evaluated using ASTM D1561 (Hveem density). Pavement reconstruction should conform to City of San Diego requirements.

8.11. Corrosivity

Laboratory testing was performed on representative soils samples to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The pH values of the tested samples ranged from 7.2 to 8.1. Electrical resistivity values ranged from 2,600 to 7,500 ohm-centimeters. Chloride content ranged from 131 to 155 parts per million (ppm). Sulfate content ranged from 271 to 304 ppm. Additional details and laboratory test results are presented in Appendix B.

Based on Caltrans (2015) corrosion criteria, a site is considered corrosive if one or more of the following conditions exist at the site: chloride concentrations of 500 ppm or greater, sulfate concentration of 2,000 ppm or greater, or pH of 5.5 or less. Based on the laboratory test results and Caltrans Corrosion Guidelines, the site is considered non-corrosive. The risk of corrosion to pipes is considered low. We recommend that a corrosion engineer be consulted for corrosion protection recommendations for the project.

8.12. Buried Metal

A factor for evaluating corrosivity to buried metal is electrical resistivity. The electrical resistivity of a soil is a measure of resistance to electrical current. Corrosion of buried metal is directly proportional to the flow of electrical current from the metal into the soil. As resistivity of the soil decreases, the corrosivity generally increases. The samples tested resulted in electrical resistivity values ranging from 2,600 to 7,500 ohm-centimeters.

Correlations between resistivity and corrosion potential (NACE, 1984) indicate that the soils have a low corrosive potential to buried metals. As such, corrosion protection for metal in contact with site soils is not required. If desired by the design team, corrosion protection may include the use of epoxy or asphalt coatings.

8.13. Concrete Placement

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical deterioration. Laboratory testing indicated maximum sulfate content of 304 ppm in the samples tested. According to American Concrete Institute (ACI) 318, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil less than 0.10 percent by weight (i.e., less than 150 ppm). Therefore, the site earth materials may be considered to have moderate potential for sulfate attack. Due to the potential for variability of soils, we recommend using Type II/V cement for concrete structures in contact with soil, and a water-cement ratio of no more than 0.45.

8.14. Site Drainage

Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage and damage to existing improvement. Under no circumstances should water be allowed to pond adjacent to surrounding improvements. In addition, surface drainage should be directed away from the top of the slopes or as determined by the civil engineer. Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

9. DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the earthwork procedures will be important to the performance of the proposed development. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

9.1. Plans and Specifications

Project plans and specifications should be reviewed by Twining, Inc. prior to bidding and construction, as the geotechnical recommendations may need to be reevaluated in the light of the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications. Based on the work already performed, this office is best qualified to provide such review.

9.2. Construction Monitoring

Site preparation, removal of unsuitable soils, assessment of imported fill materials, fill placement, and other site grading operations should be observed and tested, as appropriate. The substrata exposed during construction may differ from that encountered in the exploratory excavations. Continuous observation by a representative of Twining, Inc. during construction allows for evaluation of the soil conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where necessary.

Water condition, where suspected or encountered during construction, should be evaluated and remedied by the project civil and geotechnical consultants. The project developer and property owner, however must realize that post-construction appearances of groundwater may have to be dealt with on a site specific basis. Proper functional drainage should be implemented and maintained at the property.

10. LIMITATIONS

The recommendations and opinions expressed in this report are based on Twining, Inc.'s review of readily available background documents, on information obtained from field explorations, and on laboratory testing. In the event that any of our recommendations conflict with recommendations provided by other design professionals, we should be contacted to aid in resolving the discrepancy.

Due to the limited nature of our field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations (for example, the extent of removal of unsuitable soil) and that additional effort may be required to mitigate them.

Site conditions, including but not limited to groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Twining, Inc. has no control.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Twining, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report has been prepared for the exclusive use by the City of San Diego and its agents for specific application to the proposed project. Land use, site conditions, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of this report and the nature of the project, Twining, Inc. may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Twining, Inc. from all liability resulting from the use of this report by any unauthorized party.

Twining, Inc. has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area under similar circumstances. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this report.

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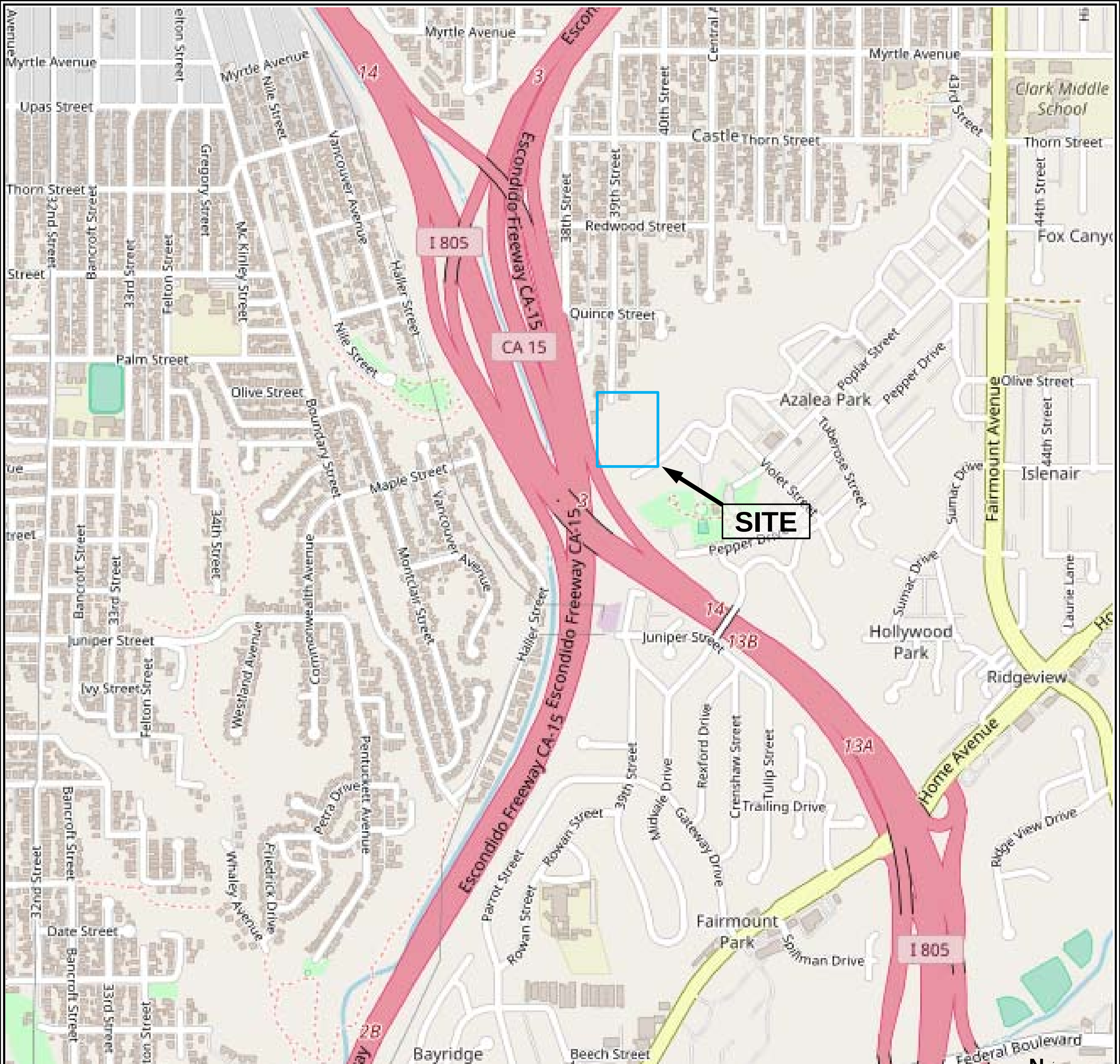
FIGURES



LEGEND

SITE LOCATION

N

SCALE 1" = 900'

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

SOURCE: OPEN STREET MAP, 2016.

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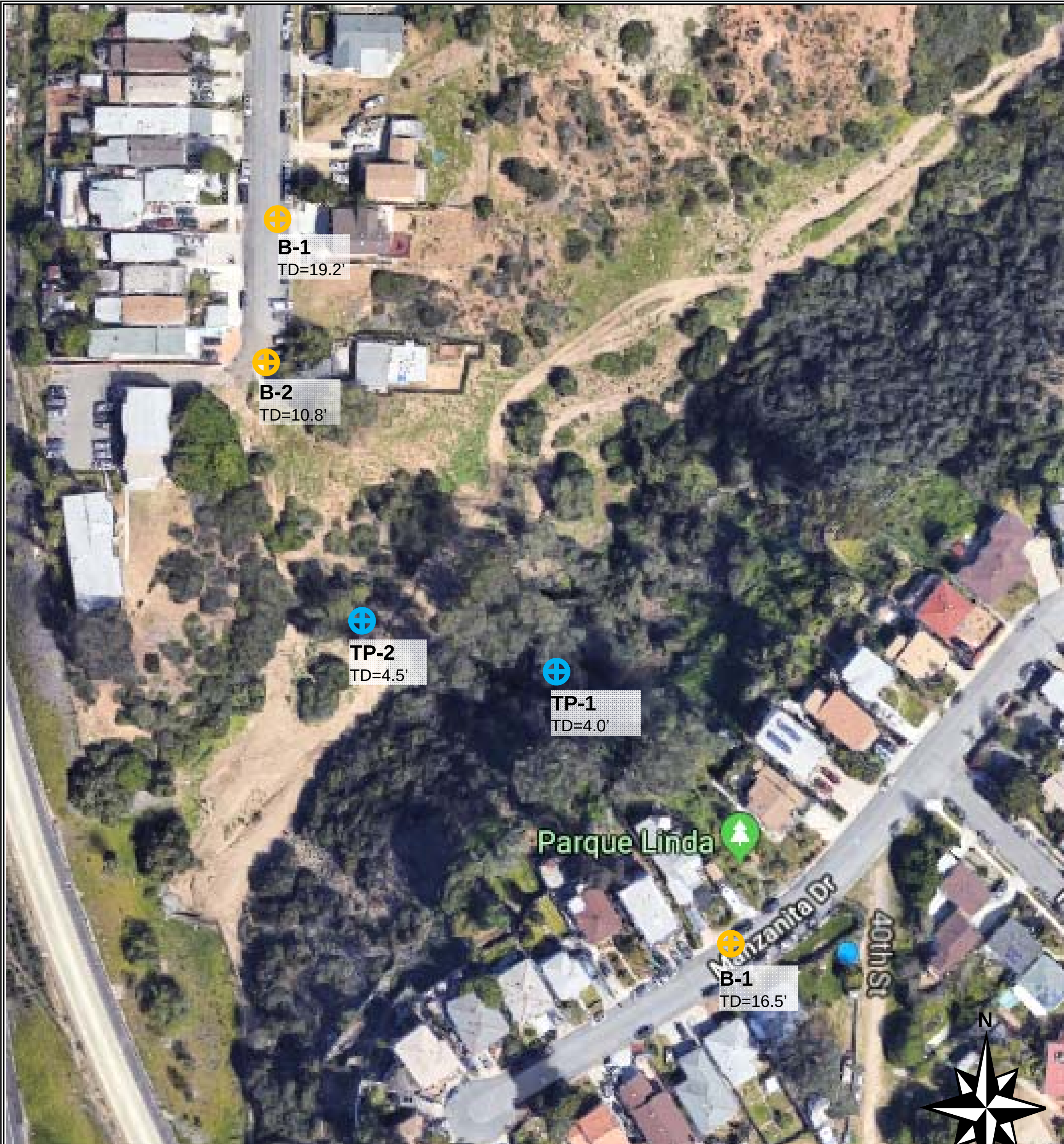
PROJECT LOCATION MAP

