REPORT OF GEOTECHNICAL INVESTIGATION
SERRA MESA STORM DRAIN
REPLACEMENT PROJECT
CITY OF SAN DIEGO

Submitted to:

RICK ENGINEERING COMPANY
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Prepared By:

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AGE Project No. 178 GS-16-C

November 14, 2019

Allied Geotechnical Engineers, Inc.

November 4, 2019

Mr. Salvador Galvan, P.E.
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Subject: **REPORT OF GEOTECHNICAL INVESTIGATION**
SERRA MESA STORM DRAIN REPLACEMENT PROJECT
CITY OF SAN DIEGO
AGE Project No. 178 GS-16-C

Dear Mr. Galvan:

Allied Geotechnical Engineers, Inc. is pleased to submit the accompanying updated report to present the findings, opinions, and recommendations of a geotechnical investigation that was performed to assist Rick Engineering Company 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,

ALLIED GEOTECHNICAL ENGINEERS, INC.

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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 Rick Engineering Company (Rick Engineering) with their design of the Serra Mesa Storm Drain Replacement Project for the City of San Diego (City). The investigation was performed in conformance with AGE's proposal dated February 19, 2016 (revised May 6, 2016), and the subconsultant agreement entered into by and between Rick Engineering and AGE on July 8, 2016.

This report has been prepared for the exclusive use of Rick Engineering and its design team 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

The Serra Mesa Storm Drain Replacement Project consists of the design and replacement of seven (7) storm drain pipes and curb inlets that convey street runoff from the mesa tops into the bottom of several natural canyons in the community of Serra Mesa in the City of San Diego (see Project Location Map, Figures 1). The canyon walls are moderately steep to steep, with gradients of up to 1.5:1 (horizontal:vertical). The canyons are densely vegetated with native and non-native shrubs and trees.

It is our understanding that the existing corrugated metal pipe (CMP) storm drains have been in operation for many years and have experienced corrosion and are leaking and in degraded condition. Two of the storm drains have failed, resulting in significant erosional damage from the uncontrolled runoff.

A review of the 60% Submittal Plans prepared by Rick Engineering (September 14, 2017) indicates that new storm drains will be installed in both the existing trenches and in new trenched excavations. The seven proposed storm drain pipelines addressed in our investigation are described below. The storm drains are identified as Sites 1 through 7, as indicated on the plans (Rick, 2017). Site plans showing the existing and proposed storm drain pipelines are presented on Figures 2 through 8.

Site 1

The existing 18-inch diameter storm drain pipeline extends in a westerly direction into a canyon from a storm drain inlet between 2866 and 2874 Monarch Street. The upper portion of the pipe extends through fill materials that were placed during construction of the residential subdivision on Monarch Street and the surrounding area. The fill slope was constructed at an approximate gradient of 2:1 (horizontal:vertical). Lower portions of the pipe extend through colluvial deposits on the canyon walls. The storm drain outfall is at the bottom of the canyon in a palm tree grove.

Approximately 138 feet of 18-inch RCP and 76 feet of 18-inch CMP starting from the backlot property line extending down to the bottom of the canyon will be retired. The new pipeline will be 24-inch diameter RCP which will connect to an existing clean out structure at the rim of the canyon, and extend down to a new headwall and rip rap at the bottom of the canyon. The new pipeline will be constructed in a new trench.

Site 2

The existing 18-inch diameter storm drain pipeline extends in a southerly direction into a canyon from a storm drain inlet on a cul-de-sac between 2703 and 2704 Monarch Street. The upper portion of the pipe extends through fill materials that were placed during construction of the residential subdivision on Monarch Street and the surrounding area. The fill slope was constructed at an approximate gradient of 2:1 (horizontal:vertical). Lower portions of the pipe extend through colluvial deposits on the canyon walls. The storm drain outfall is at the bottom of the canyon in a palm tree grove, downstream of Storm Drain 1. Storm Drains 3 and 4 also empty into the same canyon.

Approximately 46 feet of 18-inch RCP and 280 feet of 18-inch CMP starting from the backlot property line extending down to the bottom of the canyon will be retired. The new pipeline will be 24-inch diameter RCP which will connect to an existing clean out structure at the rim of the canyon, and extend down to a new headwall and rip rap at the bottom of the canyon. The new pipeline will be constructed in a new trench.

Site 3

The existing 18-inch CMP storm drain pipeline extends in an east to northeasterly direction into a canyon from a storm drain inlet at the dead end on Shawn Avenue east of Marathon Drive. The upper portion of the pipe extends through fill materials that were placed during construction of various residences and apartment buildings on Shawn Avenue and surrounding streets. The fill slope was constructed at an approximate gradient of 2:1 (horizontal:vertical). Lower portions of the pipe extend through colluvial deposits on the canyon walls. We could not locate the outfall of the storm drain, which appears to be in an area of dense brush near the canyon bottom.

An erosion scar measuring approximately 100 feet or greater in length, 30 feet in width and up to 20 feet in depth has developed on the slope along the existing storm drain pipeline alignment (see Figure 9). The headscarp of the slope failure is approximately 80 feet east of the dead end on Shawn Avenue, at an elevation approximately 35 feet lower than the street. The existing pipe is separated and broken at the top of the erosion scar. A broken section of the CMP pipe is also present in the bottom of the slope failure. Fill materials consisting of silty sands containing scattered gravel and cobbles are exposed on the side walls of the slope failure. Dense formational sandstone and conglomerate materials are exposed in the bottom of the failure.

Approximately 224 feet of the existing pipeline starting from the rim of the canyon extending down to the bottom of the canyon will be retired. The new pipeline will be a 24-inch diameter RCP which will connect to an existing clean out structure at the rim of the canyon, and extend down to a new headwall and rip rap at the bottom of the canyon. The new pipeline will be constructed in a new trench.

Site 4

The existing 18-inch storm drain pipeline extends in a northeasterly direction into a canyon from a storm drain inlet between 2673 and 2681 Palace Drive. The upper portion of the pipe extends through fill materials that were placed during construction of the residential subdivision on Palace drive and the surrounding area. The fill slope was constructed at an approximate gradient of 2:1 (horizontal:vertical). Lower portions of the pipe extend through colluvial deposits on the canyon walls. We could not locate the outfall of the storm drain, which appears to be in an area of dense brush near the canyon bottom.

Approximately 25 feet of 18-inch RCP and 251 feet of 18-inch CMP starting from the backlot property line extending down to the bottom of the canyon will be retired. The new pipeline will be 24-inch diameter RCP which will connect to an existing clean out structure at the rim of the canyon, and extend down to a new headwall and rip rap at the bottom of the canyon. The new pipeline will be constructed in a new trench.

Site 5

The 18-inch diameter storm drain pipeline extends in a southerly direction into a canyon from a storm drain inlet between 9570 and 9587 Larrabee Avenue. The upper portion of the pipe extends through fill materials that were placed during construction of the residential subdivision on Larrabee Avenue and the surrounding area. The fill slope was constructed at an approximate gradient of 2:1 (horizontal:vertical). Lower portions of the pipe extend through colluvial deposits on the canyon walls. Segments of the existing pipeline are exposed on the slope. We could not locate the outfall of the storm drain, which appears to be in an area of dense brush near the canyon bottom. This canyon descends to the Kinder Morgan tank farm facility on Qualcomm Way.

Approximately 100 feet of 18-inch RCP and 184 feet of 18-inch CMP starting from the south side of Larabee Avenue extending down to the bottom of the canyon will be retired. The new pipeline will be 24-inch diameter RCP which will connect to an existing inlet on the south side of Larabee Avenue, extend along an 8-foot wide easement between 9570 and 9587 Larrabee Avenue, and continue down to a new headwall and rip rap at the bottom of the canyon. The new pipeline will be constructed in a new trench.

Site 6

The 18-inch diameter storm drain pipeline extends in an easterly direction into a canyon from a storm drain inlet between 2811and 2821 Walker Drive. The canyon is located in the Serra Mesa Open Space. The upper portion of the pipe extends through fill materials that were placed during construction of the residential subdivision on Walker Drive and the surrounding area. The fill slope was constructed at an approximate gradient of 2:1 (horizontal:vertical). Lower portions of the pipe extend through formational soil materials and colluvial deposits mantling the canyon walls with shallow fill materials at the canyon bottom. The outfall of the storm drain is near the bottom of the canyon.

Approximately 36 feet of 18-inch RCP and 287 feet of 18-inch CMP starting from the backlot property line extending down to the bottom of the canyon will be retired. The new pipeline will be 24-inch diameter RCP which will connect to an existing clean out structure at the rim of the canyon, and extend down to a new headwall and rip rap at the bottom of the canyon. The new pipeline will be constructed in the same trench as the existing pipeline.

Site 7

The existing 18-inch diameter storm drain pipeline extends in a southwesterly direction into a canyon from a storm drain inlet between 2870 and 2884 Mobley Street. The canyon is located in the Sandrock Canyon Open Space. The entirety of the pipe extends through fill materials that were placed during construction of the residential subdivision on Mobley Street and the surrounding area. The fill slope was constructed at an approximate gradient of 2:1 (horizontal:vertical). The outfall of the storm drain is near the bottom of the canyon. A concrete apron approximately 18 feet in width and 50 feet in length is present below the outfall. A high pressure natural gas pipeline operated and maintained by SDG&E crosses below the concrete apron.

An erosion scar measuring approximately 100 feet in length, 20 feet in width and up to 30 feet in depth has developed on the slope along the existing storm drain pipeline alignment (see Figure 10). Sidewalls of the erosion scar are nearly vertical. The headscarp of the slope failure is within approximately 50 feet of the residence at 2870 Mobley Street, at an elevation approximately 25 feet lower than the street. The existing pipe is separated and broken at the top of the erosion scar, at the location of a cut-off wall. The erosion has undercut the slope by up to 15 feet behind the cut-off wall. Portions of the existing pipe are exposed in the side walls of the erosion scar.

The majority of the erosion scar occurs in fill materials which were utilized to fill an existing side canyon during grading of the residential subdivision. These soil materials generally consist of silty sand containing scattered to locally abundant sub-rounded gravel and cobble clast up to 10-inches in maximum dimension and local chunks of red-brown cemented sandstone derived from terrace deposits on the top of the mesa. Dark brown clayey sand colluvium containing abundant gravel and cobbles is exposed in the bottom of the erosion scar.

Approximately 140 feet of 18-inch RCP and 97 feet of 18-inch CMP starting from the backlot property line extending down to the bottom of the canyon will be retired. The new pipeline will be 24-inch diameter RCP which will connect to two existing inlets on the east side of Mobley Street. The new pipeline crosses to the west side of Mobley Street, extend along an 8-foot wide easement between 2870 and 2884 Mobley Street, and continue down to a new headwall structure and rip rap at the bottom of the canyon. The upper portion of the new pipeline will be constructed in the same right-of-way as the existing pipeline.

The proposed new pipeline extends through the erosion scar discussed herein. It is noted that the invert elevation of the proposed pipeline is at higher elevation than the bottom of the erosion scar. Therefore, construction of the new pipeline will require prior slope restoration.

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 alignments, including the preliminary project plans, as-built utility maps, topographic maps, published geologic literature and maps, and AGE's in-house references.

3.2 Geotechnical Field Exploration

The field exploration program for this project was performed during the period between September 18 and 27, 2019. A total of fourteen (14) test pits were performed at the approximate locations shown on the Site Plans (see Figures 2 through 8). The pits were excavated using manual labor to depths ranging from 4 feet to 9 feet below the existing ground surface (bgs). A more detailed description of the excavation and sampling activities, and 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 pit locations with respect to existing buried utilities. The utility clearance effort revealed the presence of the following buried utilities: sanitary sewer pipelines; storm drains; and natural gas transmission lines.

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, expansion index, 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

The project study area encompasses a number of natural canyons incised into a mesa top north of Mission Valley. Mapped geologic units on the mesa consist of nearly flat-lying to gently southwest dipping, marine and non-marine sediments which range from Holocene to Eocene in age. Man-made fills occur in various locations throughout the project study area. Nearby land uses include residential and commercial developments, the Kinder Morgan facility, San Diego County Credit Union (SDCCU) Stadium, schools, public parks and open spaces.

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.

4.3 Geologic Units

Based on their origin and compositional characteristics, the soil types encountered in the test pits can be categorized into five geologic units which include (in order of increasing age) fill materials; young colluvial deposits; very old paralic deposits; Mission Valley Formation; and Stadium Conglomerate. The Friars Formation is mapped in the project study area but was not encountered in the test pits. A generalized local geology map is shown on Figure 11 and a brief description of each unit is presented below.

4.3.1 Fill Materials (Qaf)

Fill materials ranging from 1 foot to 9 feet in thickness were encountered in test pits TP-1A, TP-2A, TP-3A, TP-5A, TP-6B, TP-7A, and TP-7B. The fill materials extended to the maximum depths of exploration in test pits TP-1A, TP-5A, and TP-7A. Fill materials encountered in the test pits generally consist of fine to medium-grained silty sands, clayey sands and minor sandy clay containing scattered to locally abundant sub-rounded to sub-angular gravels and cobbles. Scattered construction debris, including broken rock and glass, were encountered in several of the test pits. The fill materials appears to be associated with the original development of the various residential subdivisions in the project study area. Documentation pertaining to the original placement of the fill materials is unavailable.

Fill materials encountered in the test pits were typically dry and loose/crumbly near the surface, becoming moist and moderately well compacted below a depth of approximately 1-foot bgs. Locally abundant roots and rootlets were encountered in the upper portions of the fill, with only occasional roots and rootlets encountered below depths of approximately 1-foot bgs.

4.3.2 Young Colluvial Deposits (Qyc)

Although not mapped in the project study area, young colluvial deposits of Holocene to late Quaternary age are mapped on the lower south flanks of Mission Valley and on slope areas east of I-15 near the project study area (Kennedy and Tan, 2008). The colluvial deposits are described as poorly consolidated and poorly sorted sand and silt slopewash deposits. These deposits can generally be easily excavated with conventional heavy duty construction equipment.

Young colluvial deposits were encountered below the fill materials in test pits TP-6B and TP-7B, and from the ground surface at test pits TP-1B, TP-2B, TP-3B, TP-4B, and TP-5B. Where encountered the young colluvial deposits extended to the maximum depths of exploration except in test pit TP-1B, where the deposits are underlain by the Stadium Conglomerate.

The young colluvial deposits encountered in the test pits generally consist of gray brown to yellow brown silty sand and clayey sand containing locally abundant sub-rounded and sub-angular gravels and cobbles. Locally abundant roots and rootlets were encountered in the deposits. The deposits were typically in medium dense to dense condition, and dry to damp.

4.3.3 Very Old Paralic Deposits (Qvop8)

Very old paralic deposits Unit 8 of middle to early Pleistocene age were encountered in test pits TP-2A, TP-4A and TP-6A. The deposits extended to the maximum depths of exploration in test pits TP-2A and TP-4A, and were underlain by the Mission Valley Formation in test pit TP-6A.

The very old paralic deposits are generally described as poorly sorted, moderately permeable, reddish brown interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate. These deposits rest on Tierra Santa Terrace (Kennedy and Tan, 2008). Locally strong cementation and conglomerate layers can present difficult excavation conditions even with conventional heavy duty construction equipment.

Very old paralic deposits encountered in the test pits generally consist of reddish brown silty and clayey sandstone containing locally abundant sub-rounded gravel and cobbles. The deposits are dense to very dense, and damp.

4.3.4 Mission Valley Formation (Tmv)

The Mission Valley Formation was encountered below very old paralic deposits in test pit TP-6A, extending to the maximum depth of exploration. The Mission Valley Formation is a light olive-gray, soft and friable, fine-to medium-grained, marine and non-marine sandstone containing cobble-conglomerate tongues similar to the underlying Stadium Conglomerate (Kennedy and Tan, 2008). Based on fossil assemblages, the Mission Valley Formation has been assigned an upper Eocene age (Kennedy, 1975). The Mission Valley Formation can generally be easily excavated with conventional heavy duty construction equipment.

Mission Valley Formation encountered in our test pit consists of pale yellow, dense/very stiff, fine-grained sandstone and siltstone in a damp condition. The sandstone is moderately cemented.

4.3.5 Stadium Conglomerate (Tst)

The Stadium Conglomerate was encountered below young colluvial deposits in test pit TP-1B, and below fill materials in test pit TP-3A. This formation extended to the maximum depth of exploration in both test pits.

The Stadium Conglomerate consists of a massive cobble-conglomerate with a yellowish brown silty sand matrix that is locally strongly cemented (Kennedy and Tan, 2008). The clasts are generally rhyolite, dacite and quartzite composition, and are typically well rounded, elongated and flattened. The conglomerate is locally interbedded with lenses and layers of sandstone that is similar in composition to the matrix (Kennedy and Tan, 2008). The combination of strong cementation and locally abundant gravels and cobbles may pose difficult excavation conditions even for heavy duty construction equipment.

Stadium Conglomerate encountered in our test pits consists of a dense to very dense cobble-conglomerate with a pale yellow, medium-grained sandstone matrix in a damp to wet condition.

4.3.6 Friars Formation (Tf)

The middle Eocene age Friars Formation was not encountered in the test pits but is mapped in the eastern portions of the project study area. The Friars Formation is generally described as a yellowish gray, medium-grained, massive, poorly indurated non-marine and lagoonal sandstone and claystone with tongues of cobble-conglomerate (Kennedy and Tan, 2008). The claystone facies of the Friars Formation can pose slope stability issues. The Friars Formation can generally be easily excavated with conventional heavy duty construction equipment. The Friars Formation is overlain by the Stadium Conglomerate in the project study area.

4.3 Groundwater

At the time of our field investigation, minor seepage was encountered in test pit TP-1B above the contact with the dense, relatively impermeable Stadium Conglomerate. This test pit was excavated approximately 5 feet above the canyon bottom. Some localized standing water was present at the bottom of the canyon.

The database available at the Geotracker website (www.Geotracker.com) includes an environmental assessment report for a former gasoline service station located at 9294 Gramercy Drive. This site is located north of the project study area on the mesa top at an elevation of 375 feet above mean sea level (msl). GHD (2019) monitored a total of twelve (12) wells in March of 2019, and reported that groundwater was encountered between elevations of 341 and 351 feet msl (26 feet to 30 feet bgs).

Based on a review of the available data, the depth (elevation) of the regional groundwater table beneath the project alignments is estimated to be well below the anticipated depths of excavation for the proposed project. Groundwater may be encountered at shallow depths in the canyon bottoms, and flowing water may be encountered during the rainy season. Additionally, localized perched water conditions may be encountered along the project alignments, especially along the contact between the relatively permeable fill materials and dense, relatively impermeable formational materials.

5.0 DISCUSSIONS, OPINIONS AND RECOMMENDATIONS

5.1 Potential Geologic Hazards

A review of the City of San Diego Seismic Safety Study Geologic Hazards and Faults Map Grid 26 (2008) indicates that the mesa tops in the project study area are classified as Other Terrain, Zone 52 - Other level areas, gently sloping to steep terrain, favorable geologic structure, low risk. The lower canyon walls in the east portion of the project study area are classified as Slide-Prone Formations, Zone 23 - Friars, neutral or favorable geologic structure. Portions of the canyon bottoms are classified as Liquefaction Category 32- Low Potential, fluctuating groundwater, minor drainages. A copy of the map is shown on Figure 12.

A north trending fault is mapped east of the project study area near the west side of the I-15 freeway. This fault is classified in the Seismic Safety Study as Fault Zone 12 - Potentially active, inactive, presumed inactive or activity unknown. This fault is also mapped by Kennedy and Tan (2008).

Based on the results of this investigation, it is our opinion that none of these zones and fault are associated with geologic conditions which may pose a major hazard that cannot be mitigated during construction of the project as currently proposed by Rick Engineering (2017).

5.1.1 Faulting

There are no known (mapped) active or potentially active faults crossing the project alignments (San Diego Seismic Safety Study, 2008; Kennedy and Tan, 2008). For the purpose of this project we consider the Rose Canyon fault zone (RCFZ) to represent the most significant seismic hazard. The RCFZ is a complex set of anastomosing and en-echelon, predominantly strike slip faults that extend

from off the coast near Carlsbad to offshore south of downtown San Diego (Treiman, 1993). In San Diego Bay, this fault zone is believed to splay into multiple, subparallel strands; the most pronounced of which are the Silver Strand, Spanish Bight and Coronado Bank faults.

Previous geologic investigations on the RCFZ in the Rose Creek area (Rockwell et. al., 1991) and in downtown San Diego (Patterson et. al., 1986) found evidence of multiple Holocene earthquakes. A study by Kleinfelder (2017) at the San Diego International Airport identified two zones of active faulting. One of these faults was named the East Bay fault and the second fault was determined to be a northward extension of the Spanish Bight fault. A study by Ninyo & Moore (2018) at Seaport Village found evidence of recent movement along a fault that was determined to be a northward extension of the active Coronado Bank fault.

5.1.2 Fault Ground Rupture & Ground Lurching

There are no known (mapped) active or potentially active faults crossing the project alignments (Kennedy, 1975; City of San Diego, 2008). Therefore, the potential for fault ground rupture and ground lurching along the alignments is considered insignificant.

5.1.3 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.

Alluvial soils in the canyon bottoms are classified as having a low liquefaction potential (City of San Diego, 2008). The remainder of geologic units underlying the project study area are dense/stiff, and considered to posses a very low to negligible liquefaction potential.

The findings of our investigation determined that the project alignments are underlain with dense/stiff formational soils that are considered to have a very low to negligible liquefaction potential.

5.1.4 Landslides

A review of the published geologic maps (Kennedy & Tan, 2008 and City of San Diego, 2008) indicates that the project alignments are not located on or near any known (mapped) ancient landslides. The geologic units underlying the project study area are generally considered competent and not prone to landslide hazards. Therefore, it is our opinion that no special measures to mitigate soil landsliding will be required for design of the project.

Two large erosion gullies are present in Sites 3 and 7. These erosion gullies occurred primarily due to the failure of CMP storm drains, with the erosion largely occurring in fill materials that were placed during earthwork development of the various residential subdivisions in the study area.

5.1.5 Lateral Spread Displacement

The project alignments are underlain by competent geologic units which are not considered susceptible to seismic-induced lateral spreading.

5.1.6 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 the soil deposit. Based on the results of our investigation, the potential for differential seismic-induced settlement along the project alignments is considered very low.

5.1.7 Secondary Hazards

Given the elevation of the project alignments, the distance from the coast and absence of large bodies of water, it is our opinion that the potential of property damage along the project alignments from seismic-induced tsunamis and/or seiches is considered remote. Furthermore, the project alignments are not located within the mapped 100- and 500-year flood zones (FEMA Flood Insurance Rate Map, 2012). Therefore, the potential of property damage along the project alignments from flooding is considered low.

5.2 Soil Corrosivity

In accordance with the City of San Diego Water Facility Design Guidelines, Book 2, Chapter 7, soil is generally considered aggressive to concrete if its chloride concentration is greater than 300 parts per million (ppm) or sulfate concentration is greater than 1,000 ppm, or if the pH is 5.5 or less.

Analytical testing was performed on representative sample of the onsite soil materials 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. A summary of the test results is presented in Table 1 below. Copies of the analytical laboratory test data reports are included in Appendix B.

Table 1

Summary of Corrosivity Test Results

	pH	Resistivity (ohm-cm)	Sulfate Conc. (ppm)	Chloride Conc. (ppm)	Bicarbonates Conc. (ppm)
TP-1B Sample No. 1 @0'-1'	6.1	780	250	240	14
TP-2A Sample No. 3 @3'-3.5'	6.0	1,500	33	32	18
TP-3A Sample No. 3 @3'-4'	4.9	650	250	310	8
TP-4A Sample No. 4 @3.5'-4'	5.4	1,200	45	64	12
TP-5A Sample No. 4 @ 4.5'-5'	6.6	1,700	48	21	12
TP-6A Sample No. 4 @ 3.5'-4'	4.6	300	480	800	4
TP-7B Sample No. 4 @ 4'-5'	5.7	2,700	66	64	4

The test results indicate that the majority of the soil materials along the project alignments are generally not considered aggressive to concrete. Therefore, Type I and Type 2 Portland Cement Concrete may be used for proposed facilities along the project alignments. It should be noted here that the most effective way to prevent sulfate attack is to keep the sulfate ions from entering the concrete in the first place. This can be done by using mix designs that give a low permeability (mainly by keeping the water/cement ratio low) and, if practical, by placing moisture barriers between the concrete and the soil.

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.

5.3 Expansive Soil

Based on visual observations and soil classifications, the on-site materials are considered non-expansive or have a low expansion potential.

5.4 Slope Reconstruction

Since reconstruction of the existing slope at Sites 3 and 7 is not considered practical from worker's safety perspective, AGE recommends that the eroded gullies be backfilled using flowable fill material. The backfilling should be performed in stages. The initial stage will consist of the construction of an earthen berm with a maximum height not-to-exceed 10 feet at the bottom of the

canyon and backfilling behind the berm with the flowable fill material. Once the backfilling operation behind the berm is completed, subsequent earthen berms and flowable fill placements should be constructed at approximately 20-foot horizontal intervals. The generalized cross-section showing the conceptual slope repair design is shown on Figure 13. The operation will continue until the eroded gully is backfilled to an elevation that is considered safe for the contractor to proceed with conventional backfilling operation with soil materials.

5.4.1 Flowable Fill

Flowable fill refers to a cementitious slurry consisting of a mixture of fine aggregate or filler, water, and cementitious material(s), which is used as a fill or backfill in lieu of compacted earth. The mixture is capable of filling all voids in irregular excavations and hard to reach places, self-leveling, and hardens in a matter of a few hours without the need for compaction.

Flowable fill for the subject project should be designed with a compressive strength that will allow excavation with heavy machinery at a maximum compressive strength of 200 psi at 1 year and maximum unit weight not to exceed 110 pounds per cubic foot (pcf). The coefficient of permeability of the flowable fill should be equal or greater than that of the surrounding soil.

Flowable fill should have minimal subsidence and bleed water shrinkage. Evaporation of bleed water should not result in shrinkage of more than 1/8 inch per foot of flowable fill depth (for mixes containing high fly ash content) when measured in accordance with ASTM C 940 test method "Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory".

Flowable fill should be sampled and tested in the field in conformance with either ASTM C 94 or C 685. Samples for tests should be taken for every 150 cubic yards of material, or fraction thereof, for each day's placement. Tests should include temperature reading and four compressive strength cylinders. Compressive strength sampling and testing should conform to ASTM D 4832 with one specimen tested at 7 days, two at 28 days, and one held for each batch of four specimens.

We recommend that installation of flowable fill be performed only when approved by the Resident Engineer, and when existing and forecasted weather conditions are within the limits established by the manufacturer of the materials and products used. The mix design should produce a consistency that will result in a flowable product at the time of placement which does not require manual means to move it into place. Placement of the flowable fill should be performed in accordance with the manufacturer's mix design specifications. Flowable fill materials are considered suitable for use in slope reconstruction and as trenched excavation backfill.

5.4.2 Soil Backfill

Fill material for trench backfill and slope reconstruction should be free of biodegradable material, hazardous substance contamination, other deleterious debris, and or rocks or hard lumps greater than 6 inches. If the fill material contains rocks or hard lumps, at least 70 percent (by weight) of its particles shall pass a U.S. Standard $^{3}/_{4}$-inch sieve. Fill material should consists of predominantly granular soil (less than 40 percent passing the U.S. Standard #200 sieve) with Expansion Index of less than 30.

The majority of the soil materials observed during in the field exploration are considered suitable for use as compacted backfill materials. Materials generated from excavations in the very old paralic deposits, Stadium Conglomerate and in the existing fill embankment will require selective screening to remove large (in excess of 6 inches in maximum dimension) rock clasts prior to placement as compacted fill. It may be more economical to bring import materials for use as soil backfill. Highly expansive soil material which is not considered suitable for use as compacted fill was not encountered during the field exploration.

5.5 Earthen Berms

Prior to the slope repair operations, any loose and compressible materials shall be removed until competent materials are encountered. We recommend that all existing fill and topsoil materials be removed until competent formational material is encountered. The actual 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.

Earthen berms should be at least 20 feet wide at the top with 1 : 1 sloping sides (horizontal : vertical) or flatter. Since the berms will be left in place to become a part of the slope, we recommend that all earthern berms be constructed with 10-foot wide keys. The initial key should be extended a minimum of 24 inches into competent material to be determined by the City's Resident Engineer. Depending on the type of soil materials encountered, the initial key may need to go deeper than 24 inches. The dimensions of the key for subsequent earthern berms which will be excavated into the flowable fill may be reduced to 8 feet wide and 18 inches deep. The keys should be excavated with 2% inclination into the slope. It is noted that the dimension of the keys are based on a sliding factor of safety of 2.0 assuming a maximum of 10 feet of flowable fill behind

the berms. In the event that less than 10 feet of flowable fill is placed against the berms, the width of the keys for the berms supported on flowable fill may be reduced to 0.8H, where H is the height of the flowable fill against the face of the berms, but not less than 6 feet wide.

The earthern berms should be constructed with soil materials which meet the criteria of Section 5.4.2 of our report and compacted to 95% relative compaction (ASTM D1557). We recommend waiting until the flowable fill reaches 100 psi strength prior to constructing earthen berms on the flowable fill.

Following completion of the placement of the flowable fill to a safe elevation, the remainder of the eroded gully may be backfilled with soil backfill. Flowable fill and soil backfill should meet the specifications presented in Section 5.4.

5.6 Fill Placement

We recommend that the key at the toe of the proposed initial soil backfill for the slope repair be embedded a minimum of 18 inches into competent formational material. The key should be at a minimum 8 feet wide and excavated into the slope with 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.

The fill materials should be moisture-conditioned, placed and uniformly compacted in layers until final elevations are reached. Each layer should be no thicker than will allow for adequate bonding and compaction, but shall not exceed 6 inches in loose (uncompacted) thickness. Unless otherwise specified, all fills shall be compacted to at least 90 percent of maximum dry density as determined in the laboratory by the ASTM D1557 test method. Field density testing shall be performed in accordance with either the Sand Cone Method (ASTM D1556) or the Nuclear Gauge Method (ASTM D2922 and D3017).

5.7 Slope Stability Analyses

To evaluate the stability of the proposed slope repair configuration, we have performed a slope stability analysis using FLAC/Slope Ver. 7 for the generalized slope configurations shown on Figure 13. The stability analyses were performed using "Shear Strength Reduction Technique" which is applied to the factor-of-safety calculations by progressively reducing the shear strength of the material to bring the slope to a state of limiting equilibrium. A series of simulations were performed using trial values to reduce the cohesion (c) and friction angle (ö) until slope failures occur.

The parameters of the soil materials used in the analyses are based on engineering judgement based on our experience with similar materials (refer to Table 2). A summary of the calculated factor of safety obtained from the slope stability analysis is shown in Table 3.

Table 2

Summary of Material Properties

Material	Unit Weight (γ) (pcf)	Cohesion (c) (psf)	Shear Angle (φ) (degree)
Compacted Fill	120	0	30
Flowable Fill	110	40,000	0
Formational Material	100	0	35

Table 3

Calculated Factor of Safety

Slope Configuration	F.S. drained	F.S. undrained
Post-Repair Configuration (see Figure 13)	3.7	2.5

Generally a fill slope is designed to have a minimum factor of safety of 1.5 under static drained condition, and 1.2 under undrained or pseudo static conditions. Our analyses indicate that the post-repair slope configuration will have a factor of safety of 1.5 or greater with respect to gross and surficial slope stability. Furthermore, if properly constructed, we do not anticipate that the proposed slope restoration as shown on Figure 13 will destabilize or result in settlement of the adjacent properties. It is noted that the slope stability analysis is only applicable to the portion of the slope which will be reconstructed. It does not apply to the adjacent portions of the existing slope which are not the subject of this investigation.

5.8 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, with the exception of Site 7, the soils at the proposed invert level along the storm drain pipeline alignments are expected to provide a stable trench bottom. Site 7 will require repair of the erosion scar prior to installation of the new storm drain pipeline. 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 City's Resident Engineer.

5.8.1 Soil and Excavation Characteristics

The materials within the anticipated depths of the storm drain pipe trench excavation will likely be comprised of materials which can be readily excavated with conventional heavy-duty construction equipment.

5.8.2 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 100 pcf for the properly compacted 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 formational material and/or properly compacted filled ground, an equivalent fluid density of 350 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.

5.8.3 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 fine 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 material placed in this zone should meet or exceed the criteria presented in Section 5.4. for either flowable fill or soil backfill.

5.8.4 Placement and Compaction of Backfill

Prior to placement, all soil backfill material 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. Field density testing shall be performed in accordance with either the Sand Cone Method (ASTM D1556) or the Nuclear Gauge Method (ASTM D2922 and D3017).

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. Compaction is not required in the event that flowable fill is used to backfill the trenched excavations.

5.8.5 Concrete Anchor/Cutoff Wall

We recommend that for segments of the proposed storm drain that are 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 25-foot intervals.

6.0 CONSTRUCTION-RELATED CONSIDERATIONS

6.1 Construction Dewatering

The depth of the local groundwater table is expected to be well below the anticipated depth of the proposed trenched excavations for this project. With the exception of minor seepage in test pit TP-1B, no groundwater or seepage was encountered in the test pit excavations. We therefore do not anticipate the need for dewatering of 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. Localized perched water conditions would most likely occur at the interface between fill materials and formational materials, 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, the presence of other buried utilities, and the need to avoid excessive community disruption dictate that a shored excavation will be needed along the entire pipeline alignment. Design and construction of temporary shoring should be the sole responsibility of the contractor.

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 building floors and pavement structures. We further recommend that a weekly survey of existing utilities be performed during the construction phase.

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 colluvial deposits, 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, Mission Valley Formation, Stadium Conglomerate and Friars 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.

Lateral Bearing Capacity

Resistance to lateral loads will be provided by passive soil resistance. The allowable passive pressure for the fill materials and young colluvial deposits may be assumed to be equivalent to a fluid weighing 250 pcf. Allowable lateral bearing pressure in fill material and young colluvial deposits should not exceed 2,500 psf. Allowable passive pressure for old paralic deposits, Mission Valley Formation, Stadium Conglomerate and Friars 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 alignment. During our subsurface investigation soil samples were field screened for the presence of volatile organics using a RAE Systems MiniRAE 3000 organic vapor meter (OVM). The field screening did not reveal elevated levels of volatile organics in the samples.

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 Rick Engineering 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 test pits. 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.

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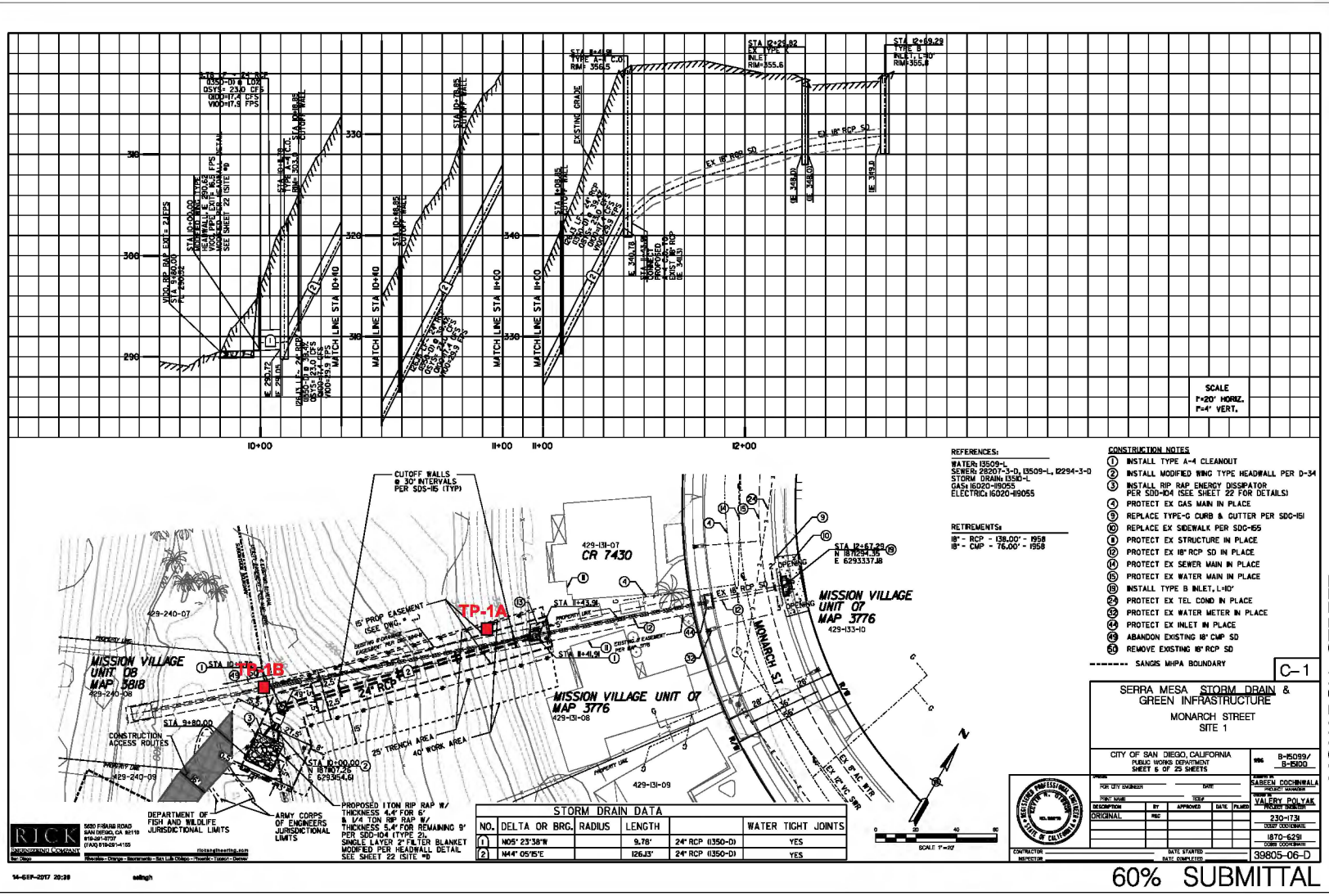
Aerial Photographs

U.S. Department of Agriculture black and white aerial photograph Nos. AXN-3M- 98, 99, 100,191 and 192 (dated 1953).