15GT15 – MANZANITA CANYON WATER AND STORM DRAIN GROUP 968
MANZANITA DRIVE & 39th STREET,
SAN DIEGO, CALIFORNIA

REPORT DATE:	PROJECT NO.:	FIGURE 1
SEP 2018	180325.2	



LEGEND

B-1 APPROXIMATE BORING LOCATION
TD=5.5' TD= TERMINATION DEPTH IN FEET

TP-1 APPROXIMATE BORING LOCATION
TD=4.0' TD= TERMINATION DEPTH IN FEET

SCALE 1" = 100'

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

REFERENCE: GOOGLE MAPS (2017)

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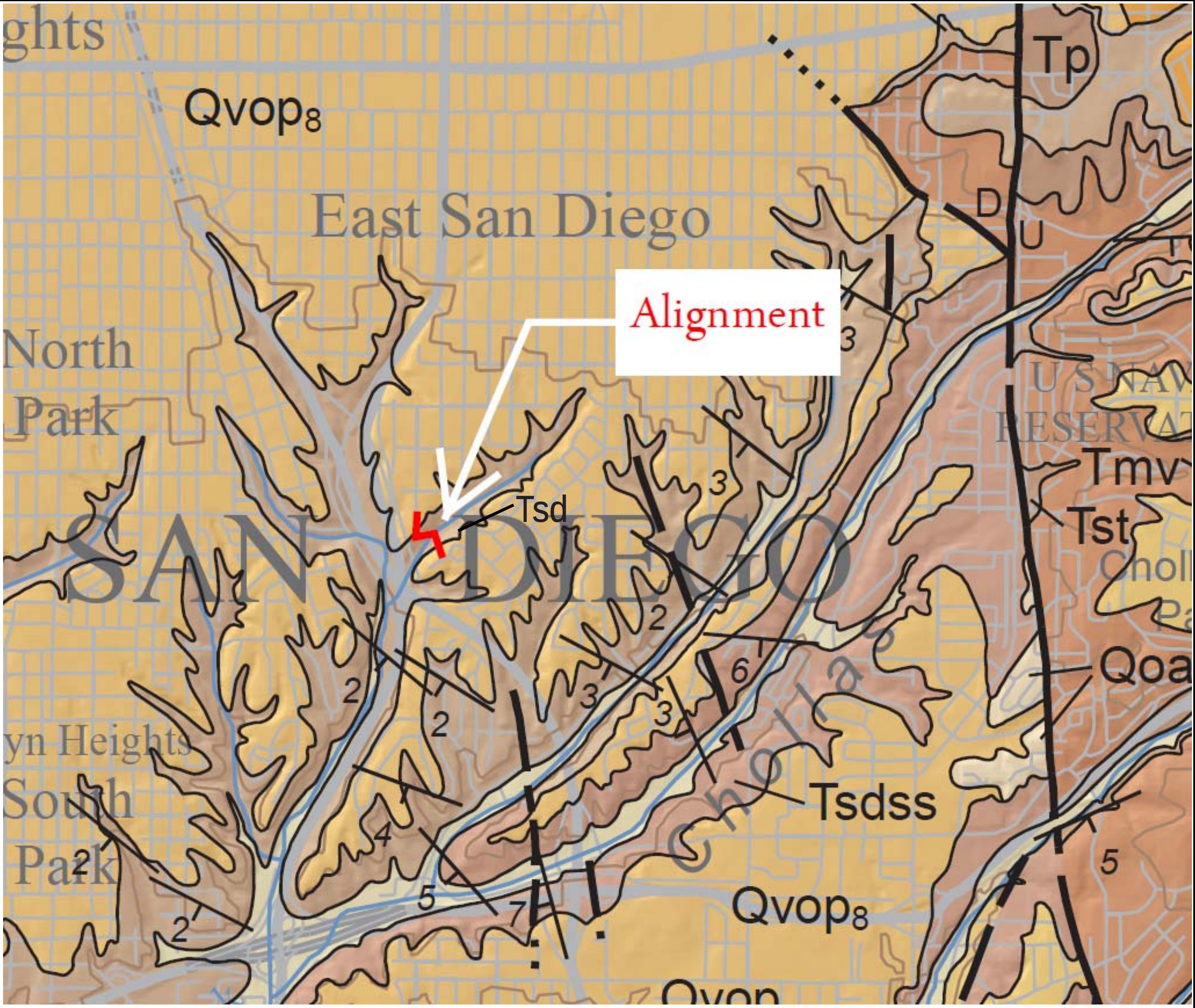
EXPLORATION LOCATION MAP

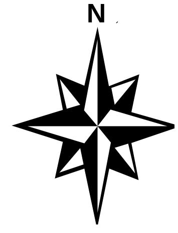
15GT15- MANZANITA CANYON WATER AND STORM DRAIN
GROUP 968
MANZANITA DRIVE & 39th STREET,
SAN DIEGO, CALIFORNIA

REPORT DATE:	PROJECT NO.:	FIGURE 2
FEB 2018	180325.2	



LEGEND

Qoa	OLD ALLUVIAL FLOOD PLAIN DEPOSITS
Tsd	SAN DIEGO FORMATION
Tsdss	SAN DIEGO FORMATION SAND STONE)
$Qvop_8$	VERY OLD PARALIC DEPOSITS (Unit 8)
Tst	STADIUM CONGLOMERATE
Tmv	MISSION VALLEY FORMATIONS



SCALE 1" = 1,500’

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

SOURCE: GEOLOGIC MAP OF EL CAJON, CALIFORNIA GEOLOGIC SURVEY, 2008.

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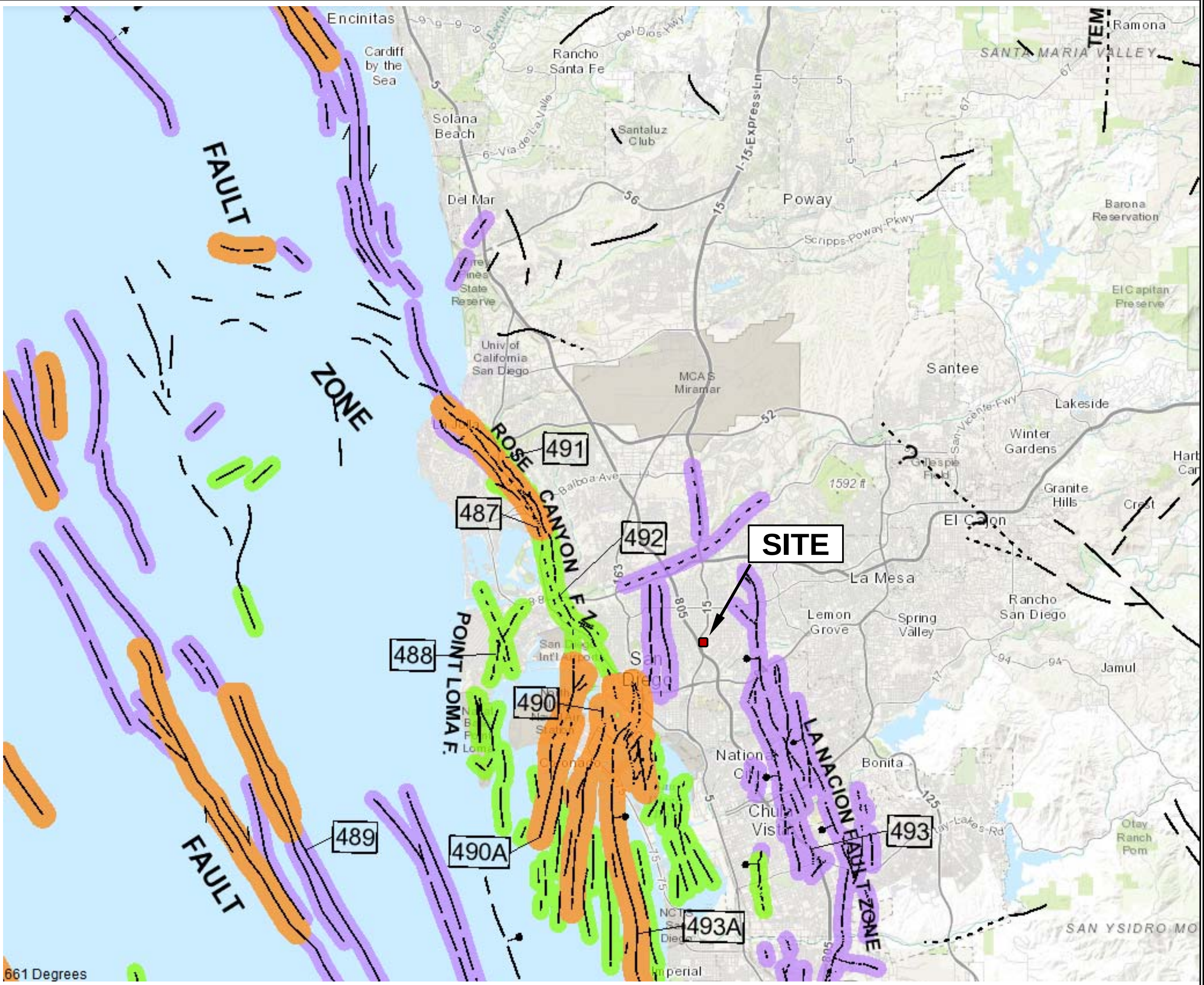
REGIONAL GEOLOGIC MAP

15GT15 – MANZANITA CANYON WATER & STORM DRAIN GROUP 968
MANZANITA DRIVE & 39th STREET,
SAN DIEGO, CALIFORNIA

REPORT DATE:	PROJECT NO.:	FIGURE 3
FEB 2018	180004.2	



LEGEND

FAULT ALONG WHICH HISTORIC DISPLACEMENT HAS OCCURRED

HOLOCENE FAULT DISPLACEMENT

LATE QUATERNARY FAULT DISPLACEMENT

QUATERNARY FAULT DISPLACEMENT

PRE-QUATERNARY FAULT DISPLACEMENT

N

SCALE 1" = 4 MILES

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

REFERENCE: CGS, 2010, FAULT ACTIVITY MAP OF CALIFORNIA (2010)