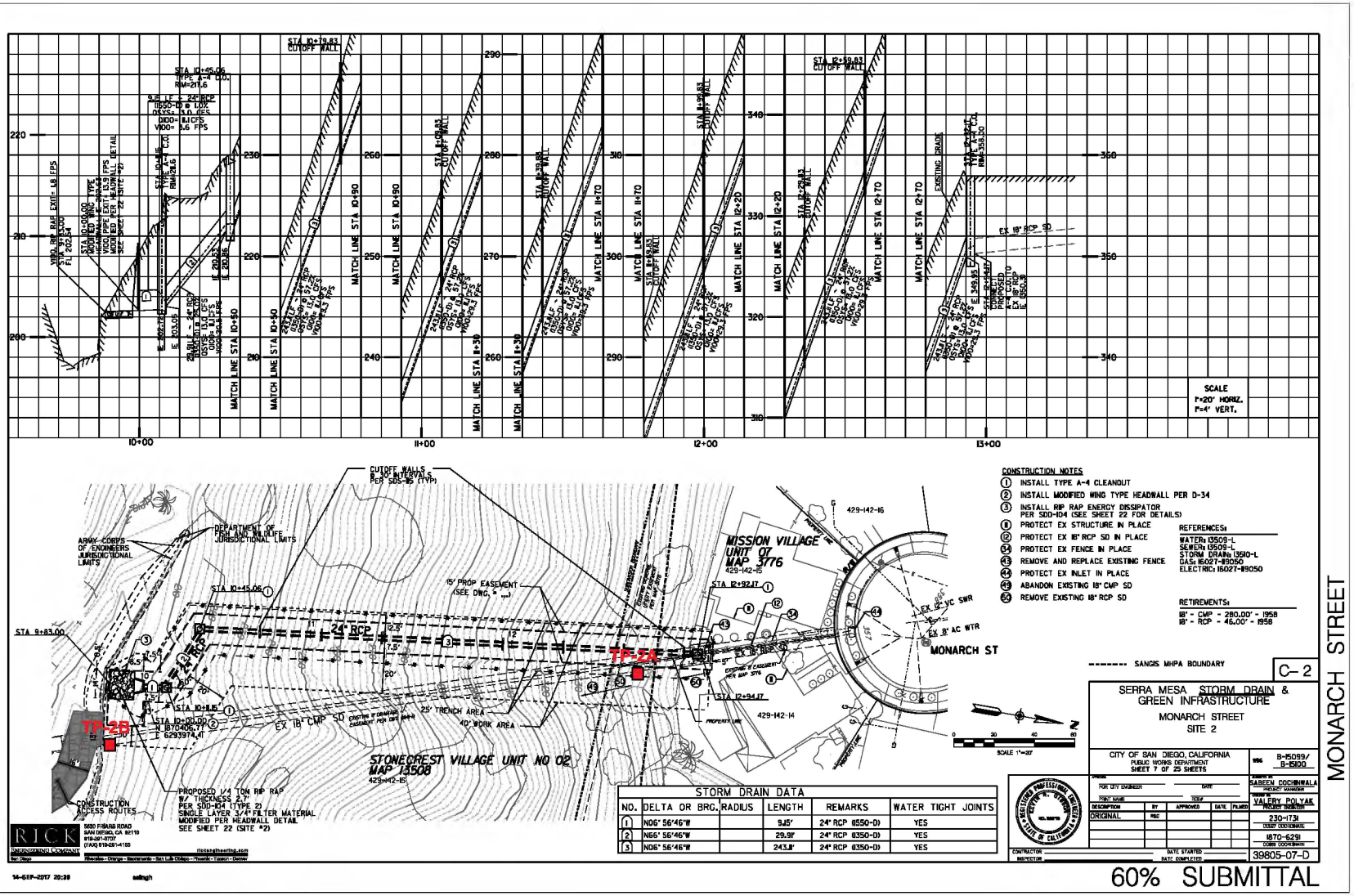
FIGURES



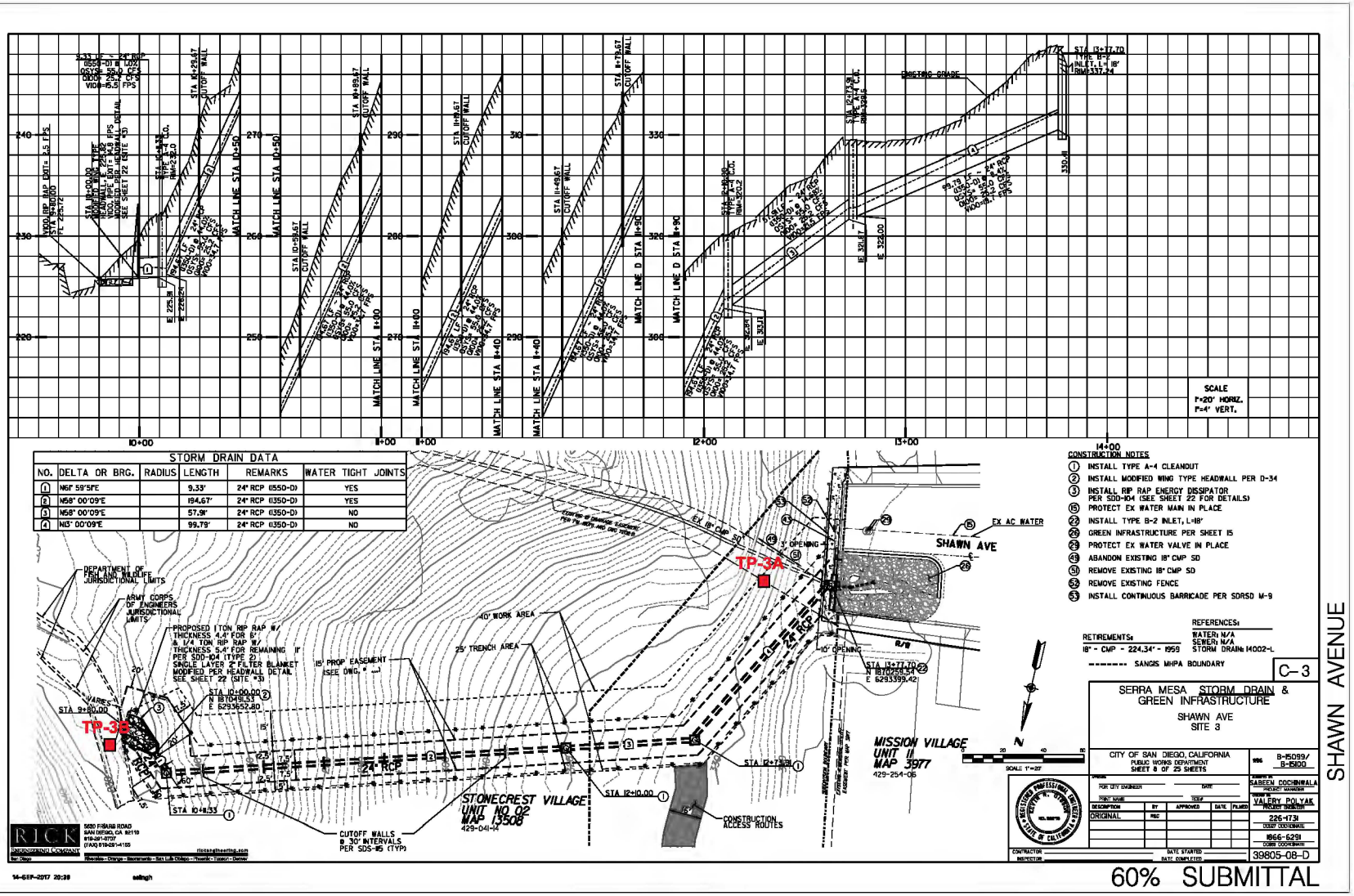
SERRA MESA STORM DRAIN REPLACEMENT PROJECT		LOCATION MAP
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 1



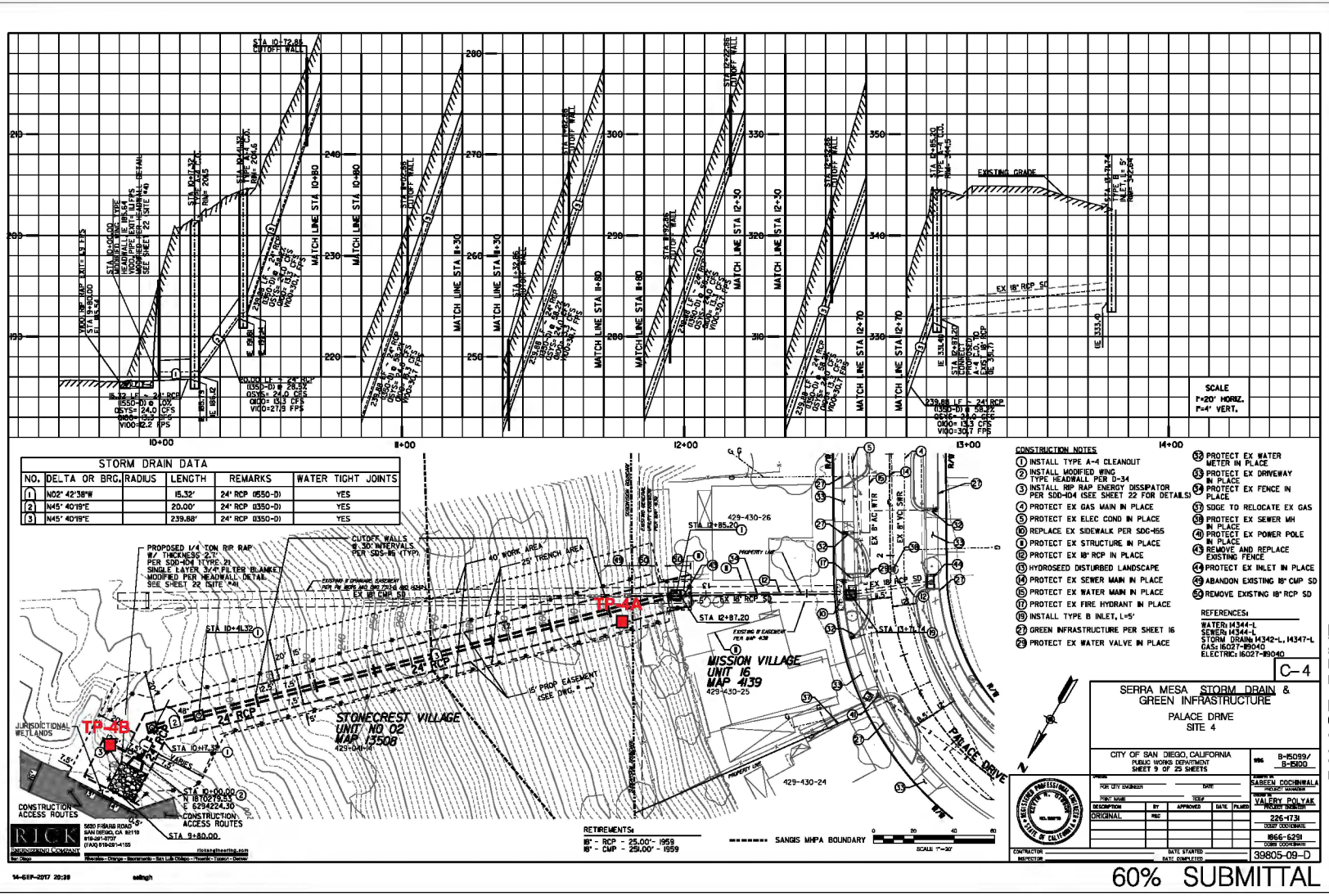
SERRA MESA STORM DRAIN REPLACEMENT PROJECT		**SITE PLAN (SITE 1)**
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE 2**



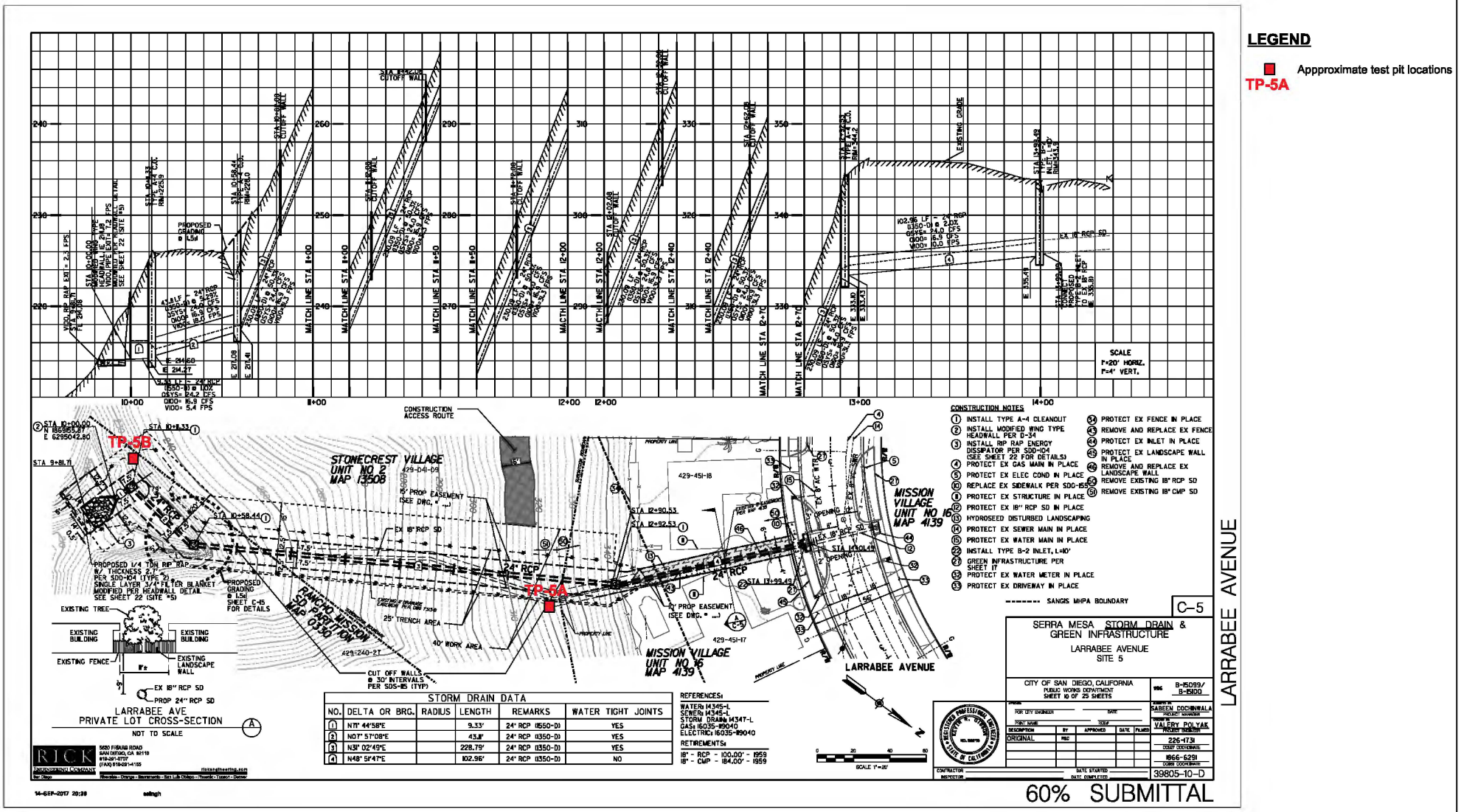
SERRA MESA STORM DRAIN REPLACEMENT PROJECT	**SITE PLAN (SITE 2)**
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.**
	FIGURE 3



SERRA MESA STORM DRAIN REPLACEMENT PROJECT	**SITE PLAN (SITE 3)**
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.** — **FIGURE 4**



SERRA MESA STORM DRAIN REPLACEMENT PROJECT	SITE PLAN (SITE 4)
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.
	FIGURE 5