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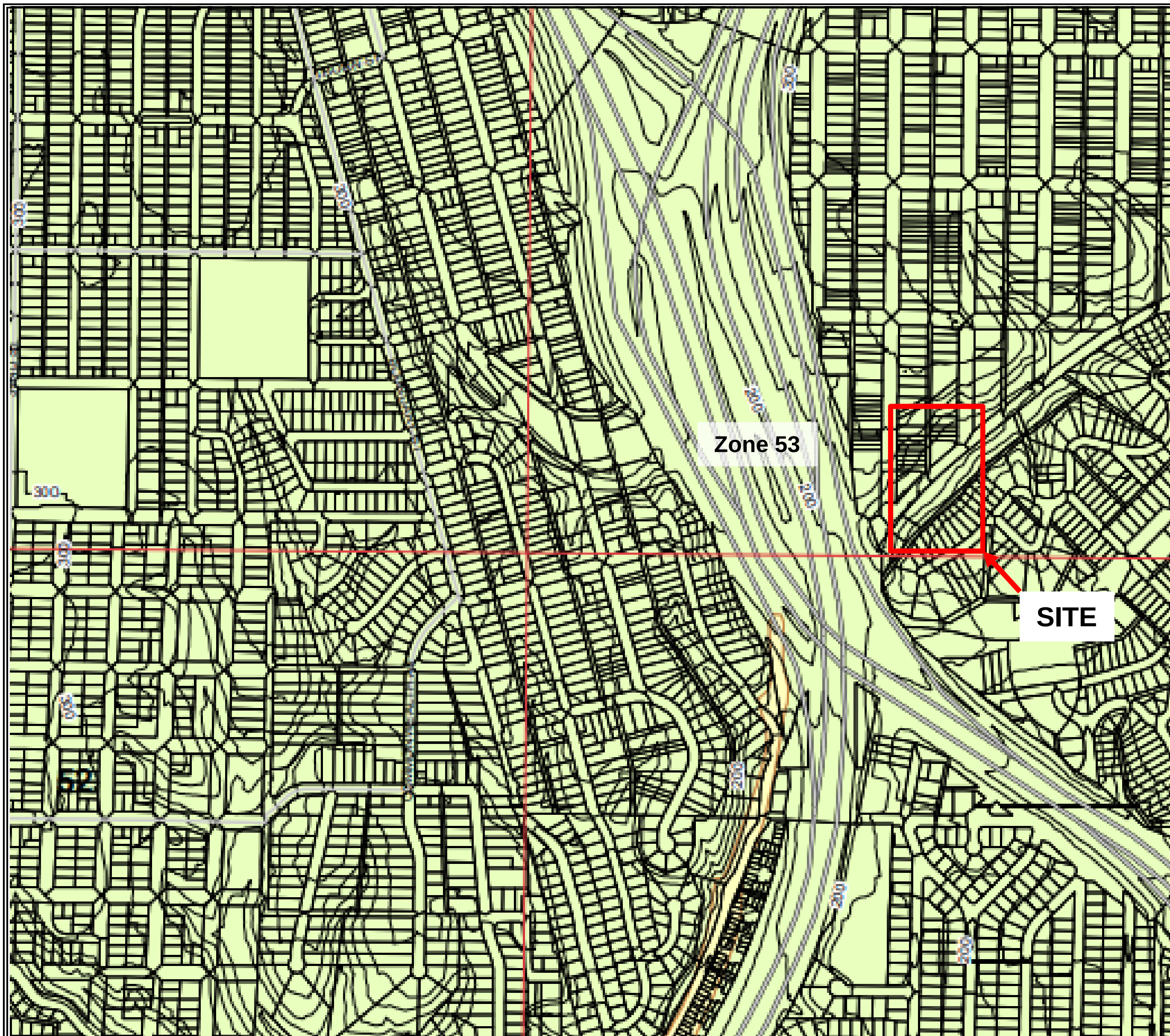
FAULT LOCATION MAP

15GT15 – MANZANITA CANYON WATER AND STORM DRAIN GROUP 968
MANZANITA DRIVE & 39th STREET,
SAN DIEGO, CALIFORNIA

REPORT DATE:	PROJECT NO.:	FIGURE 4
SEP 2018	180325.2	



LEGEND

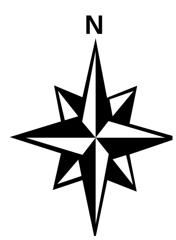
51 Level mesas -- underlain by terrace deposits and bedrock nomimal risk

52 Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk

53 Level or sloping terrain, unfavorable geologic structure, Low to moderate risk

54 Steeply sloping terrain, unfavorable or fault controlled geologic structure, Moderate risk

55 Modified terrain (graded sites) Nominal risk



SCALE 1" = 400'

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

REFERENCE: CITY OF SAN DIEGO SEISMIC SAFETY STUDY MAP(2008)

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SEISMIC SAFETY MAP

15GT15 – MANZANITA CANYON WATER AND STORM DRAIN GROUP 968
MANZANITA DRIVE & 39th STREET,
SAN DIEGO, CALIFORNIA

REPORT DATE:	PROJECT NO.:	FIGURE 5
FEB 2018	180325.2	

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APPENDIX A
FIELD EXPLORATION

Appendix A
Field Exploration

General

The subsurface exploration program for the proposed project included drilling and logging five, 8-inch diameter borings. The borings were advanced using a CME-75 truck-mounted hollow-stem-auger drill rig. The borings reached depths of approximately 10.8 feet to 19.5 feet below existing grades. Hand excavated test pits were excavated to depths of approximately 4.5 feet.

Drilling and Sampling

The Boring and Test Pit Logs are presented in Figures A-2 through A-6. An explanation of these logs is presented in Figure A-1. The exploration Logs describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The log also shows the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by a Twining, Inc. engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate and the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

A California modified sampler was used to obtain drive samples of the soils encountered. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft that is driven into the soil a total of 18 inches using a 140-pound, automatic-drop hammer falling approximately 30 inches. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs. The soil was retained in brass rings for laboratory testing. Additional soil from each drive remaining in the cutting shoe was usually discarded after visually classifying the soil.

Disturbed samples were obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft that is driven into the soil a total of 18 inches using a 140-pound, automatic-drop hammer falling approximately 30 inches. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs. Soil samples obtained by the SPT were retained in plastic bags.

Bulk samples of the soil cuttings were collected in plastic bags for testing in our laboratory.

UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

COARSE-GRAINED SOILS			FINE-GRAINED SOILS	
Relative Density	**SPT (blows/ft)**	**Relative Density (%)**	**Consistency**	**SPT (blows/ft)**
Very Loose	<4	0 - 15	Very Soft	<2
Loose	4 - 10	15 - 35	Soft	2 - 4
Medium Dense	10 - 30	35 - 65	Medium Stiff	4 - 8
Dense	30 - 50	65 - 85	Stiff	8 - 15
Very Dense	>50	85 - 100	Very Stiff	15 - 30
			Hard	>30

NOTE: SPT blow counts based on 140 lb. hammer falling 30 inches

Sample Symbol	Sample Type	Description
	SPT	1.4 in I.D., 2.0 in. O.D. driven sampler
	California Modified	2.4 in. I.D., 3.0 in. O.D. driven sampler
	Bulk	Retrieved from soil cuttings
	Thin-Walled Tube	Pitcher or Shelby Tube

LABORATORY TESTING ABBREVIATIONS

ATT	Atterberg Limits
C	Consolidation
CORR	Corrosivity Series
DS	Direct Shear
EI	Expansion Index
GS	Grain Size Distribution
K	Permeability
MAX	Moisture/Density (Modified Proctor)
O	Organic Content
RV	Resistance Value
SE	Sand Equivalent
SG	Specific Gravity
TX	Triaxial Compression
UC	Unconfined Compression

TWINING

EXPLANATION FOR LOG OF BORINGS

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	
180325.2	September 2018	FIGURE A-1

DATE DRILLED	8/28/2018	LOGGED BY	SM	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Exploration	SURFACE ELEVATION (ft.)	276 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									ASPHALT: 4" Asphalt concrete pavement
								SM	AGGREGATE BASE: 8" aggregate base
									ARTIFICIAL FILL (QAF-UNDOCUMENTED): Silty SAND(SM): Dense, moist, reddish brown, chunk of clay, gravel up to 1/2", fine to medium
271	5			23				SW-SM	VERY OLD PARALIC DEPOSITS (QVOP8): Well graded SAND with Silt and Gravel (SW-SM): Dense, moist, dark reddish brown, with gravel up to 3/4", medium
266	10			50/2"				SW-SM	-very dense, black mottling.
261	15			50/1"				SW-SM	-broken cobble after grinding sizes up to 3/4", difficulty in drilling
								SW-SM	Practical refusal at 16.5 feet
									Total Depth = 16.5 feet Backfilled on 8/28/2018 Groundwater not observed at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
256	20								
251	25								
246	30								

TWINING

LOG OF BORING

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	
180325.2	September 2018	FIGURE A - 2

DATE DRILLED	8/28/2018	LOGGED BY	SM	BORING NO.	B-2
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Exploration	SURFACE ELEVATION (ft.)	250 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									ASPHALT: 4" Asphalt Concrete Pavement
								SM	AGGREGATE BASE: 8" aggregate base
								SM	ARTIFICIAL FILL (QAF-UNDOCUMENTED): Silty SAND with Gravel (SM): Dense, moist, brown, chunk of clay with broken gravel up to 3/4", grinding observed from 1'6"
245	5			47					VERY OLD PARALIC DEPOSITS (QVOP8): Silty SAND (SM): Very Dense, moist, yellowish brown, with gravel up to 1" (possibly broken cobble), fine grained, difficulty in drilling
240	10			50/2" 50/5"	5.9	140.1		SM SM SM	-No recovery -Continues as silty sand, possibly broken cobble found as a gravel up to 1" -Practical refusal at 10.75 feet.
									Total Depth = 10.8 feet Backfilled on 8/28/2018 Groundwater not observed at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
235	15								
230	20								
225	25								
220	30								

TWINING

LOG OF BORING

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE A - 3
180325.2	September 2018	

DATE DRILLED	8/28/2018	LOGGED BY	SM	**BORING NO.**	**B-3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Exploration	SURFACE ELEVATION (ft.)	272 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0								ASPHALT: 4" Asphalt concrete pavement
									AGGREGATE BASE: 8" Aggregate base
								SM	VERY OLD PARALIC DEPOSITS (QVOP8): Silty SAND (SM): Very dense, moist, light brown, with broken gravel up to 1" observed on the shoe. fine, quartz present
267	5		X	89/8"	11.3	108.0			
262	10		X	73	12.7	107.8		SM	-grades to dark reddish brown, with broken gravel up to 1/2", medium to fine
257	15		X	51	9.9	96.2		SM	-dense, light brown, fine
			X	50/2"				SM	- very dense
252	20								Total Depth = 19.2 feet Backfilled on 8/28/2018 Groundwater not observed at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
247	25								
242	30								

TWINING

LOG OF BORING

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE A - 4
180325.2	September 2018	