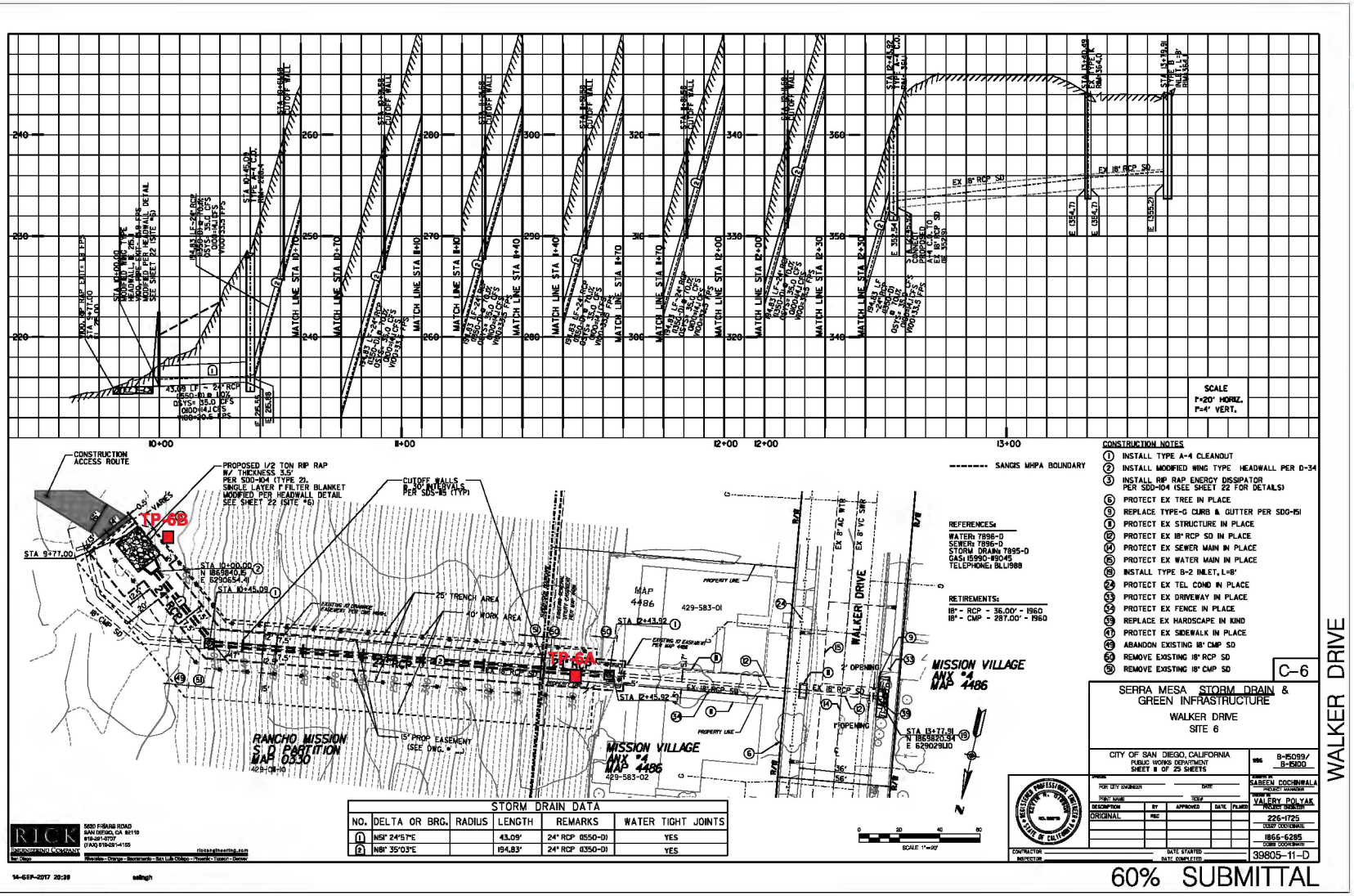
SERRA MESA STORM DRAIN REPLACEMENT PROJECT

SITE PLAN (SITE 5)

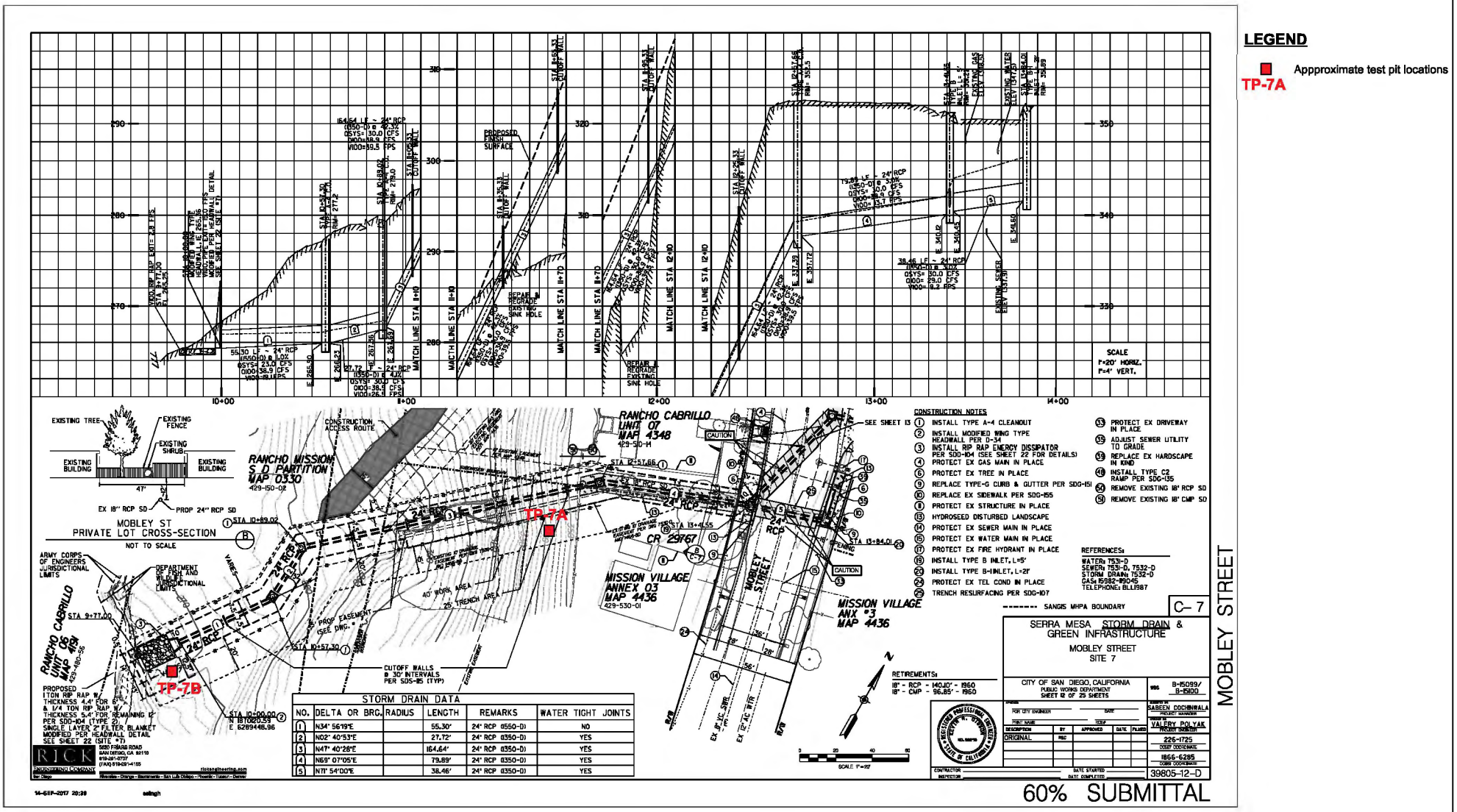
PROJECT NO.
178 GS-16-C

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE 6



SERRA MESA STORM DRAIN REPLACEMENT PROJECT		**SITE PLAN (SITE 6)**
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE 7**



SERRA MESA STORM DRAIN REPLACEMENT PROJECT

SITE PLAN (SITE 7)

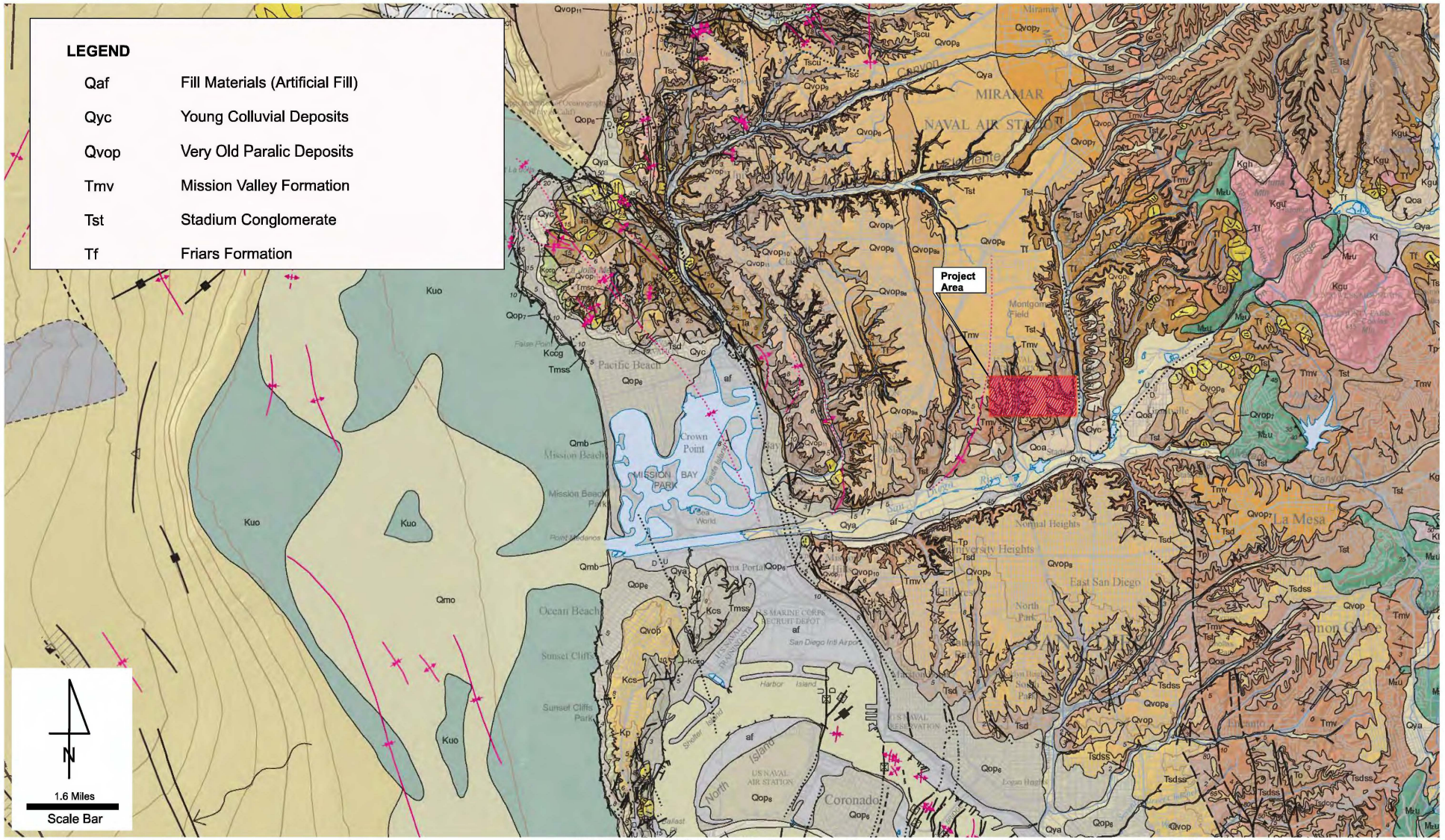
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 8



SERRA MESA STORM DRAIN REPLACEMENT PROJECT		SITE 3 SCOUR SCAR
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 9

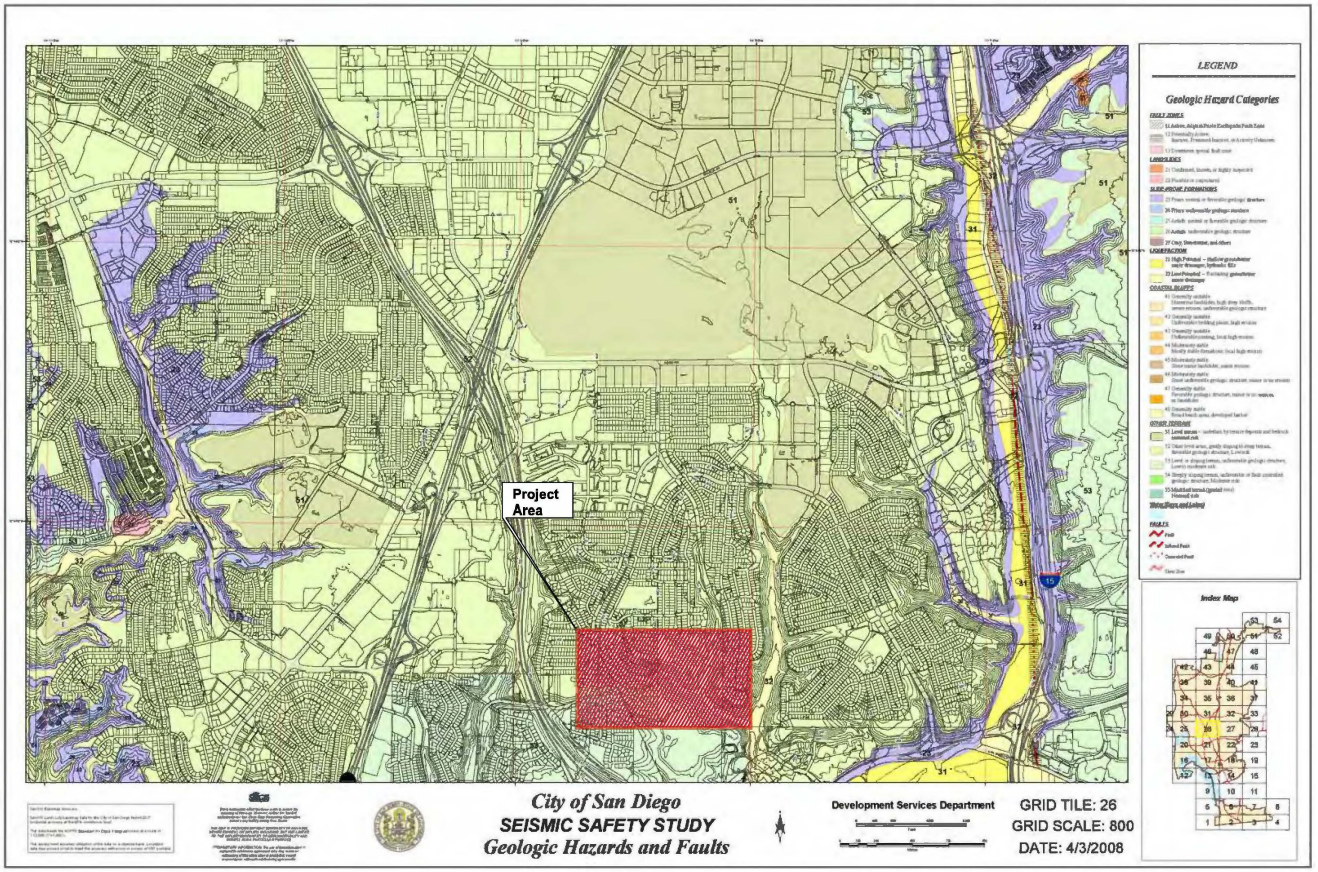


SERRA MESA STORM DRAIN REPLACEMENT PROJECT		SITE 7 SCOUR SCAR
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 10

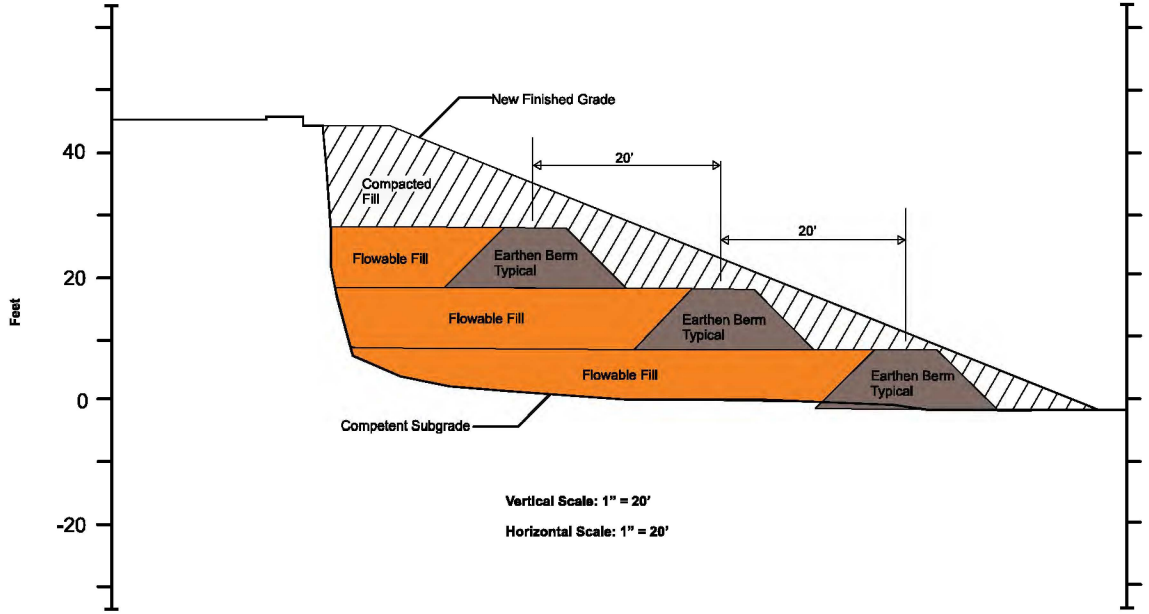


SOURCE: USGS, "Geologic Map of the San Diego 30' x 60' Quadrangle, California", by Michael P. Kennedy and Siang S. Tan, 2008.

SERRA MESA STORM DRAIN REPLACEMENT PROJECT	GENERALIZED LOCAL GEOLOGY MAP
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC. — FIGURE 11



SERRA MESA STORM DRAIN REPLACEMENT PROJECT		CITY OF SAN DIEGO SEISMIC SAFETY STUDY
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 12



SERRA MESA STORM DRAIN REPLACEMENT PROJECT	CONCEPTUAL SLOPE REPAIR DESIGN	
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 13

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

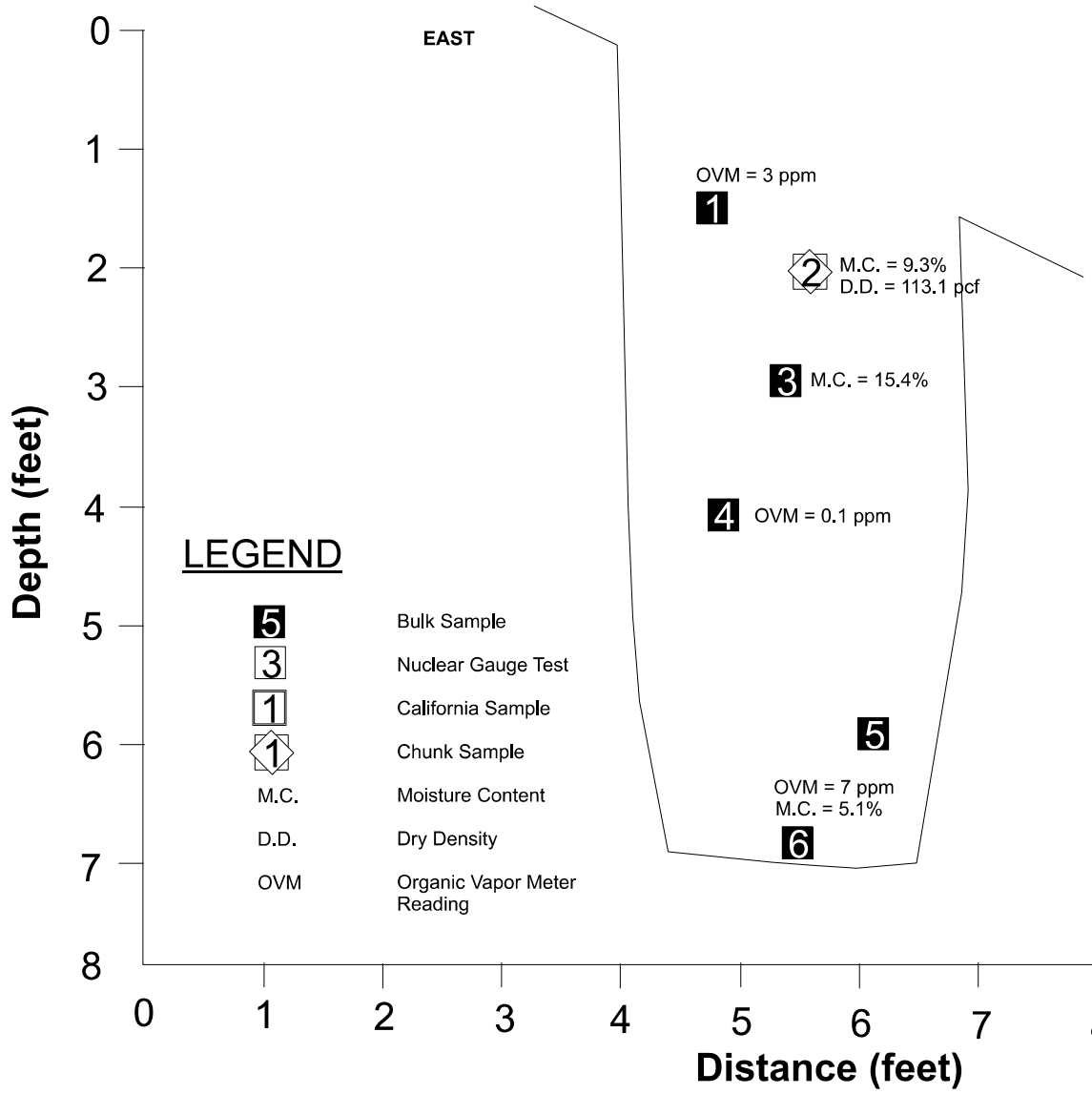
The field exploration program for this project was performed during the period between September 18 and 27, 2019. A total of fourteen (14) test pits were performed at the approximate locations shown on the Site Plans (see Figures 2 through 8). The pits were excavated using manual labor to depths ranging from 4 feet to 9 feet below the existing ground surface (bgs).