DATE DRILLED	8/28/2018	LOGGED BY	SM	**BORING NO.**	**TP-1**
DRIVE WEIGHT	NA	DROP	NA	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	Hand Digging	DRILLER	Native Drilling	SURFACE ELEVATION (ft.)	185 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SM	COLLUVIUM (QCOL): Silty SAND with Gravel (SM): Loose to medium dense, damp, dark brown, with abundant gravel & cobble up to 3.5"
								SM	YOUNG ALLUVIUM (QAL): Silty SAND with Gravel (SM): Loose, damp, light brown, with abundant cobble up to 5"
								GP	Poorly Graded GRAVEL with Sand (GP): Medium dense, moist, dark reddish brown, Gravel and coble size up to 3"
								SM	SAN DIEGO FORMATION (TSD): Silty SAND with Gravel (SM): Dense, moist, dark orange brown, fine, weakly cemented
180	5								Total Depth = 4.5 feet Backfilled on 8/28/2018 Groundwater not observed at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
175	10								
170	15								
165	20								
160	25								
155	30								

TWINING

LOG OF BORING

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE A - 5
180325.2	September 2018	

DATE DRILLED	8/28/2018	LOGGED BY	SM	**BORING NO.**	**TP-2**
DRIVE WEIGHT	NA	DROP	NA	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	Hand Digging	DRILLER	Native Drilling	SURFACE ELEVATION (ft.)	181 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SM	COLLUVIUM (QCOL): Silty SAND (SM): Loose, moist, dark brown, with abundant gravel and cobble
		X						SM SM	Poorly Graded GRAVEL with Sand and Silt (SP-SM): Medium dense, moist, dark brown, gravel and cobble up to 4"
		X							-less gravel and cobble
								SM	- refusal on cobble at 4'6"
176	5								Total Depth = 4.5 feet Backfilled on 8/28/2018 Groundwater not observed at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
171	10								
166	15								
161	20								
156	25								
151	30								

TWINING

LOG OF BORING

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE A - 6
180325.2	September 2018	

TWINING
GEOTECHNICAL

15950 Bernardo Center Drive, Suite J
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APPENDIX B
LABORATORY TESTING

Appendix B
Laboratory Testing

Laboratory Moisture Content and Density Tests

The moisture content and dry density of selected driven samples obtained from the exploratory borings was evaluated in general accordance with the latest version of ASTM D2937. The test results are presented on the logs of the exploratory borings in Appendix A and also summarized in Table B-1.

Table B-1
Laboratory Moisture Content and Dry Density

Boring No.	Depth (feet)	Moisture Content (%)	Dry Unit Weight (pcf)
B-1	10	9.4	101.9
B-3	5	11.3	108.0
B-3	10	12.7	107.8
B-3	15	9.9	96.2

Atterberg Limits

Atterberg limits tests were performed on selected soil samples to evaluate plasticity characteristics and to aid in the classification of the soil. The tests were performed in general accordance with ASTM D4318. The results are presented in Figure B-1.

Maximum Dry Density and Optimum Moisture Content

A Standard Proctor test was performed on two samples of near-surface soils to determine the maximum dry density and optimum water content for compaction. The tests were performed in accordance with ASTM D 1557. The results have been presented in Figure B-12.

Sieve Analyses

The grain-size distribution of selected soil samples was evaluated in general accordance with ASTM C136/C117. Test results are presented on Figures B-2 through B-10.

Corrosivity

Soil pH and resistivity tests were performed on a representative soil samples in accordance with California Test Method 643. Chloride content of the selected samples was evaluated in accordance with California Test Method 422. Sulfate content of the selected samples was evaluated in accordance with California Test Method 417. The tests were performed by AP Engineering and Testing. Test results are presented on Table B-2.

Table B-2
Corrosivity Test Results

Boring No.	Depth (feet)	pH	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-1	0-5.0’	8.1	271	155	2,600
TP-1	0-2.0’	7.2	304	131	7,500

Direct Shear Test

Direct shear tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D3080 to evaluate the shear strength characteristics of the material. The samples were inundated during shearing to represent adverse field conditions. Test results are presented on Figure B-12.

Expansion Index Test

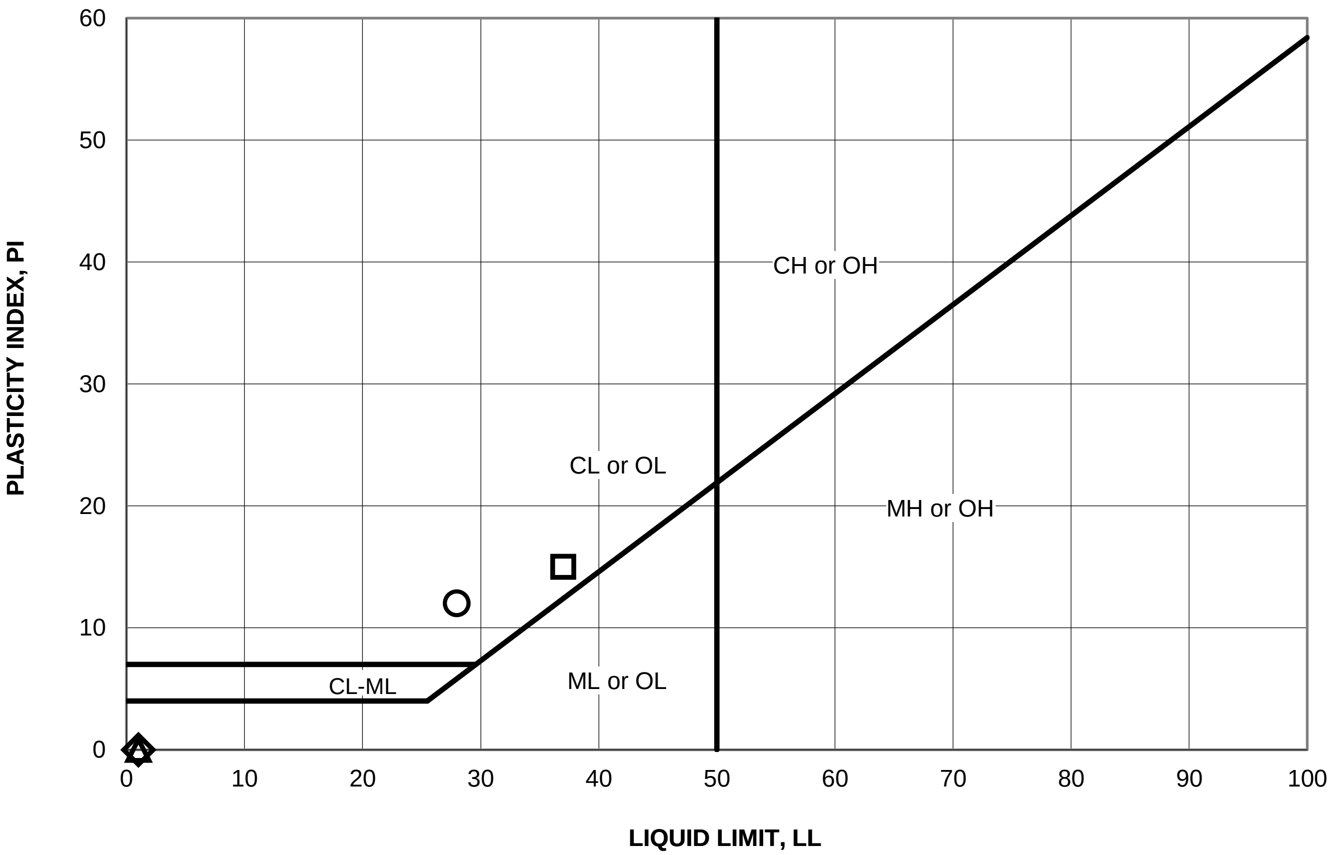
The expansion index of selected soil samples was evaluated in general accordance with ASTM D4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch-thick by 4-inch-diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of the Expansion Index test are presented on Table B-3.

Table B-3
Expansion Index Test Result

Boring No.	Depth (feet)	Expansion Index	Expansion Potential
B-1	0.0'-5.0'	77	Medium
TP-1	0.0'-2.0'	0	Very low (Not Expansive)

SYMBOL	SAMPLE LOCATION	SAMPLE DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS (% Finer than No. 40)	USCS (Entire Sample)
△	B-1	0-5'	NP	NP	NP	SM	SM
□	B-2	5'	37	22	15	CL	SM
○	TP-1	3.5-4.5	28	16	12	CL	SM
◇	TP-2	1'-3'	NP	NP	NP	SP-SM	SP-SM

NP - INDICATES NON-PLASTIC



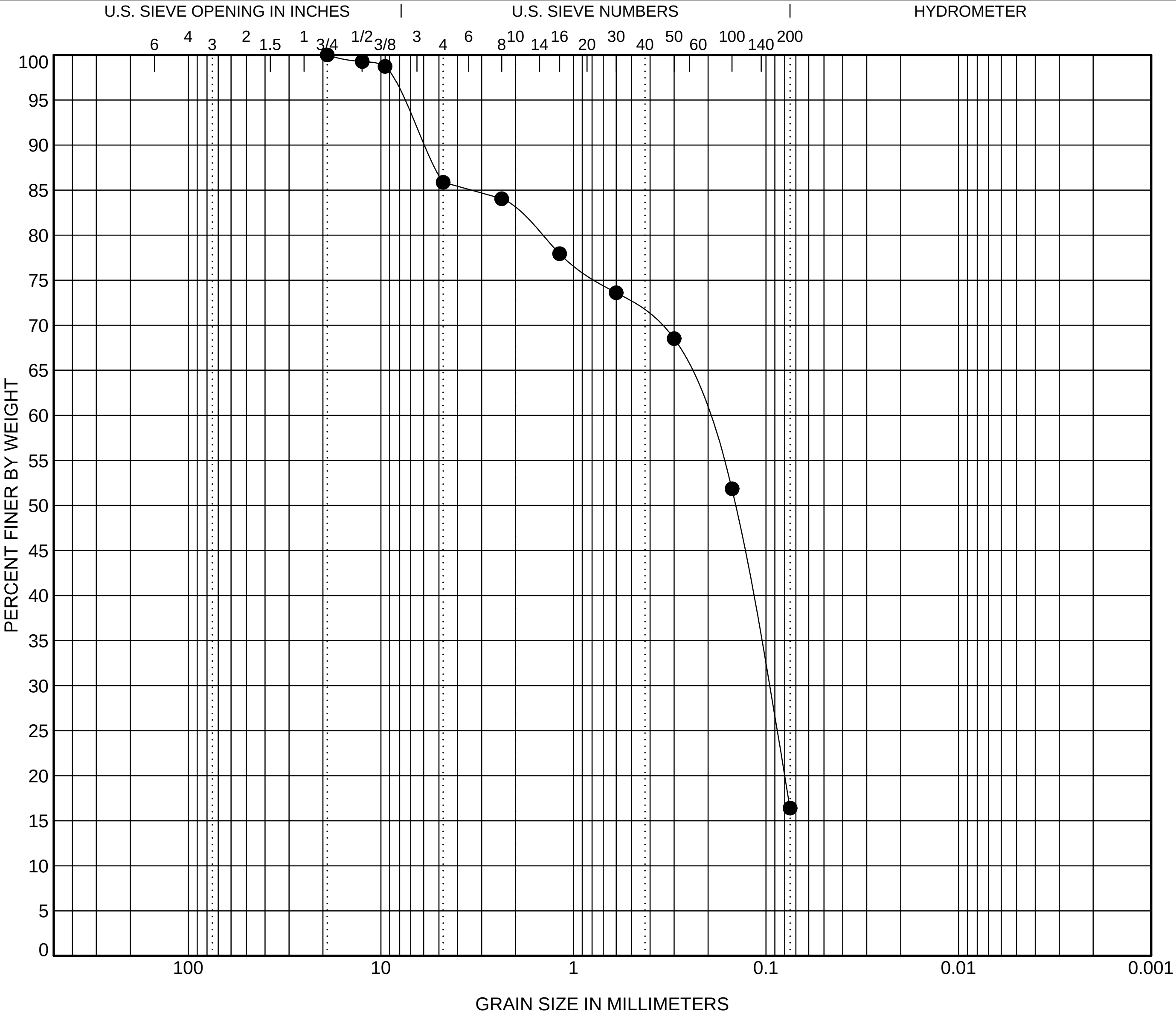
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