The test excavations were performed by Mansolf Excavation. The soils encountered in the pits were visually classified and logged by an experienced engineering geologist from AGE. The test pit logs are presented on Figures A-1 thru A-14. The logs depict the various soil types encountered and indicate the depths at which tests were performed and samples were obtained for laboratory testing and analysis.

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.

During the excavation, moisture and density test readings were taken in the test pits using a nuclear soil gauge (ASTM D6938-10). In addition, relatively undisturbed samples were 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 some of the test pits. The sampler was driven a distance of approximately 12 inches into the soil at the bottom of the test pit with a drop weight. A 6-inch long section of soil sample that was retained in the brass rings was extracted from the sampling tube and transported to our laboratory in close-fitting, waterproof containers. The samples were field screened for the presence of volatile organics using a RAE Systems MiniRAE 3000 organic vapor meter (OVM). The OVM readings are indicated on the test pit logs. In addition, loose bulk samples were also collected.

Upon completion of the field exploration activities, all of the test pits were backfilled with excess soil cuttings and compacted.



Ice plants, organic rich material and rootlets between 0 - 2".

FILL

Reddish brown to reddish yellow, dry, loose to crumbly silty sand (SM) with sub-rounded gravels and cobbles up to 4" in maximum dimension. Approximately 10 - 15% rock content. Appreciable rootlets and some roots up to 0.5" in diameter. Soil become denser below depth of 12", moist and few rootlets.

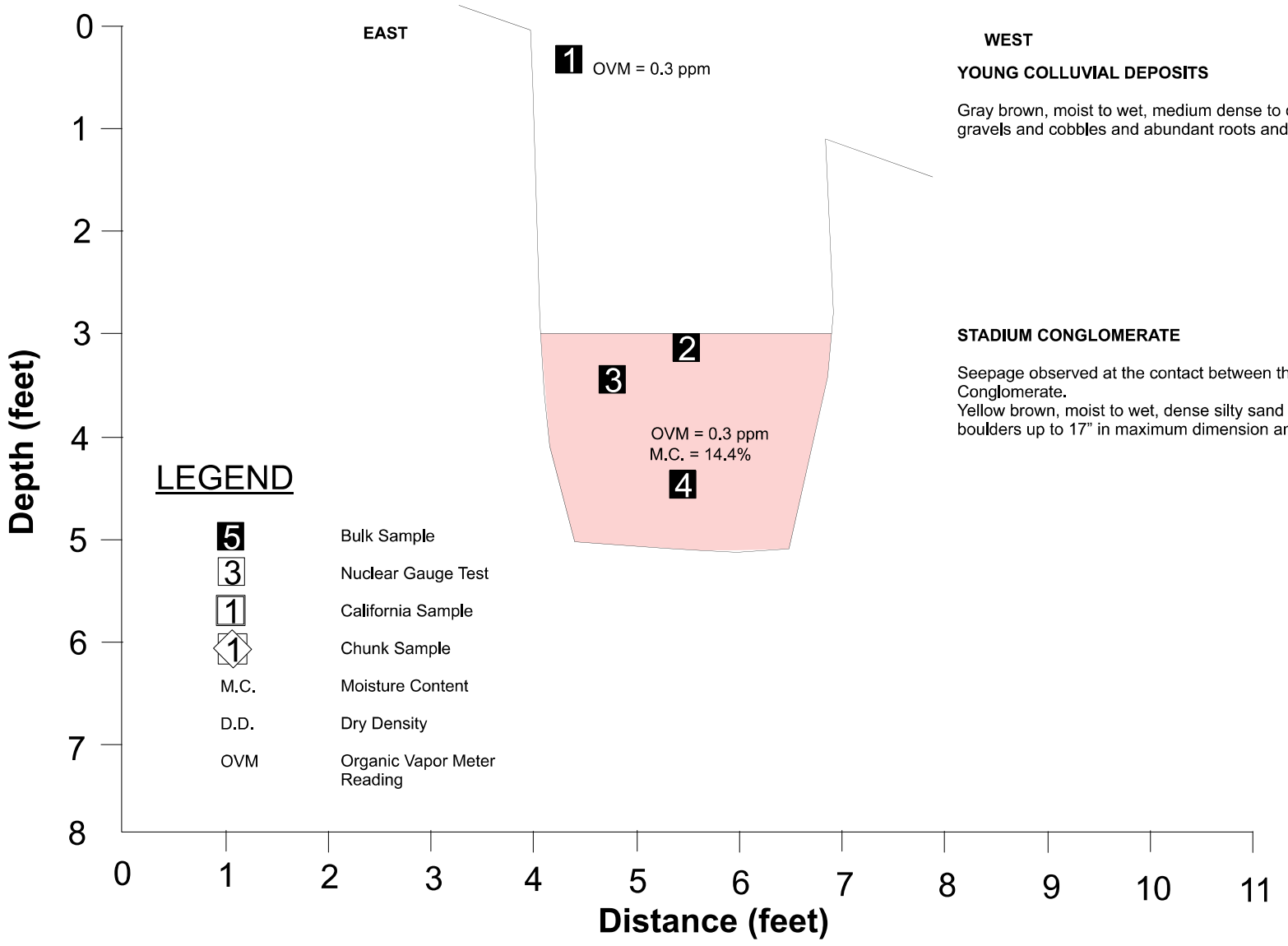
Dark gray brown, moist to wet, stiff sandy clay (CL). Approximately 10 - 15% rock up to 4" in maximum dimension.

Yellowish red to reddish brown, moist, silty sand (SM).

8" sub-rounded cobble observed in east wall.

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

SERRA MESA STORM DRAIN PROJECT	**TEST PIT LOG OF TP-1A**
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.** — **FIGURE A-1**



YOUNG COLLUVIAL DEPOSITS

Gray brown, moist to wet, medium dense to dense, clayey sand (SC) with gravels and cobbles and abundant roots and rootlets.

STADIUM CONGLOMERATE

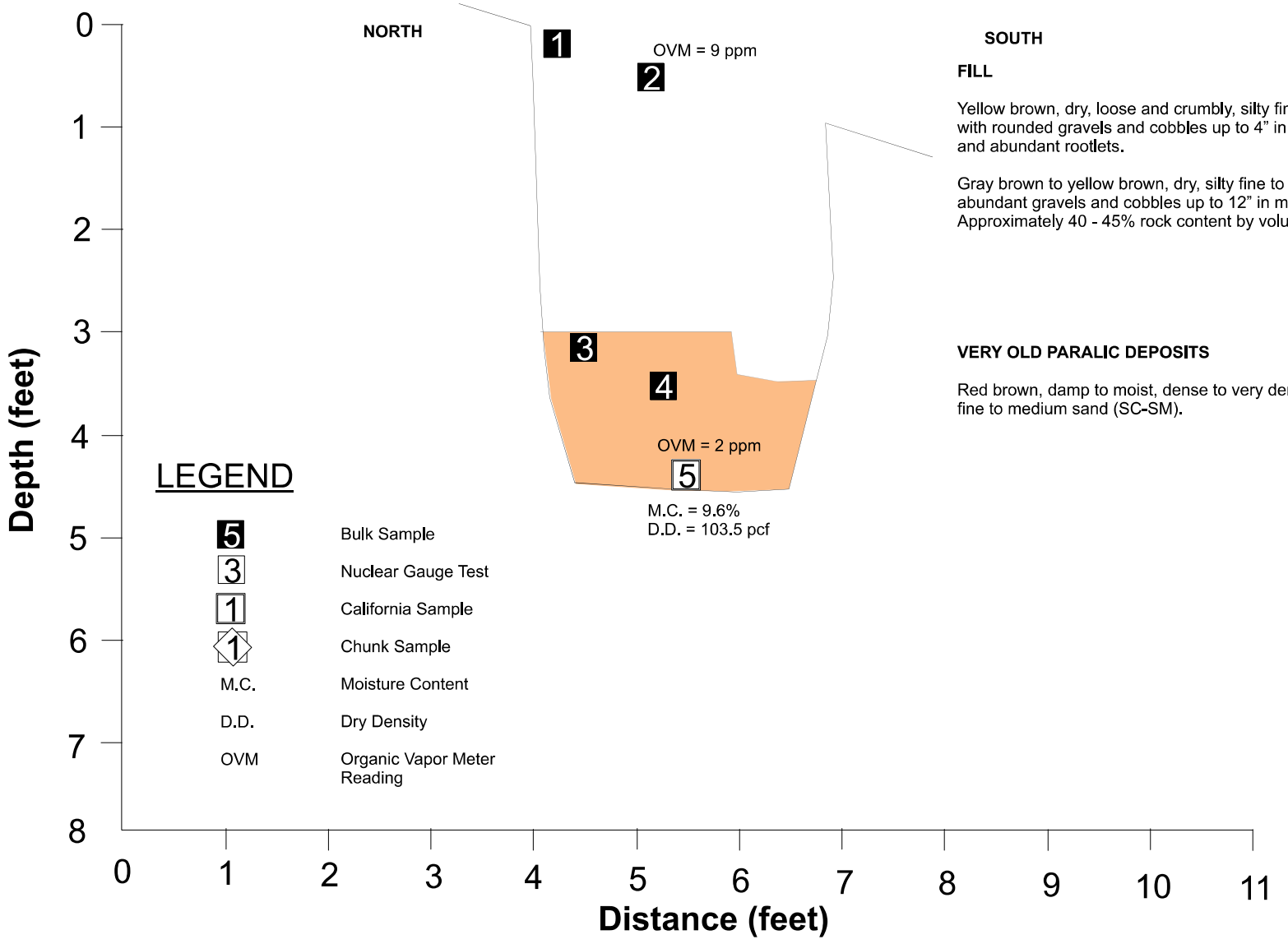
Seepage observed at the contact between the colluvium and Stadium Conglomerate.
Yellow brown, moist to wet, dense silty sand (SM) with gravels, cobbles and boulders up to 17" in maximum dimension and some rootlets.

LEGEND

5	Bulk Sample
3	Nuclear Gauge Test
1	California Sample
1	Chunk Sample
M.C.	Moisture Content
D.D.	Dry Density
OVM	Organic Vapor Meter Reading

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

SERRA MESA STORM DRAIN PROJECT	**TEST PIT LOG OF TP-1B**	
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-2**



FILL

Yellow brown, dry, loose and crumbly, silty fine to medium sand (SM) with rounded gravels and cobbles up to 4" in maximum dimension and abundant rootlets.

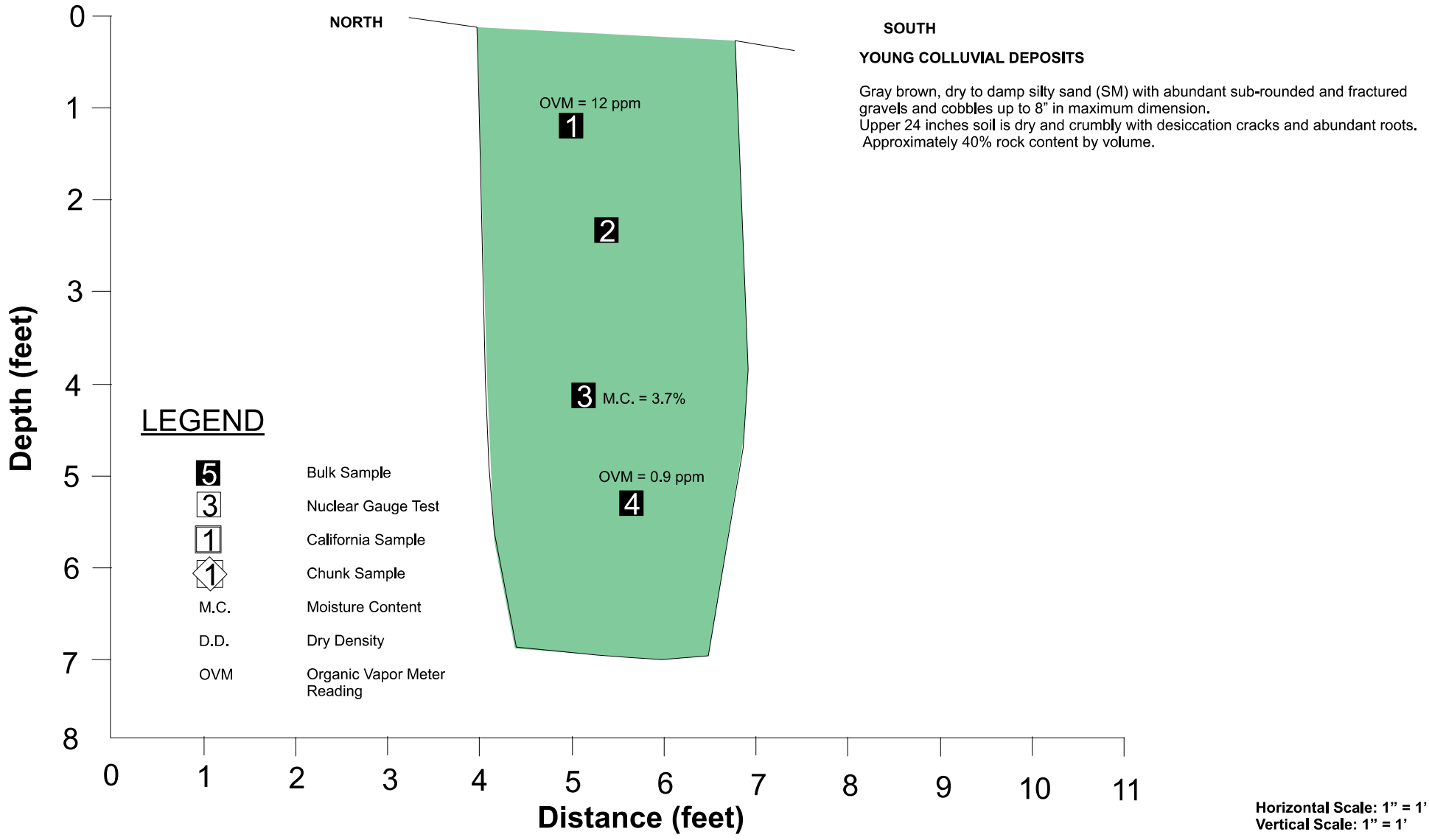
Gray brown to yellow brown, dry, silty fine to medium sand (SM) with abundant gravels and cobbles up to 12" in maximum dimension. Approximately 40 - 45% rock content by volume.

VERY OLD PARALIC DEPOSITS

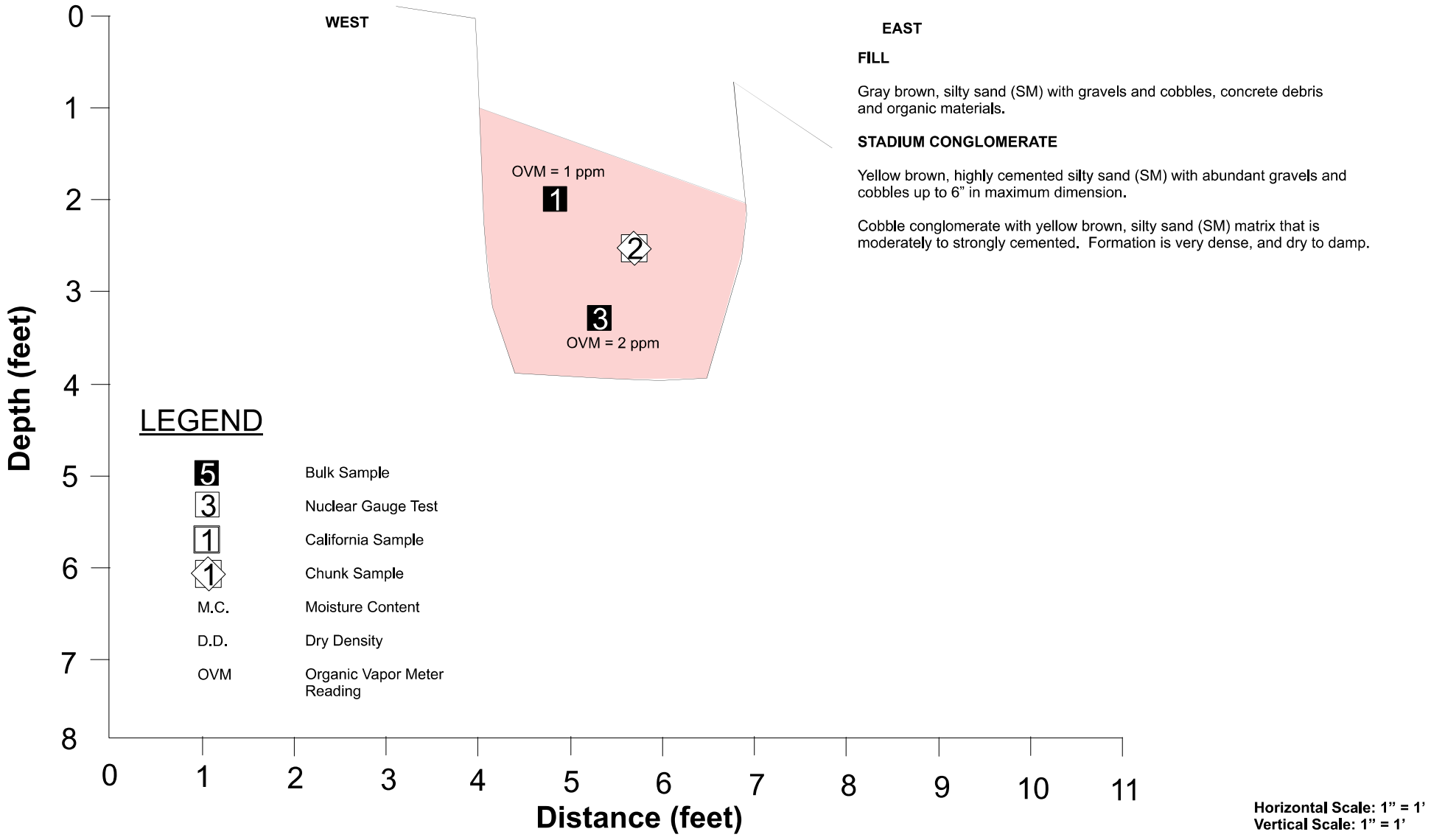
Red brown, damp to moist, dense to very dense silty to slightly clayey fine to medium sand (SC-SM).