TWINING

ATTERBERG LIMITS

15GT15 - Manzanita Canyon Water and Storm Drain Group 968
Manzanita Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE B-1
180325.2	September 2018	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-1 at 0.0-5.0'**	**Silty SAND (SM)**		

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
19	**0.21**	**0.145**	**0.098**		**14.1**	**69.5**	**16.4**	

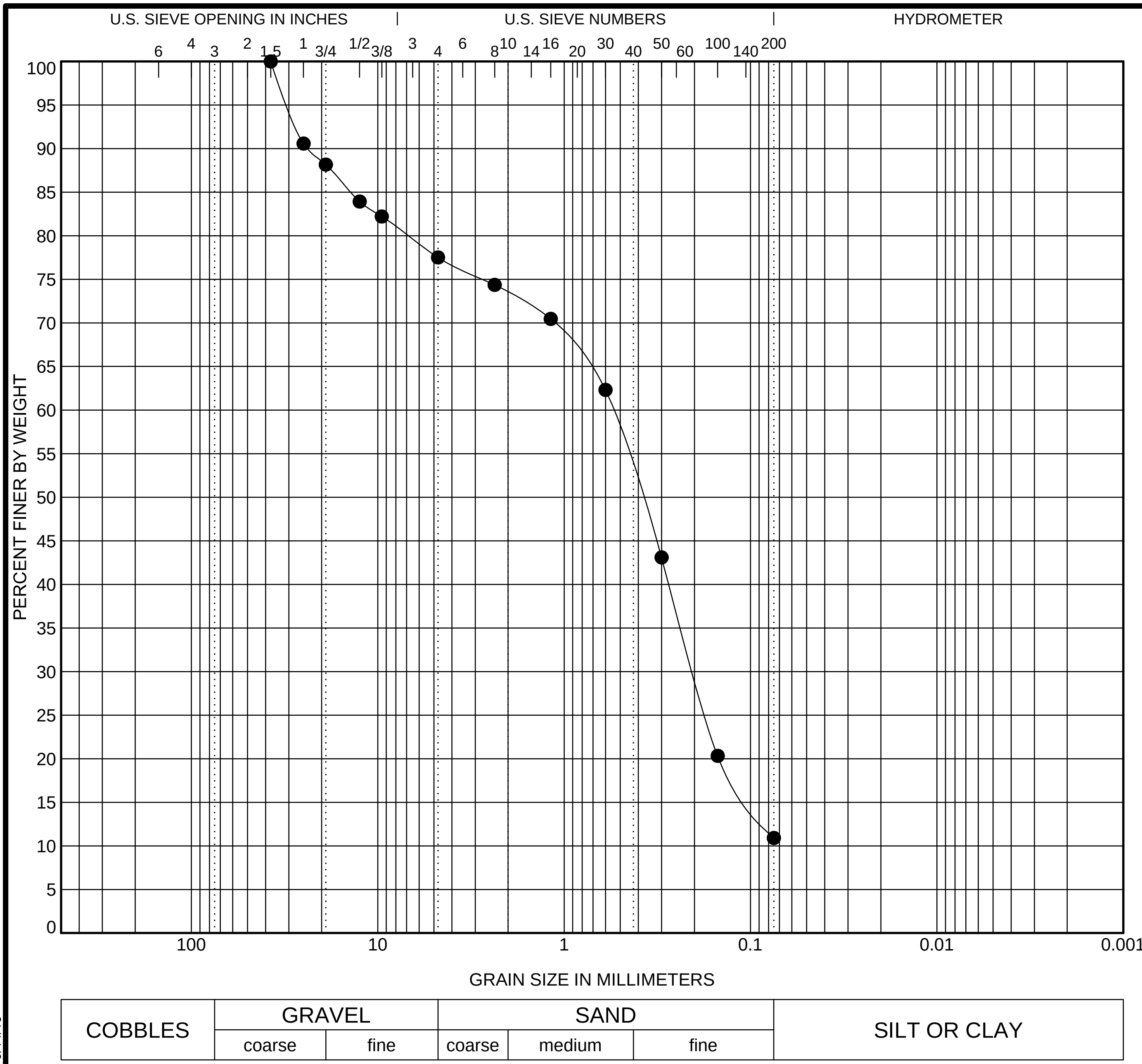
GRAIN SIZE 15GT15 MANZANITA DRIVE WATER GROUP 968.GPJ TWINING LABS.GDT 9/14/18

TWINING

GRAIN SIZE DISTRIBUTION

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE B- 2
180325.2	September 2018	



Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-1 at 5 ft**	**Well Graded SAND with Silt and Gravel (SW-SM)**	**1.05**	**7.87**

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
37.5	**0.552**	**0.385**	**0.201**		**22.5**	**66.6**	**10.9**	

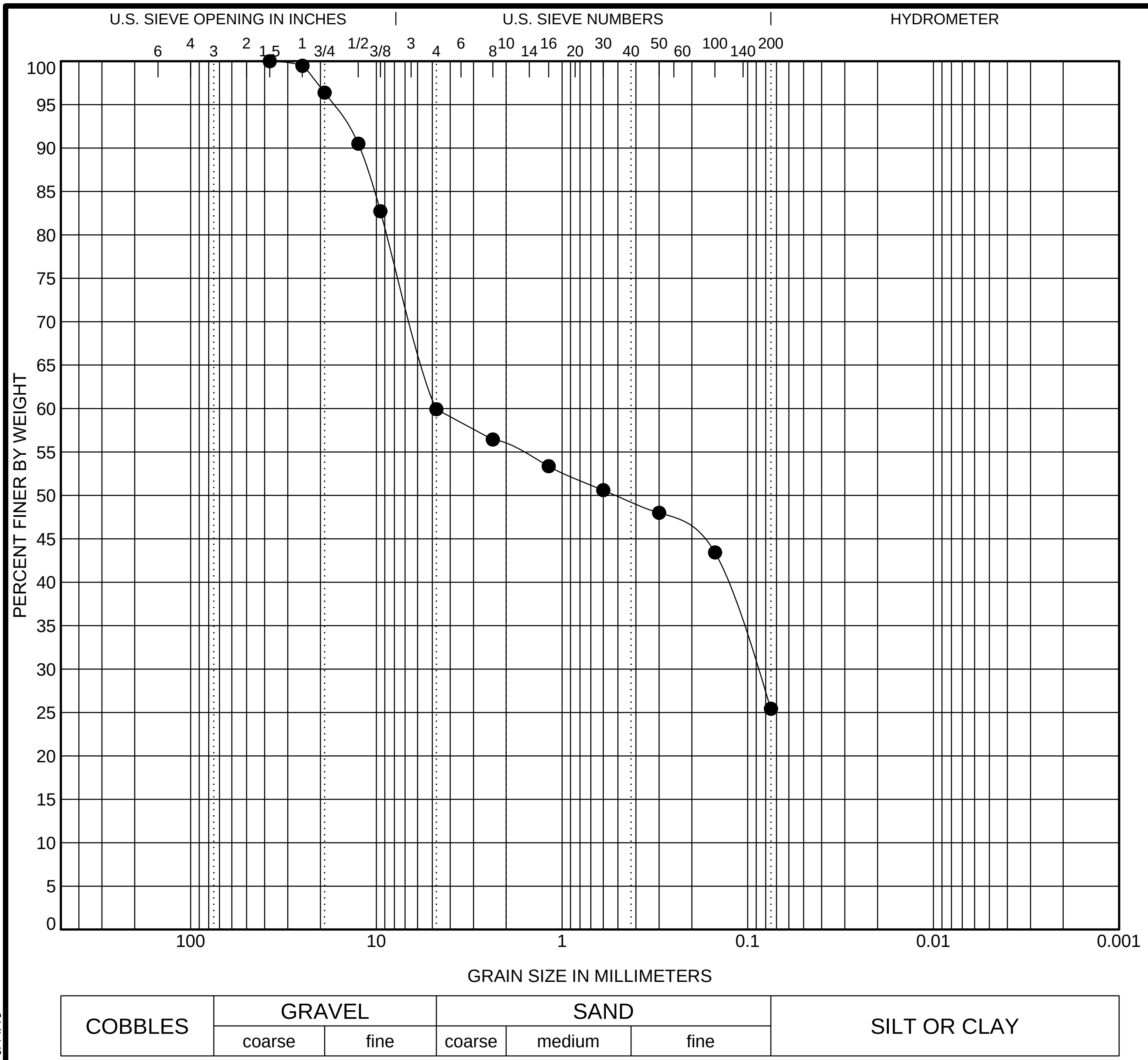
TWINING

GRAIN SIZE DISTRIBUTION

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	
180325.2	September 2018	FIGURE B- 3

GRAIN SIZE 15GT15 MANZANITA DRIVE WATER GROUP 968.GPJ TWINING LABS.GDT 9/14/18



Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-2 at 0.0-5.0'**	**Silty SAND with Gravel (SM)**		

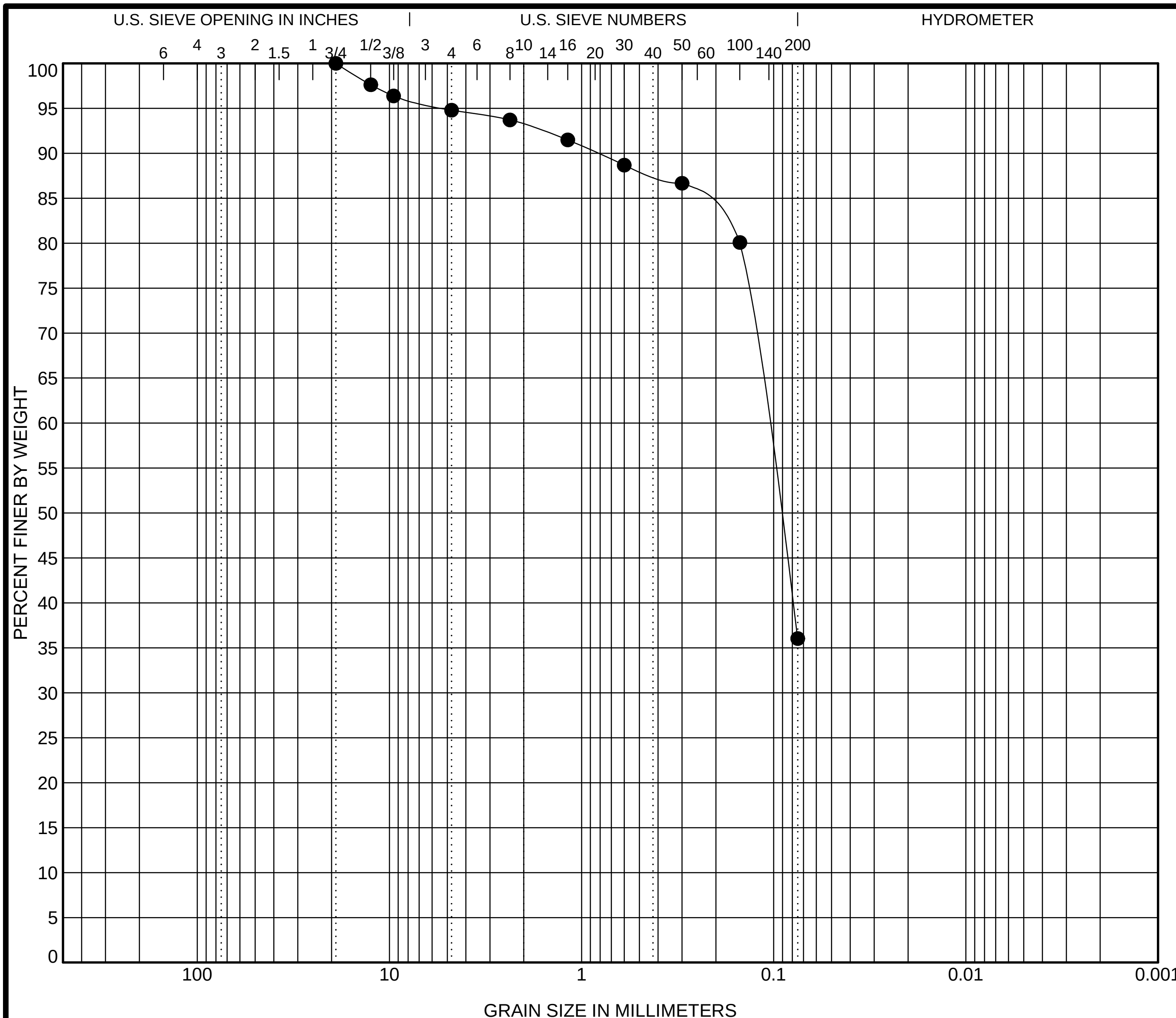
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
37.5	**4.761**	**0.511**	**0.089**		**40.1**	**34.5**	**25.4**	

TWINING

GRAIN SIZE DISTRIBUTION

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE B- 4
180325.2	September 2018	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-2 at 5 ft**	**Silty SAND (SM)**		

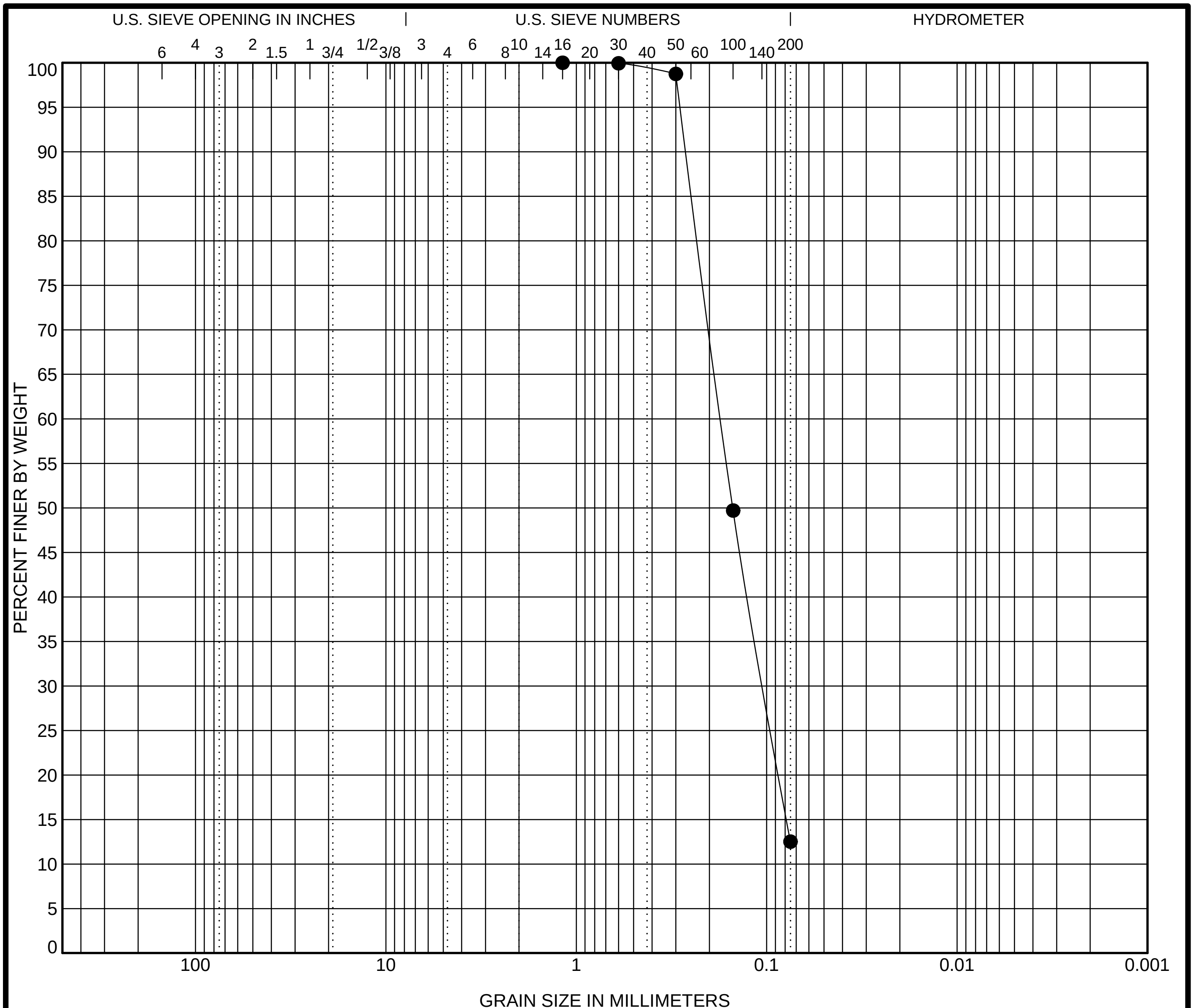
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
19	**0.109**	**0.093**			**5.2**	**58.8**	**36.0**	

TWINING

GRAIN SIZE DISTRIBUTION

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE B- 5
180325.2	September 2018	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-3 at 15 ft**	**Silty SAND (SM)**		

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
1.18	**0.173**	**0.151**	**0.104**		**0.0**	**87.5**	**12.5**	

GRAIN SIZE 15GT15 MANZANITA DRIVE WATER GROUP 968.GPJ TWINING LABS.GDT 9/14/18

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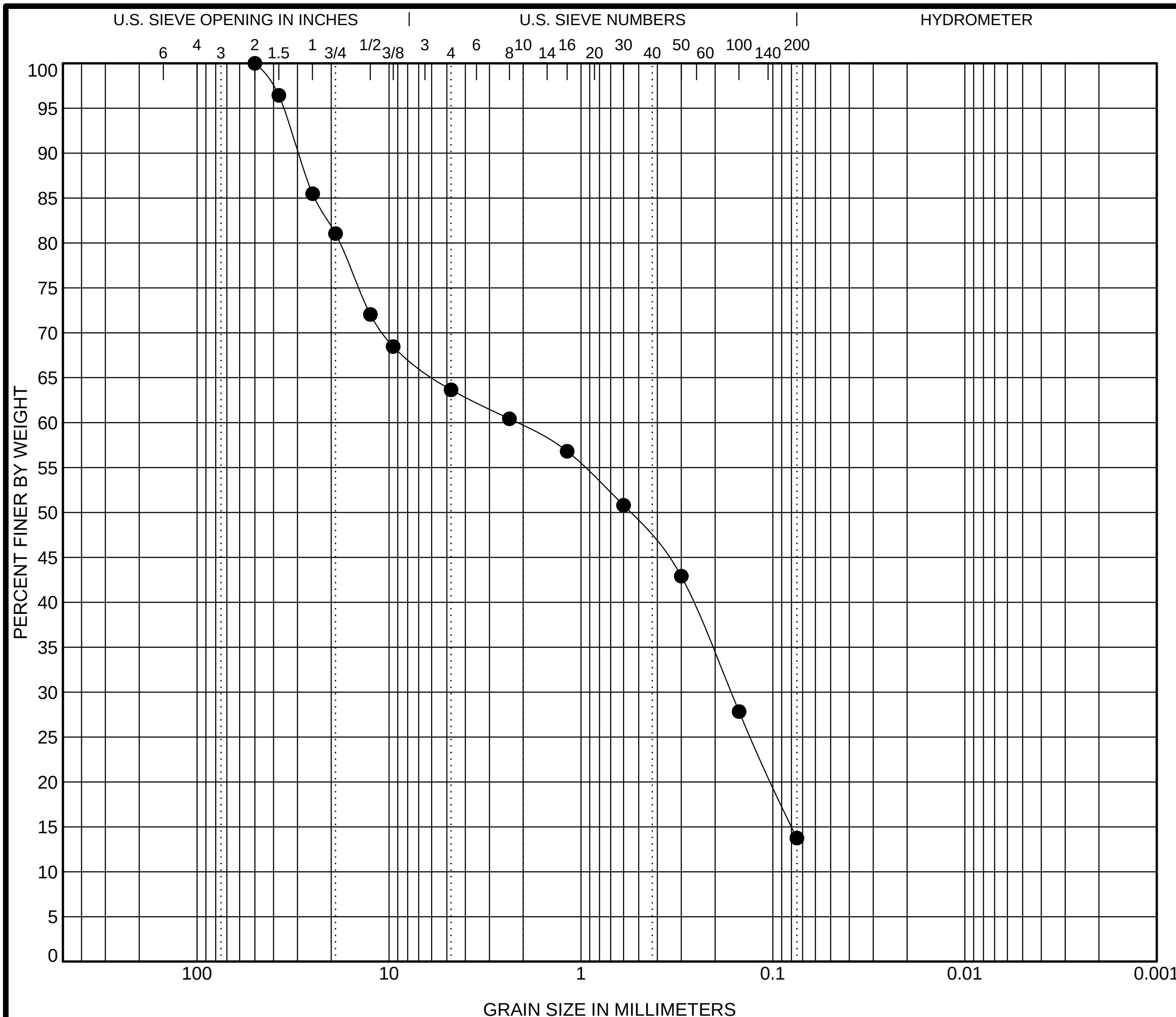
GRAIN SIZE DISTRIBUTION