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

SERRA MESA STORM DRAIN PROJECT	**TEST PIT LOG OF TP-2A**	
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-3**



SERRA MESA STORM DRAIN PROJECT	TEST PIT LOG OF TP-2B
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.
	FIGURE A-4



FILL

Gray brown, silty sand (SM) with gravels and cobbles, concrete debris and organic materials.

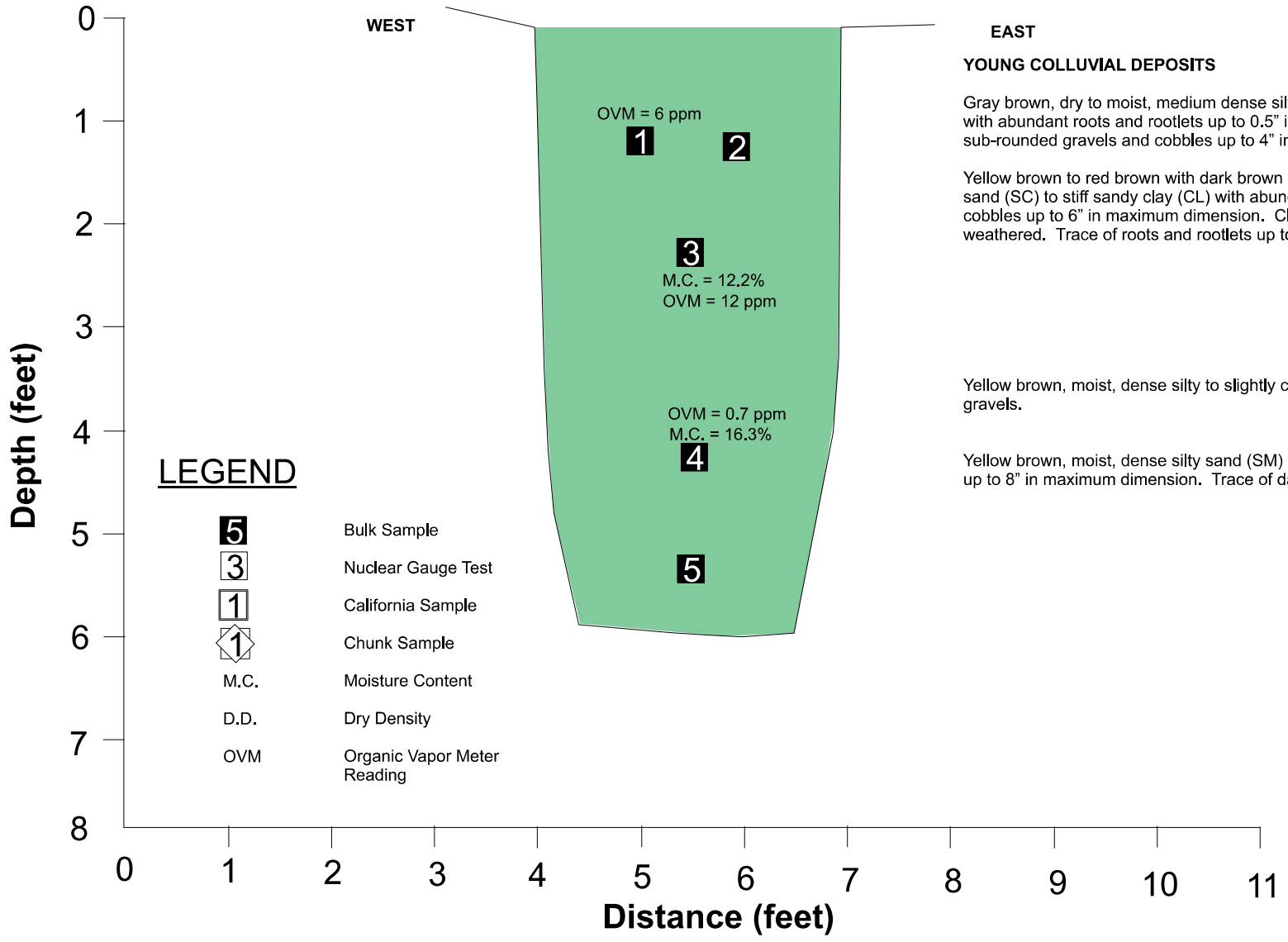
STADIUM CONGLOMERATE

Yellow brown, highly cemented silty sand (SM) with abundant gravels and cobbles up to 6" in maximum dimension.

Cobble conglomerate with yellow brown, silty sand (SM) matrix that is moderately to strongly cemented. Formation is very dense, and dry to damp.

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

SERRA MESA STORM DRAIN PROJECT	**TEST PIT LOG OF TP-3A**
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.** — **FIGURE A-5**



YOUNG COLLUVIAL DEPOSITS

Gray brown, dry to moist, medium dense silty to clayey sand (SC-SM) with abundant roots and rootlets up to 0.5" in diameter, and abundant sub-rounded gravels and cobbles up to 4" in maximum dimension.

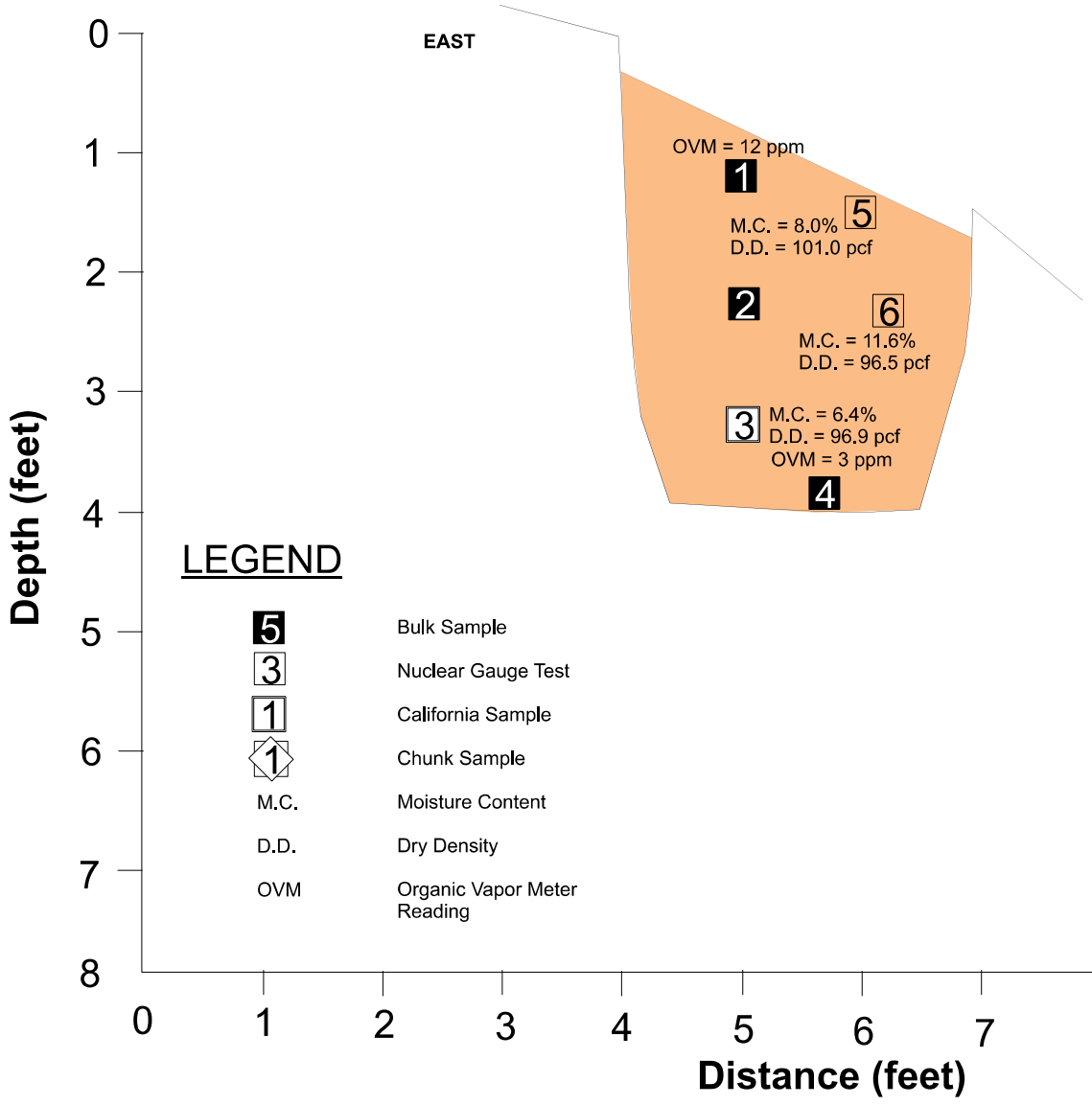
Yellow brown to red brown with dark brown mottling, moist, dense clayey sand (SC) to stiff sandy clay (CL) with abundant sub-rounded gravels and cobbles up to 6" in maximum dimension. Clasts are slightly to heavily weathered. Trace of roots and rootlets up to 0.5" in diameter were observed.

Yellow brown, moist, dense silty to slightly clayey sand (SC-SM) with gravels.

Yellow brown, moist, dense silty sand (SM) with abundant gravels and cobbles up to 8" in maximum dimension. Trace of dark brown mottling was observed.

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

SERRA MESA STORM DRAIN PROJECT	**TEST PIT LOG OF TP-3B**	
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-6**



TOPSOIL

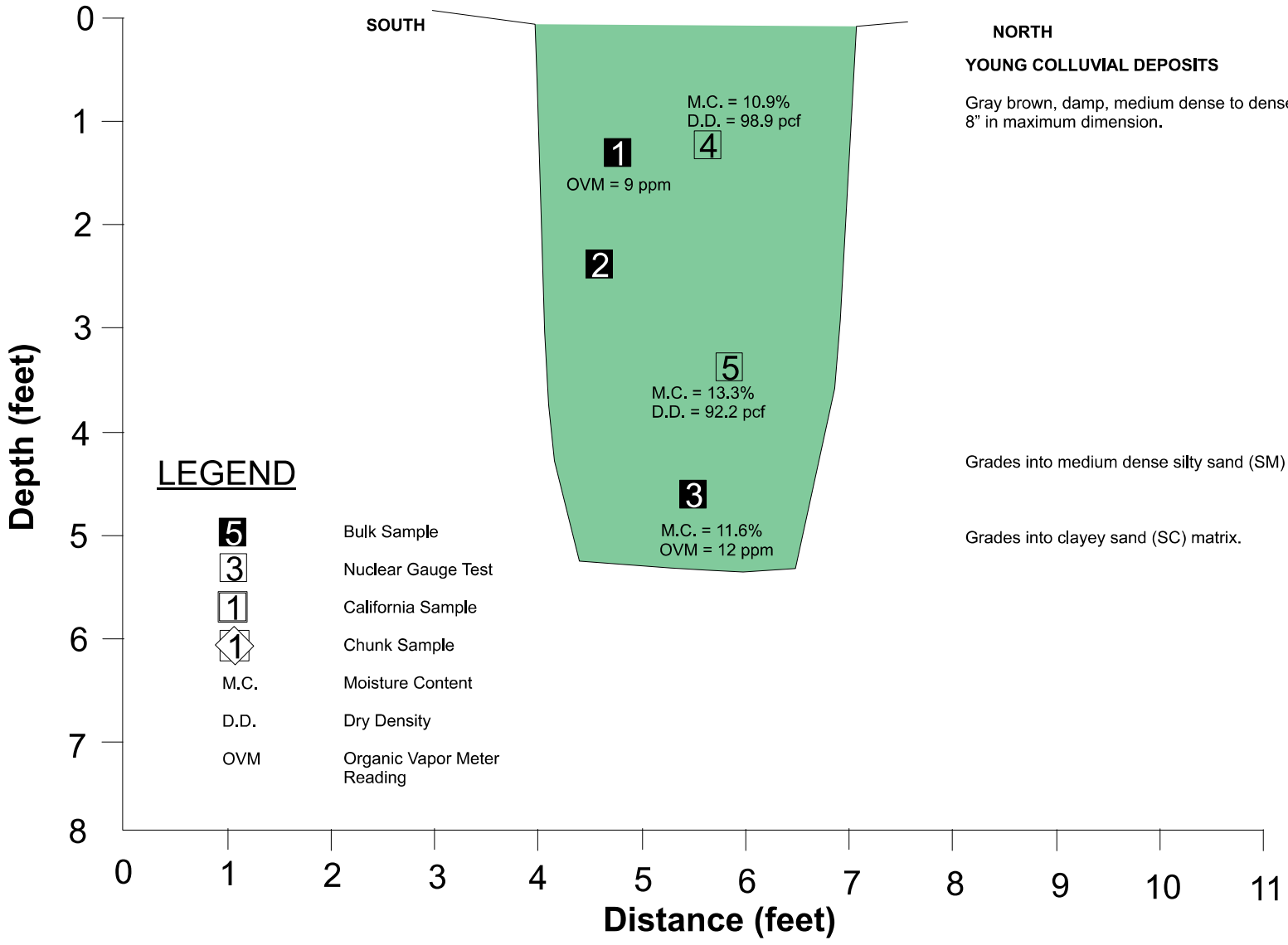
Dark brown, organic-rich material with abundant roots and rootlets in the upper 5" to 6".

VERY OLD PARALIC DEPOSITS

Reddish brown to yellow brown, damp, medium dense to dense silty sand (SM) with gravels and occasional cobbles up to 4" in maximum dimension.

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

SERRA MESA STORM DRAIN PROJECT	**TEST PIT LOG OF TP-4A**	
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-7**



YOUNG COLLUVIAL DEPOSITS

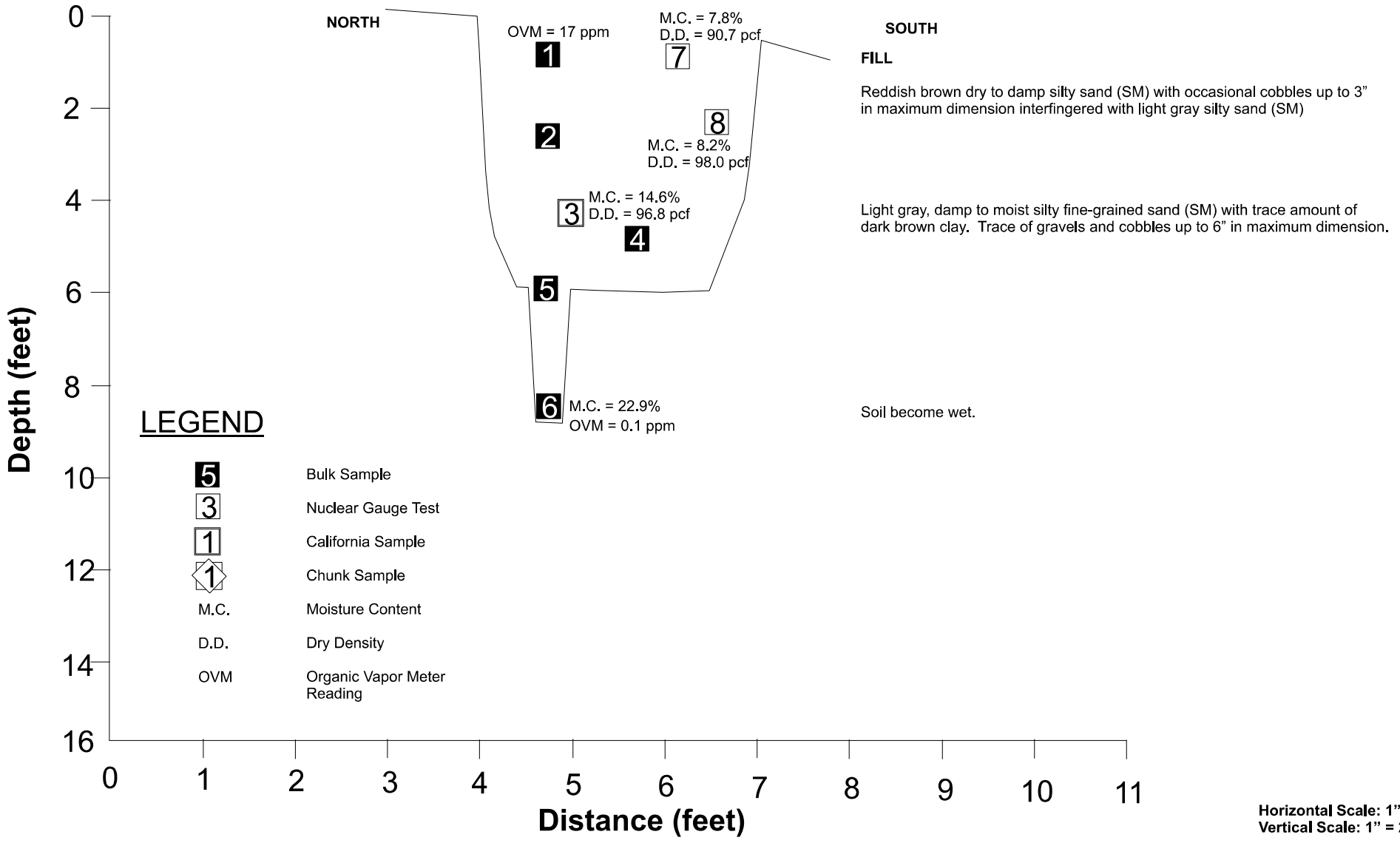
Gray brown, damp, medium dense to dense clayey sand (SC) with cobbles up to 8" in maximum dimension.