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	
180325.2	September 2018	FIGURE B- 6



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **TP-1 at 1.0-3.0'**	**Silty SAND with Gravel (SM)**		

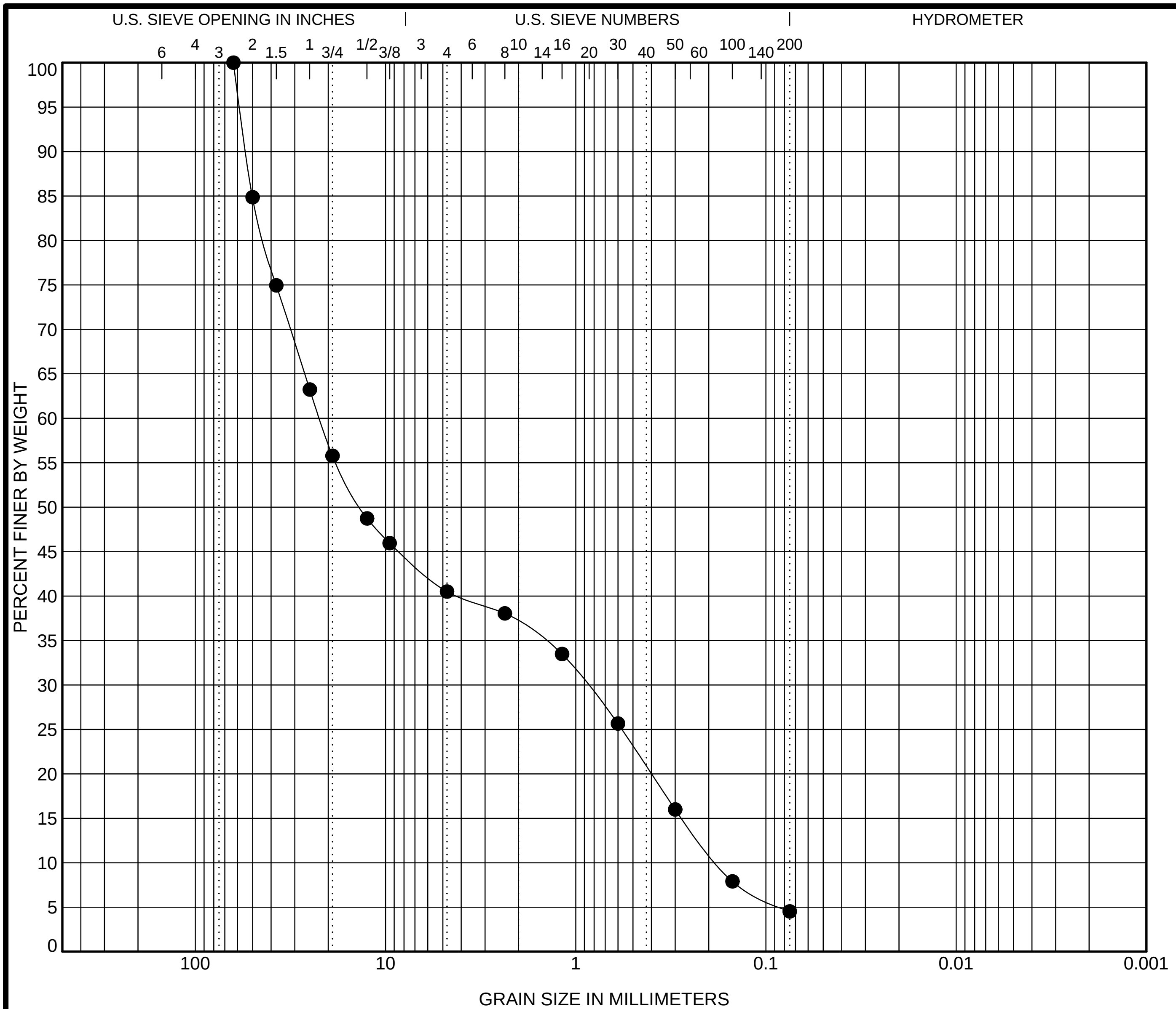
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
50	**2.174**	**0.559**	**0.166**		**36.3**	**49.9**	**13.7**	

TWINING

GRAIN SIZE DISTRIBUTION

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE B- 7
180325.2	September 2018	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **TP-1 at 3.0-3.5'**	**Poorly graded GRAVEL with Sand (GP)**	**0.19**	**123.59**

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
63	**22.194**	**13.477**	**0.873**	**0.18**	**59.5**	**36.0**	**4.5**	

GRAIN SIZE 15GT15 MANZANITA DRIVE WATER GROUP 968.GPJ TWINING LABS.GDT 9/14/18

TWINING

GRAIN SIZE DISTRIBUTION

15GT15-Manzanita Canyon Water & Storm Drain Group 968
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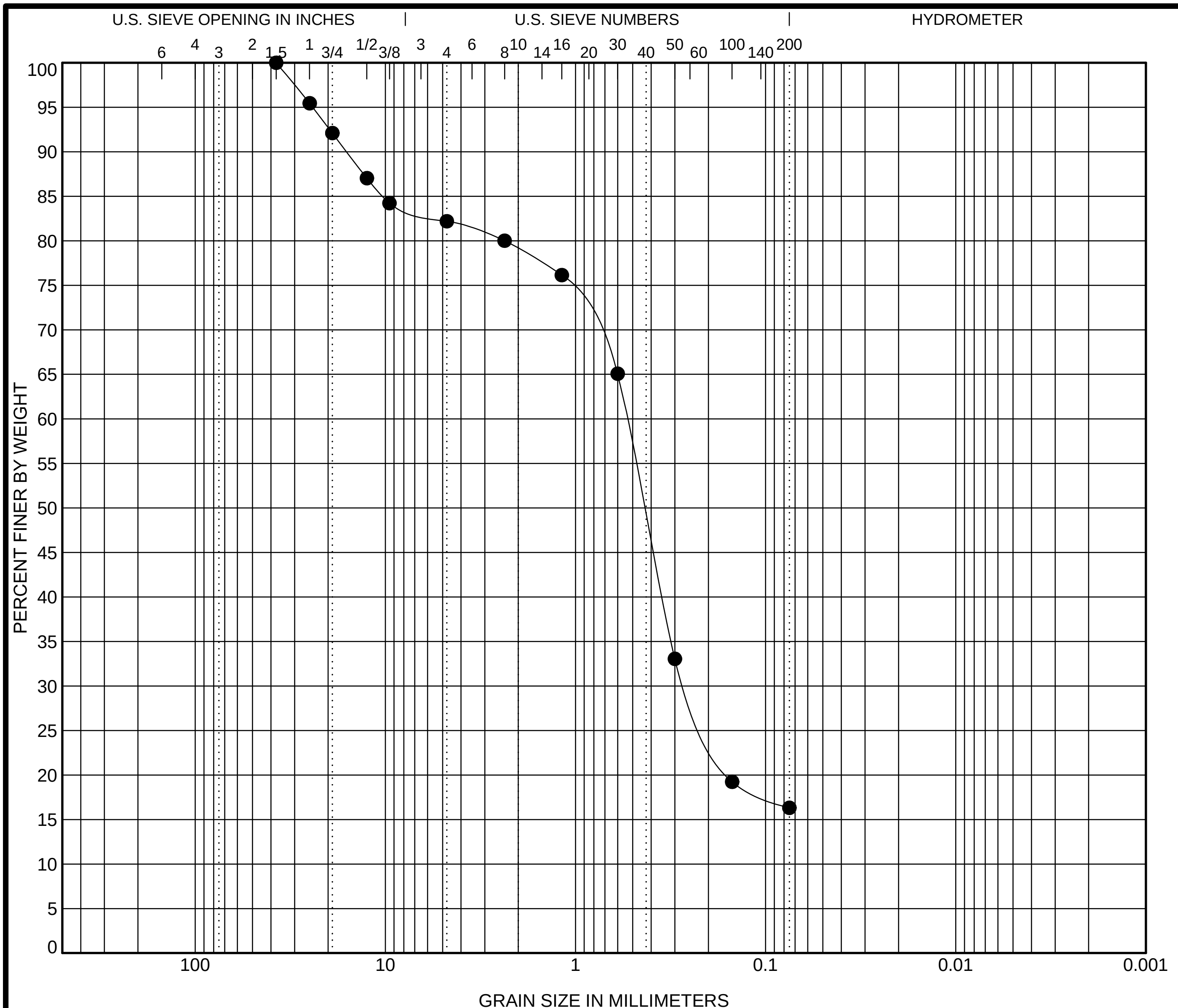
PROJECT NO.	REPORT DATE	
180325.2	September 2018	FIGURE B- 8



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

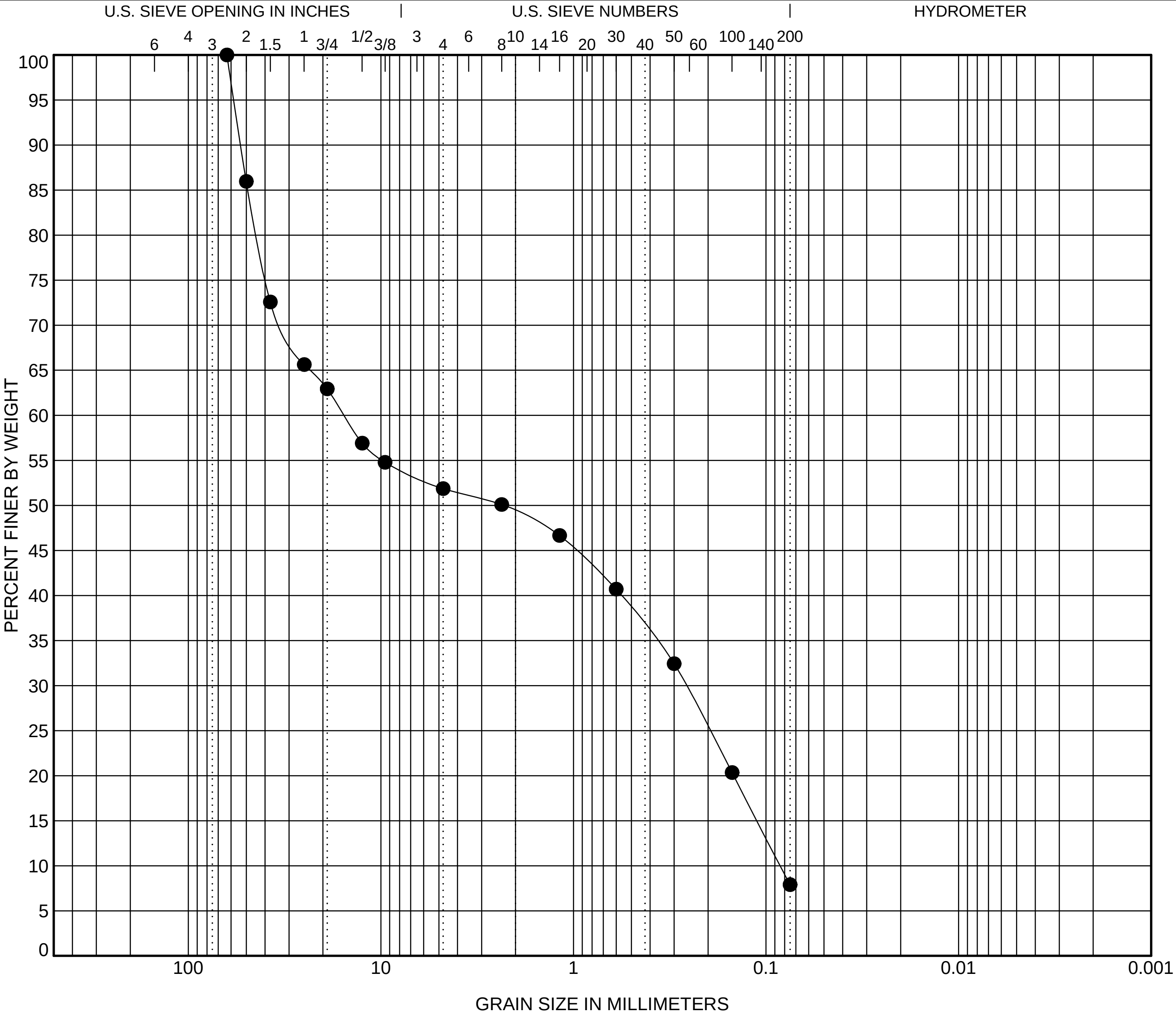
Sample Location	U.S.C.S. Classification	Cc	Cu
● **TP-1 at 3.5-4.5'**	**Silty SAND with gravel (SM)**		