Grades into medium dense silty sand (SM) with gravels and abundant cobbles.

Grades into clayey sand (SC) matrix.

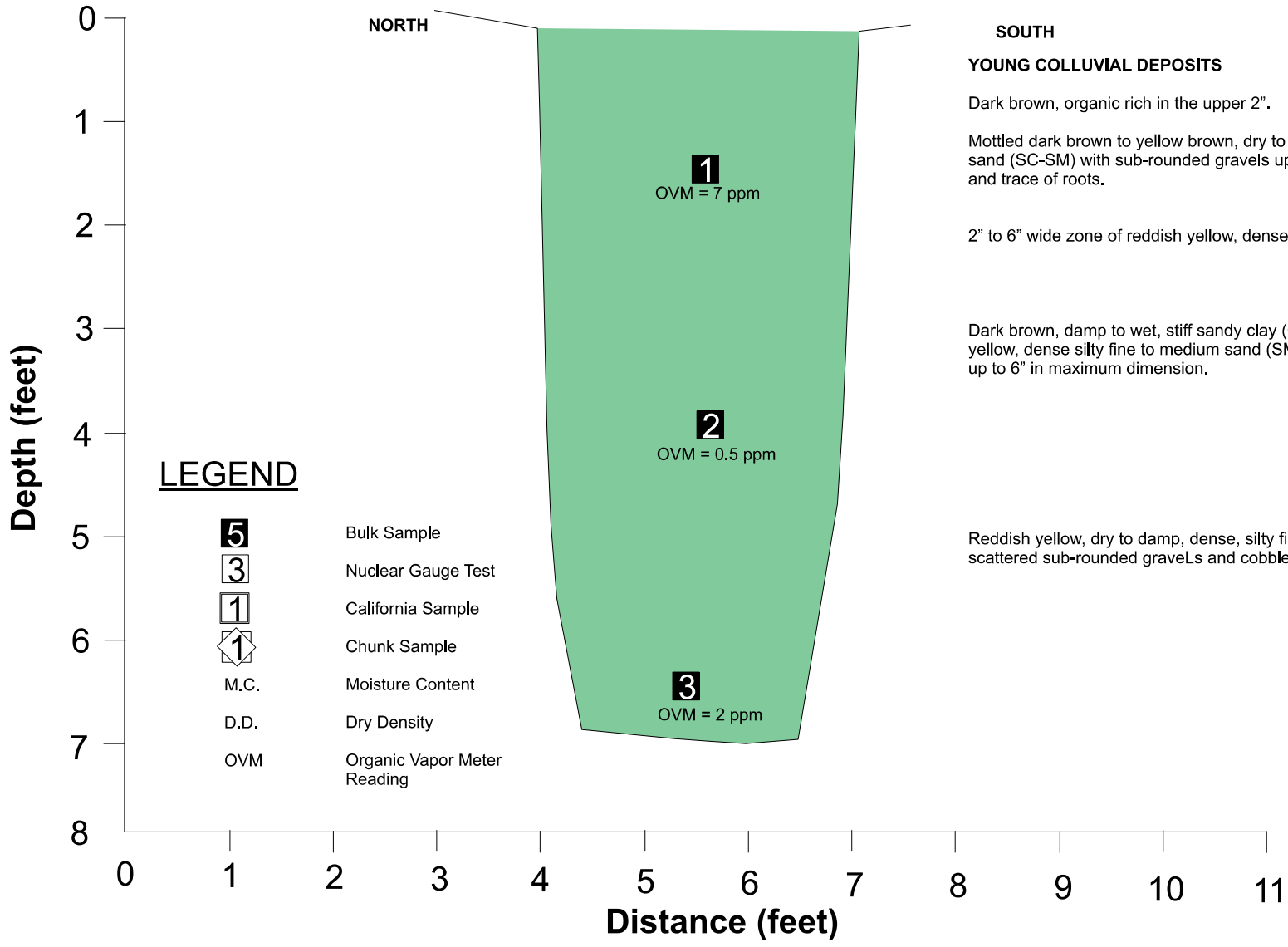
Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

SERRA MESA STORM DRAIN PROJECT	**TEST PIT LOG OF TP-4B**	
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-8**



Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 2'

SERRA MESA STORM DRAIN PROJECT	TEST PIT LOG OF TP-5A
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.
	FIGURE A-9



YOUNG COLLUVIAL DEPOSITS

Dark brown, organic rich in the upper 2".

Mottled dark brown to yellow brown, dry to moist, clayey sand to silty sand (SC-SM) with sub-rounded gravels up to 3" in maximum dimension and trace of roots.

2" to 6" wide zone of reddish yellow, dense, silty fine to medium sand (SM)

Dark brown, damp to wet, stiff sandy clay (CL) interfingered with reddish yellow, dense silty fine to medium sand (SM) with trace of sub-rounded gravels up to 6" in maximum dimension.

Reddish yellow, dry to damp, dense, silty fine to medium sand (SM) with scattered sub-rounded graveLs and cobbles up to 10" in maximum dimension.

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

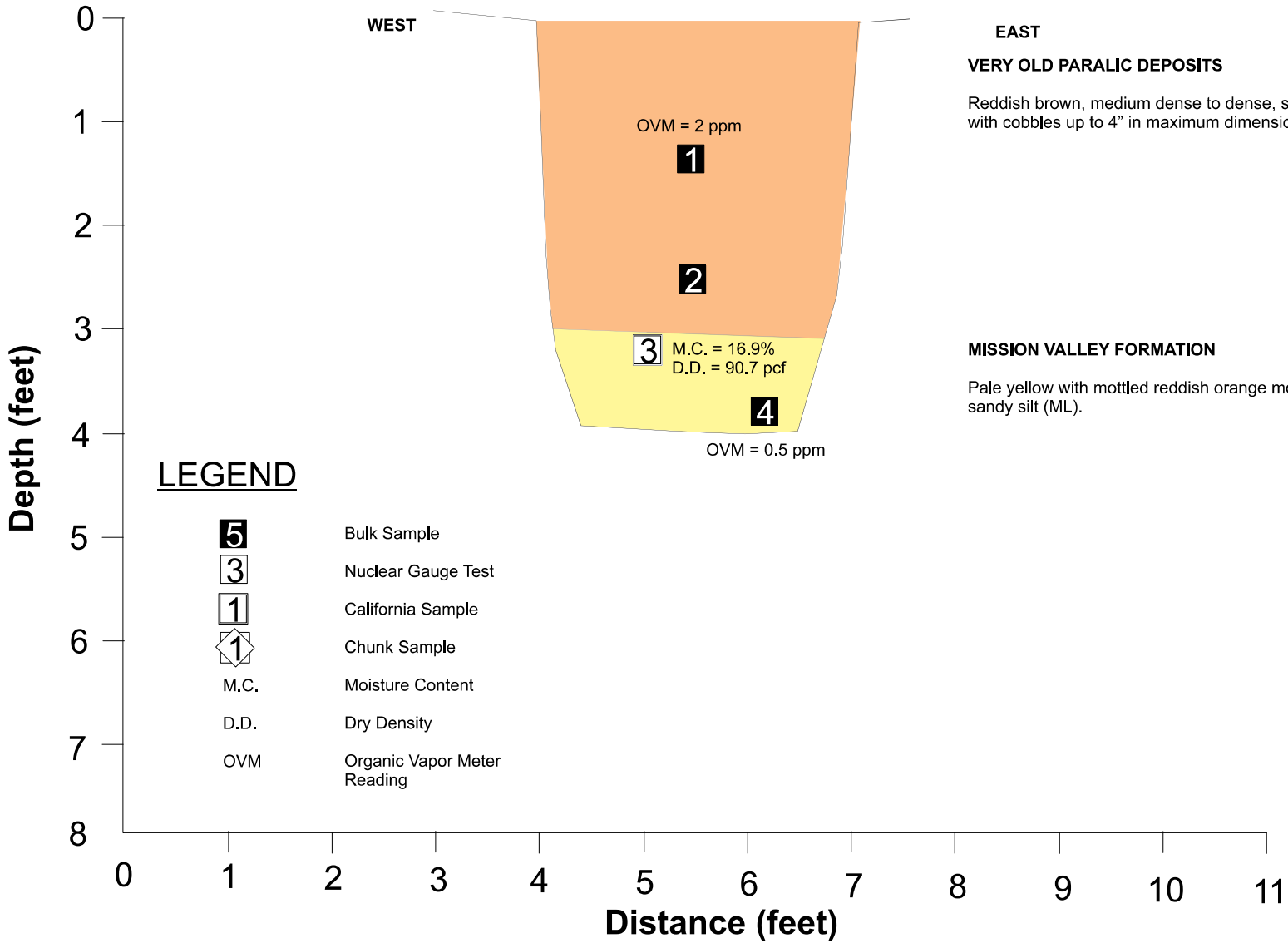
SERRA MESA STORM DRAIN PROJECT

TEST PIT LOG OF TP-5B

PROJECT NO. 178 GS-16-C

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE A-10



VERY OLD PARALIC DEPOSITS

Reddish brown, medium dense to dense, silty to clayey sand (SC-SM) with cobbles up to 4" in maximum dimension.

MISSION VALLEY FORMATION

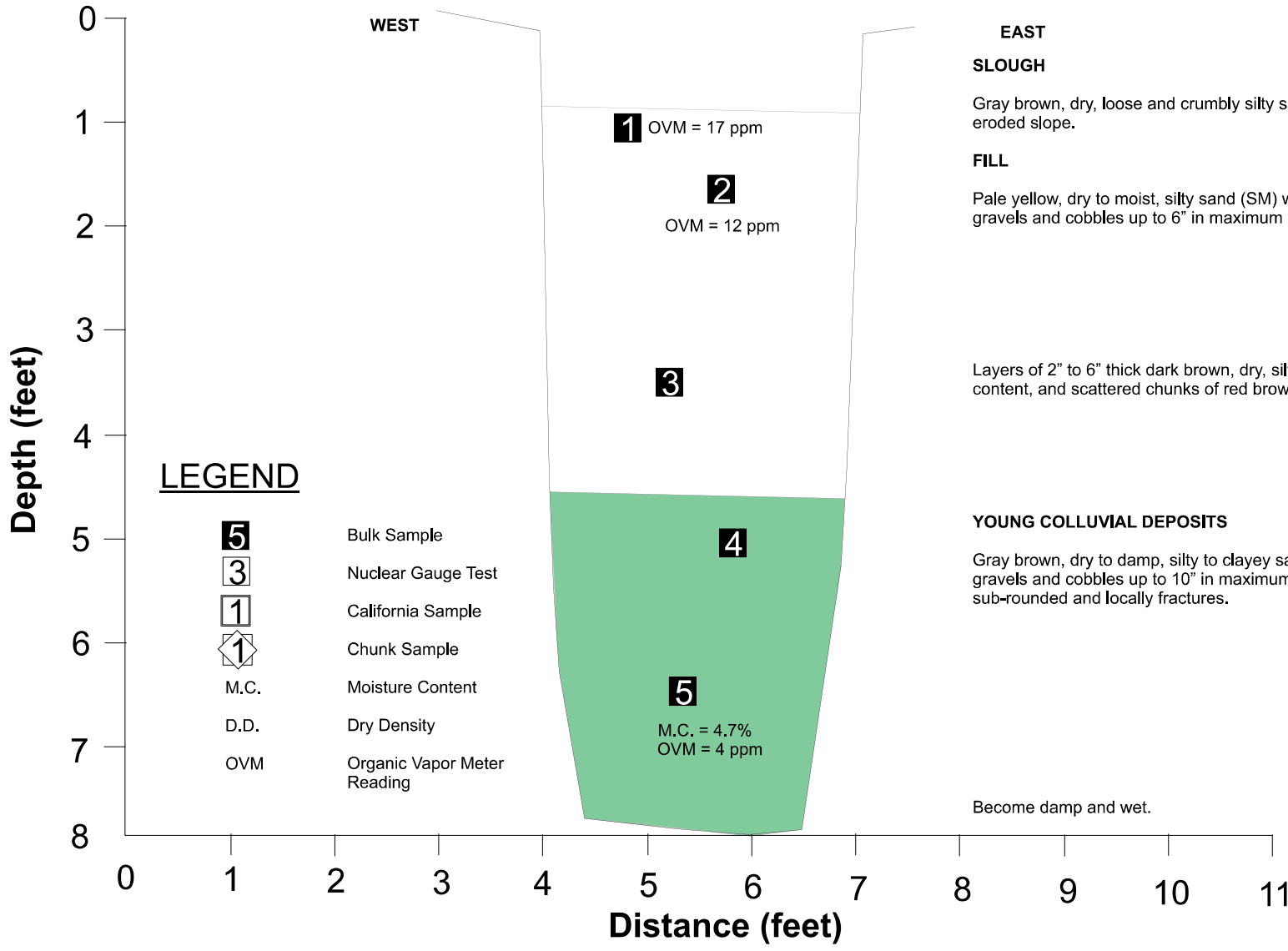
Pale yellow with mottled reddish orange mottling, moist to wet, stiff fine sandy silt (ML).

LEGEND

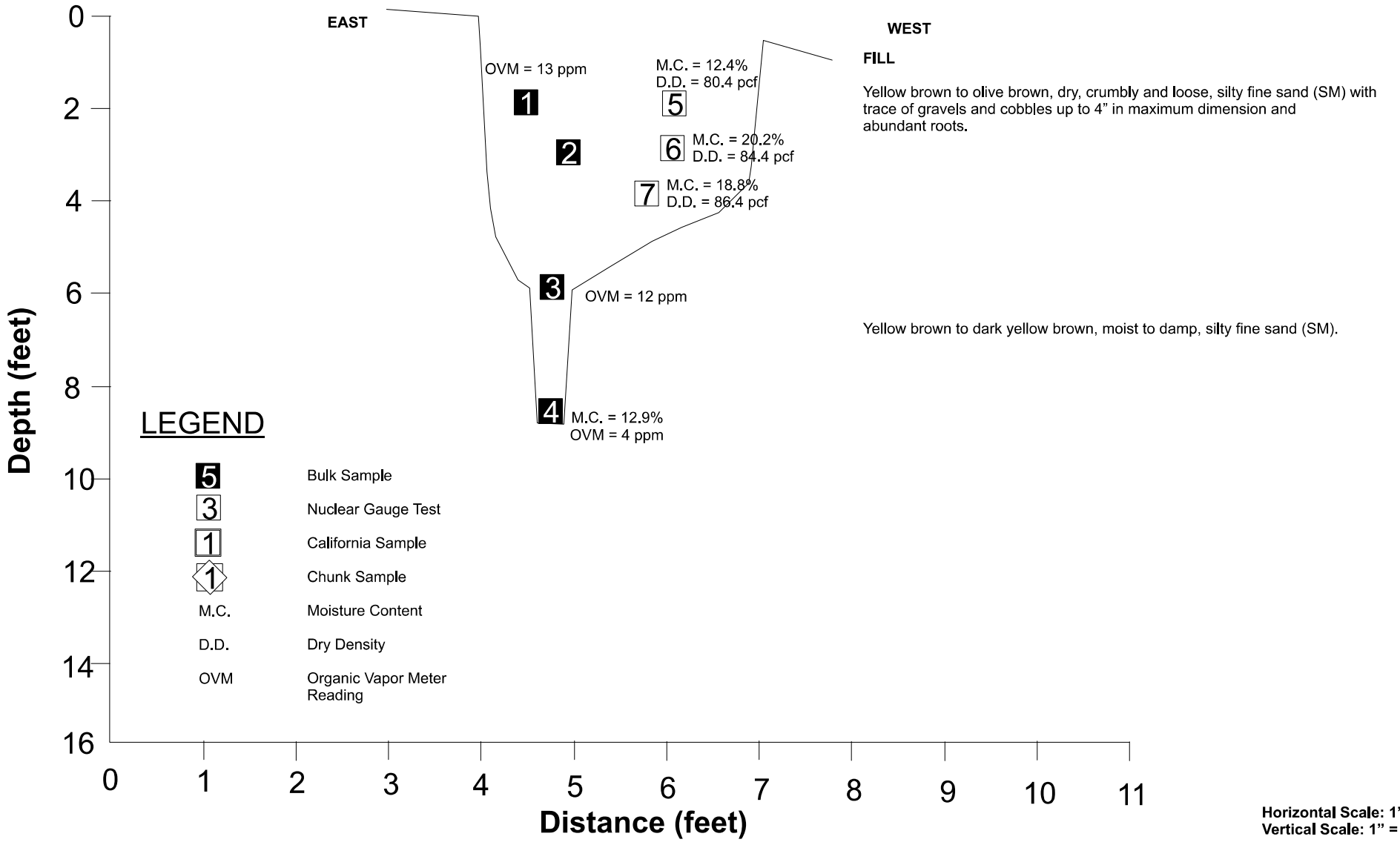
5	Bulk Sample
3	Nuclear Gauge Test
1	California Sample
1	Chunk Sample
M.C.	Moisture Content
D.D.	Dry Density
OVM	Organic Vapor Meter Reading

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

SERRA MESA STORM DRAIN PROJECT	TEST PIT LOG OF TP-6A	
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-11



SERRA MESA STORM DRAIN PROJECT	TEST PIT LOG OF TP-6B	
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-12



FILL

Yellow brown to olive brown, dry, crumbly and loose, silty fine sand (SM) with trace of gravels and cobbles up to 4" in maximum dimension and abundant roots.