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
37.5	**0.538**	**0.433**	**0.257**		**17.8**	**65.9**	**16.3**	

GRAIN SIZE DISTRIBUTION

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	
180325.2	September 2018	FIGURE B- 9



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **TP-2 at 1.0-3.0'**	**Poorly graded SAND with Silt and Gravel (SP-SM)**	**0.05**	**183.84**

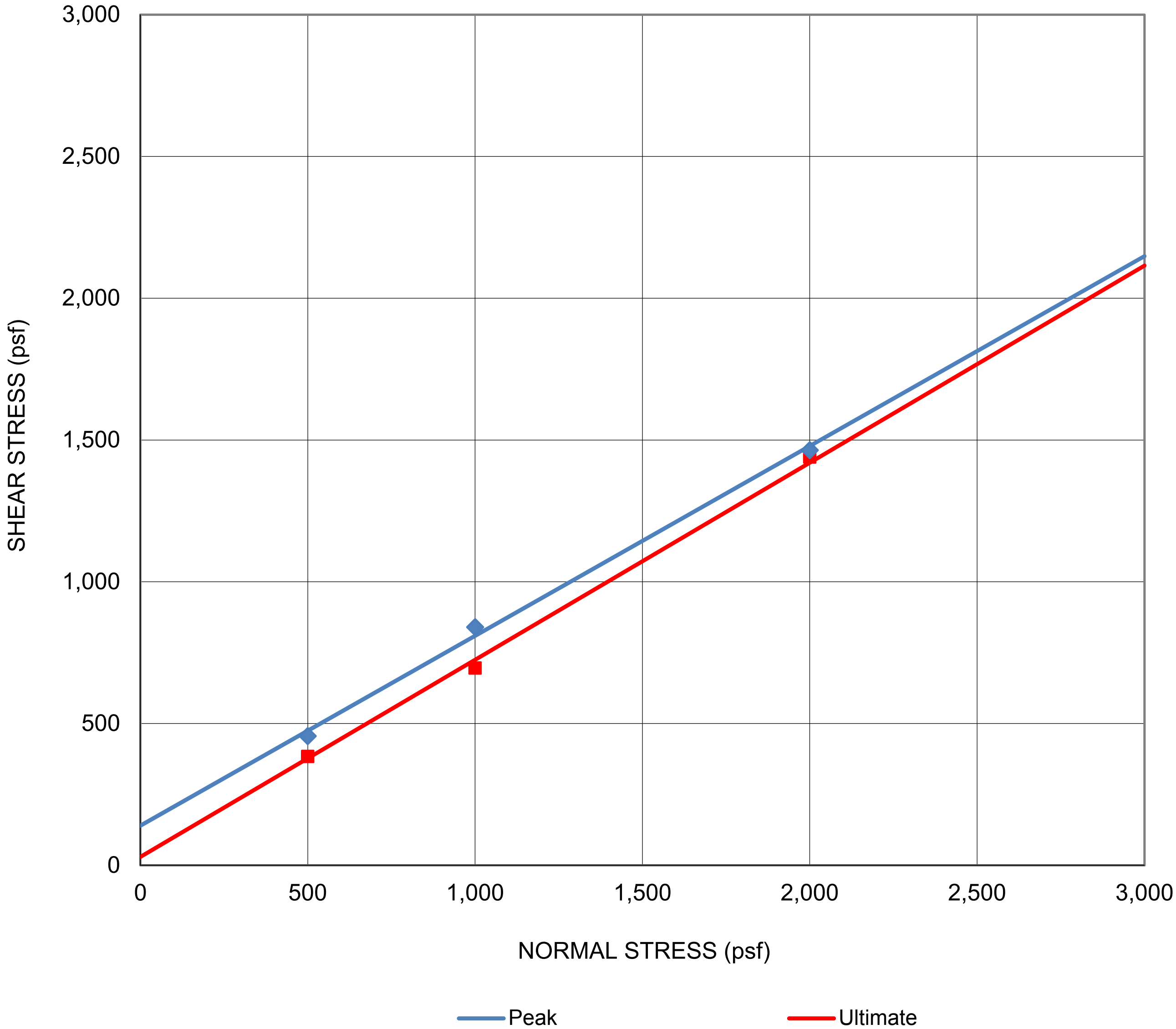
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
63	**15.488**	**2.309**	**0.261**	**0.084**	**48.1**	**44.0**	**7.9**	

TWINING

GRAIN SIZE DISTRIBUTION

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	
180325.2	September 2018	FIGURE B- 10



	Peak	Ultimate
Friction Angle (Φ):	33.8 °	34.8 °
Apparent Cohesion (C):	140 psf	30 psf

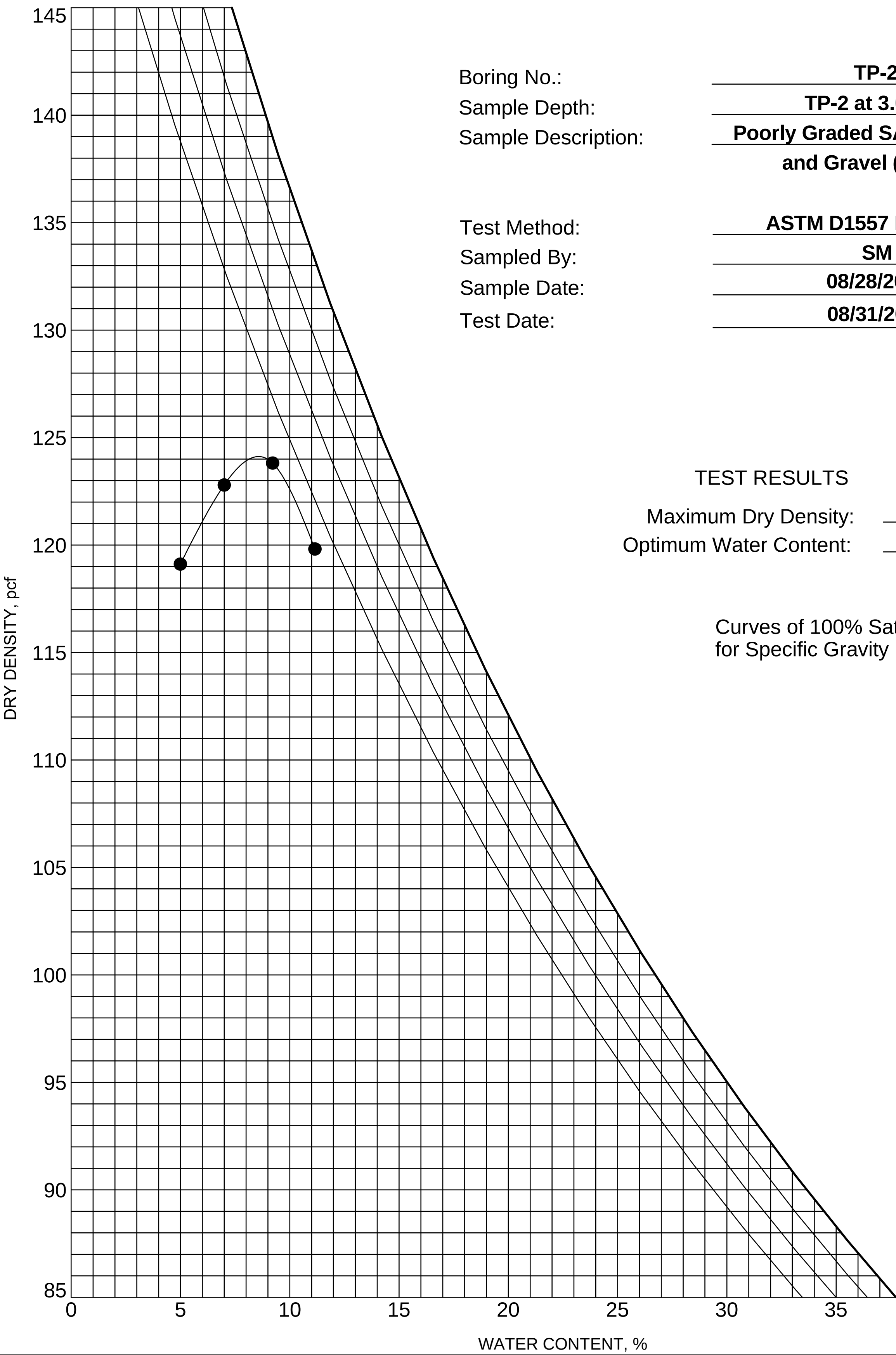
Sample Location:	B-1
Sample Depth (ft.):	10'
USCS Soil Type:	SM
Soil Description:	Silty sand

TWINING

DIRECT SHEAR TEST

15GT15 - Manzanita Canyon Water and Storm Drain Group 968
Manzanita Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE B-11
180325.2	Sep-2018	



Boring No.:	**TP-2**
Sample Depth:	**TP-2 at 3.0-4.0'**
Sample Description:	**Poorly Graded SAND with Silt and Gravel (SP-SM)**

Test Method:	**ASTM D1557 Method C**
Sampled By:	**SM**
Sample Date:	**08/28/2018**
Test Date:	**08/31/2018**

TEST RESULTS

Maximum Dry Density: **124.0** **pcf**

Optimum Water Content: **8.5** **%**

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

2.50

MOISTURE-DENSITY RELATIONSHIP

15GT15-Manzanita Canyon Water & Storm Drain Group 968
Manzanitar Drive & 39th Street
San Diego, California

PROJECT NO.	REPORT DATE	FIGURE B-12
180325.2	September 2018	

TWINING