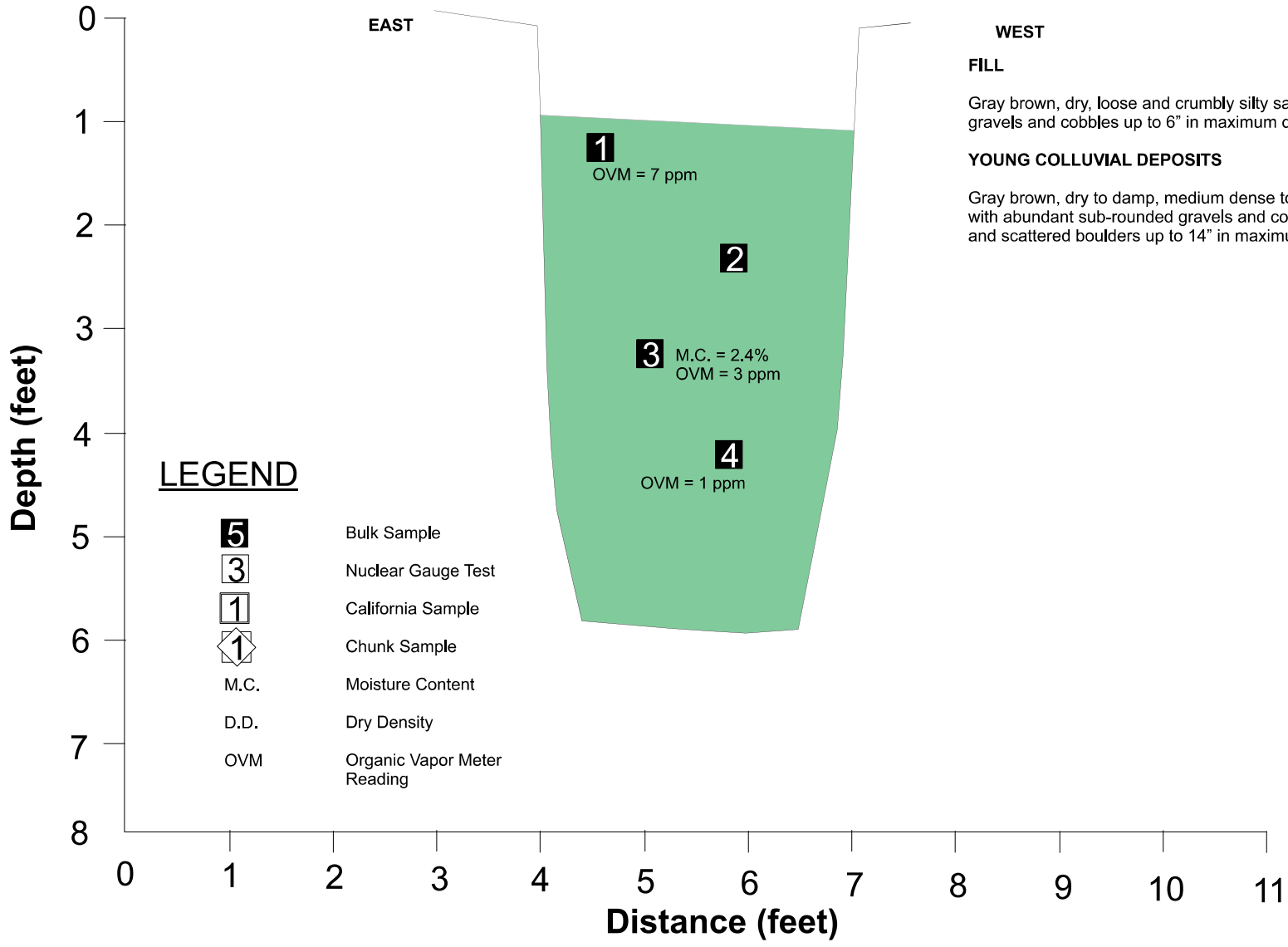
Yellow brown to dark yellow brown, moist to damp, silty fine sand (SM).

LEGEND

5	Bulk Sample
3	Nuclear Gauge Test
1	California Sample
1	Chunk Sample
M.C.	Moisture Content
D.D.	Dry Density
OVM	Organic Vapor Meter Reading

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 2'

SERRA MESA STORM DRAIN PROJECT	TEST PIT LOG OF TP-7A	
PROJECT NO. 178 GS-16-C	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-13



FILL

Gray brown, dry, loose and crumbly silty sand (SM) with abundant sub-rounded gravels and cobbles up to 6" in maximum dimension and appreciable roots.

YOUNG COLLUVIAL DEPOSITS

Gray brown, dry to damp, medium dense to dense silty to clayey sand (SM-SC) with abundant sub-rounded gravels and cobbles up to 8" in maximum dimension, and scattered boulders up to 14" in maximum dimension.

Horizontal Scale: 1" = 1'
Vertical Scale: 1" = 1'

SERRA MESA STORM DRAIN PROJECT	**TEST PIT LOG OF TP-7B**	
PROJECT NO. 178 GS-16-C	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE A-14**

APPENDIX B

LABORATORY TESTING

APPENDIX B

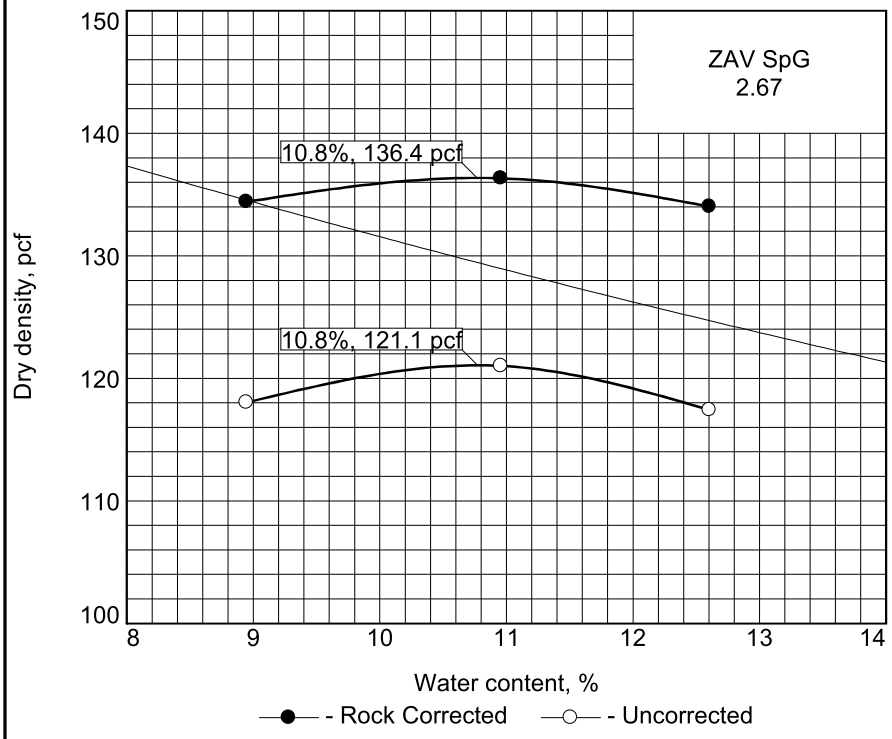
LABORATORY TESTING

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-3;
- Sieve and hydrometer analyses (ASTM D422), and the final test results are plotted as gradation curves on Figures B-4 through B-8; and
- Direct shear test (ASTM D3080). The test results are presented on Figures B-9 through B-14.

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 TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-91 Procedure A Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Preparation Method Wet
Hammer Wt. 10 lb.
Hammer Drop 18 in.
Number of Layers five
Blows per Layer 25
Mold Size 0.03333 cu. ft.

Test Performed on Material Passing #4 **Sieve**

NM **LL** **PI**
Sp.G. (ASTM D 854) 2.67
%>#4 50.0 **%<No.200** 10.3
USCS SP-SC **AASHTO**
Date Sampled
Date Tested 10/18/2019
Tested By B. Hayes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5976.0	6062.0	6031.0			
WM	4032.0	4032.0	4032.0			
WW + T #1	670.6	395.5	399.0			
WD + T #1	625.7	363.2	360.8			
TARE #1	123.8	68.4	57.7			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	8.9	11.0	12.6			
DRY DENSITY	134.4	136.3	134.0			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 136.4 pcf	121.1 pcf
Optimum moisture = 10.8 %	10.8 %

Material Description
Gray brown, clayey sand with gravel and cobble.

Remarks:

Project No. 178 GS-16-C **Client:** RICK ENGINEERING COMPANY
Project: SERRA MESA STORM DRAINS PROJECT

○ **Source of Sample:** TP1-B **Depth:** 3' **Sample Number:** 2

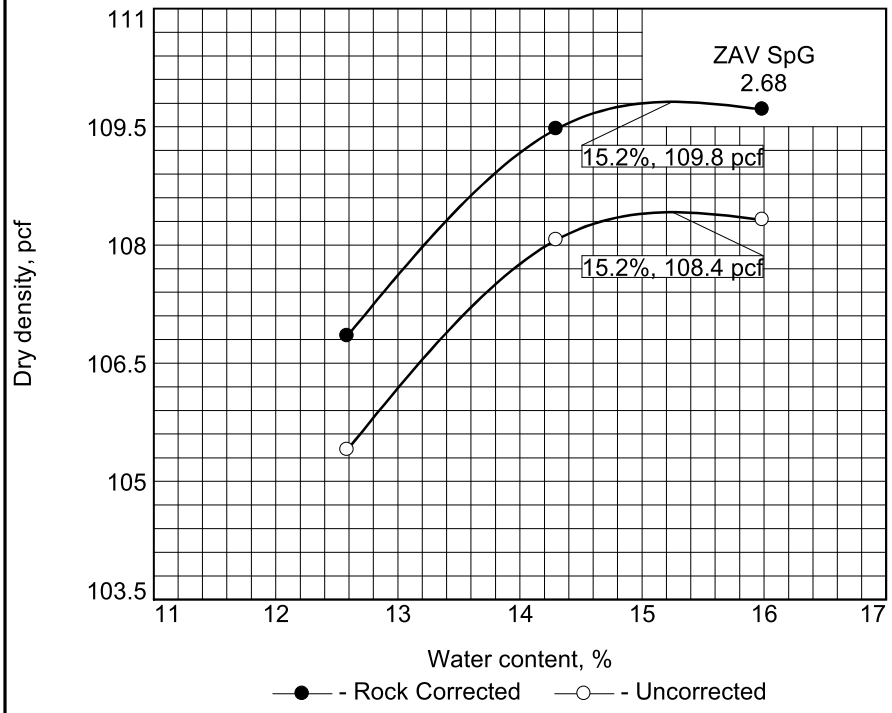
Checked by: Sani Sutanto
Title: Senior Project Manager

Allied Geotechnical Engineers, Inc.

Santee, CA

Figure B-1

COMPACTION TEST REPORT



Test Specification:
ASTM D 1557-91 Procedure C Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Preparation Method Wet
Hammer Wt. 10 lb.
Hammer Drop 18 in.
Number of Layers five
Blows per Layer 56
Mold Size 0.075 cu. ft.

Test Performed on Material Passing 3/4 in. **Sieve**

NM _____ **LL** _____ **PI** _____
Sp.G. (ASTM D 854) 2.68
%>3/4 in. 4.0 **%<No.200** _____
USCS SC-SM **AASHTO** _____
Date Sampled _____
Date Tested 10/21/2019
Tested By B. Hayes

TESTING DATA

	1	2	3	4	5	6
WM + WS	10637.0	10802.0	10874.0			
WM	6600.0	6600.0	6600.0			
WW + T #1	476.5	620.8	461.0			
WD + T #1	431.1	560.0	406.5			
TARE #1	70.4	134.8	65.6			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	12.6	14.3	16.0			
DRY DENSITY	106.8	109.5	109.7			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 109.8 pcf	108.4 pcf
Optimum moisture = 15.2 %	15.2 %

Material Description
Gray brown, silty to clayey sand with gravel and cobbles.

Remarks:

Project No. 178 GS-16-C **Client:** RICK ENGINEERING COMPANY
Project: SERRA MESA STORM DRAINS PROJECT

○ **Source of Sample:** TP3-B **Depth:** 1 **Sample Number:** 2

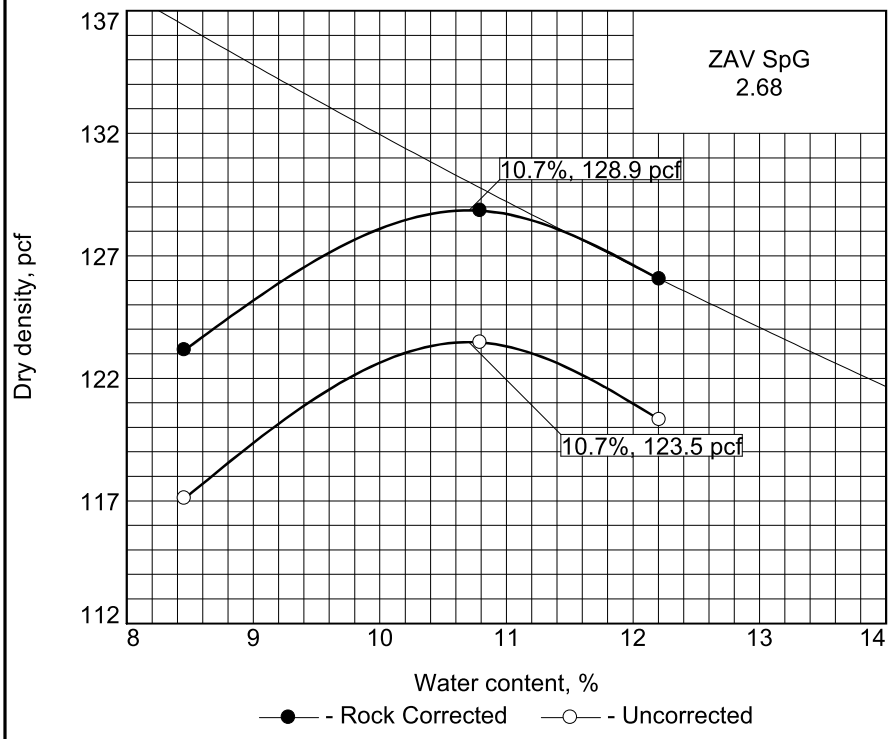
Checked by: Sani Sutanto
Title: Senior Project Manager

Allied Geotechnical Engineers, Inc.

Santee, CA

Figure B-2

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-91 Procedure A Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Preparation Method Wet
Hammer Wt. 10 lb.
Hammer Drop 18 in.
Number of Layers five
Blows per Layer 25
Mold Size 0.03333 cu. ft.

Test Performed on Material Passing #4 **Sieve**

NM **LL** **PI**
Sp.G. (ASTM D 854) 2.68
%>#4 18.6 **%<No.200** 29.8
USCS SC **AASHTO**
Date Sampled
Date Tested 10/17/2019
Tested By B. Hayes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5952.0	6100.0	6073.0			
WM	4032.0	4032.0	4032.0			
WW + T #1	650.6	530.0	426.9			
WD + T #1	609.3	498.7	387.9			
TARE #1	121.0	208.7	68.5			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	8.5	10.8	12.2			
DRY DENSITY	123.2	128.8	126.0			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 128.9 pcf	123.5 pcf
Optimum moisture = 10.7 %	10.7 %

Material Description
Gray brown, clayey sand with gravel

Remarks:

Project No. 178 GS-16-C **Client:** RICK ENGINEERING COMPANY
Project: SERRA MESA STORM DRAINS PROJECT

○ **Source of Sample:** TP4-B **Depth:** 2 **Sample Number:** 2

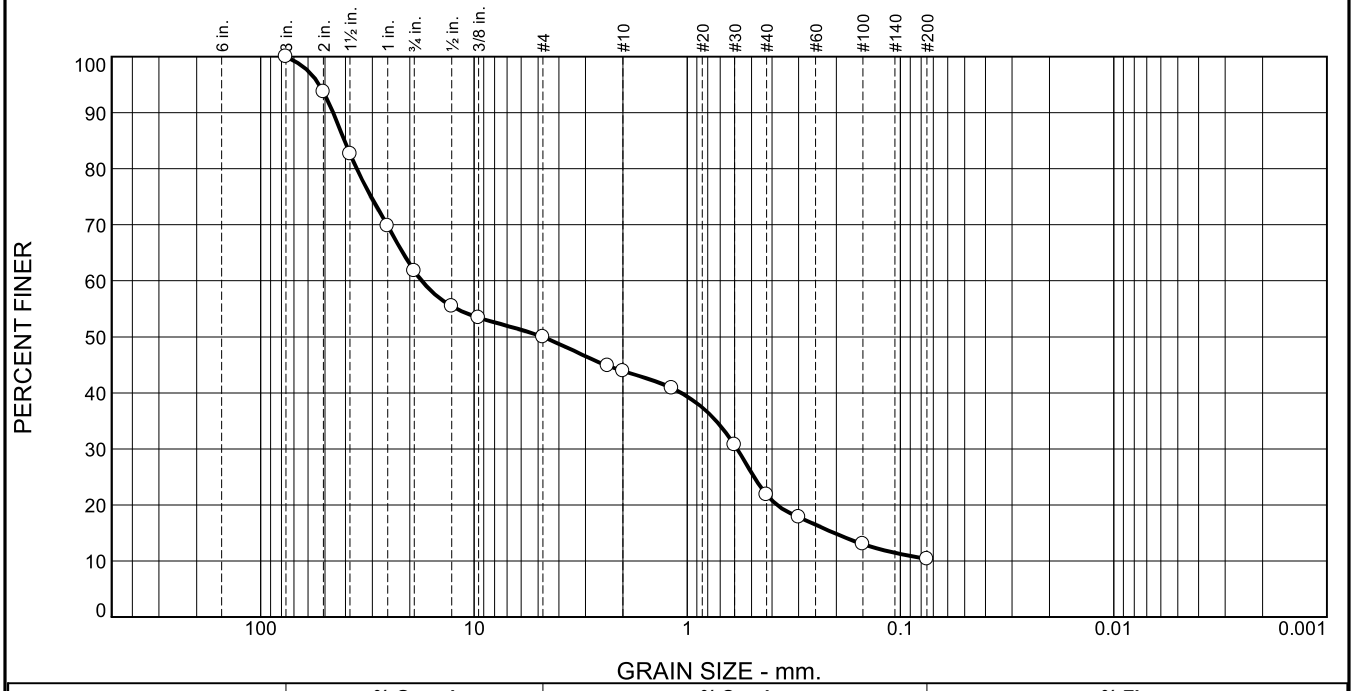
Checked by: Sani Sutanto
Title: Senior Project Manager

Allied Geotechnical Engineers, Inc.

Santee, CA

Figure B-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	38.2	11.8	6.1	22.1	11.5	10.3	

Test Results (ASTM D 422 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3	100.0		
2	93.7		
1.5	82.6		
1	69.8		
.75	61.8		
0.5	55.4		
0.375	53.4		
#4	50.0		
#8	44.8		
#10	43.9		
#16	40.8		
#30	30.7		
#40	21.8		
#50	17.8		
#100	13.0		
#200	10.3		

* (no specification provided)

Material Description

Gray brown, clayey sand with gravel and cobble.

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SP-SC AASHTO (M 145)=

Coefficients

D_{90}= 45.7188 D_{85}= 40.4198 D_{60}= 17.5610
D_{50}= 4.7742 D_{30}= 0.5837 D_{15}= 0.2048
D_{10}= C_u= C_c=

Remarks

Date Received: **Date Tested:** 10/21/2019

Tested By: B. Hayes

Checked By: Sani Sutanto

Title: Senior Project Manager

Source of Sample: TP1-B **Depth:** 3'
Sample Number: 2

Date Sampled:

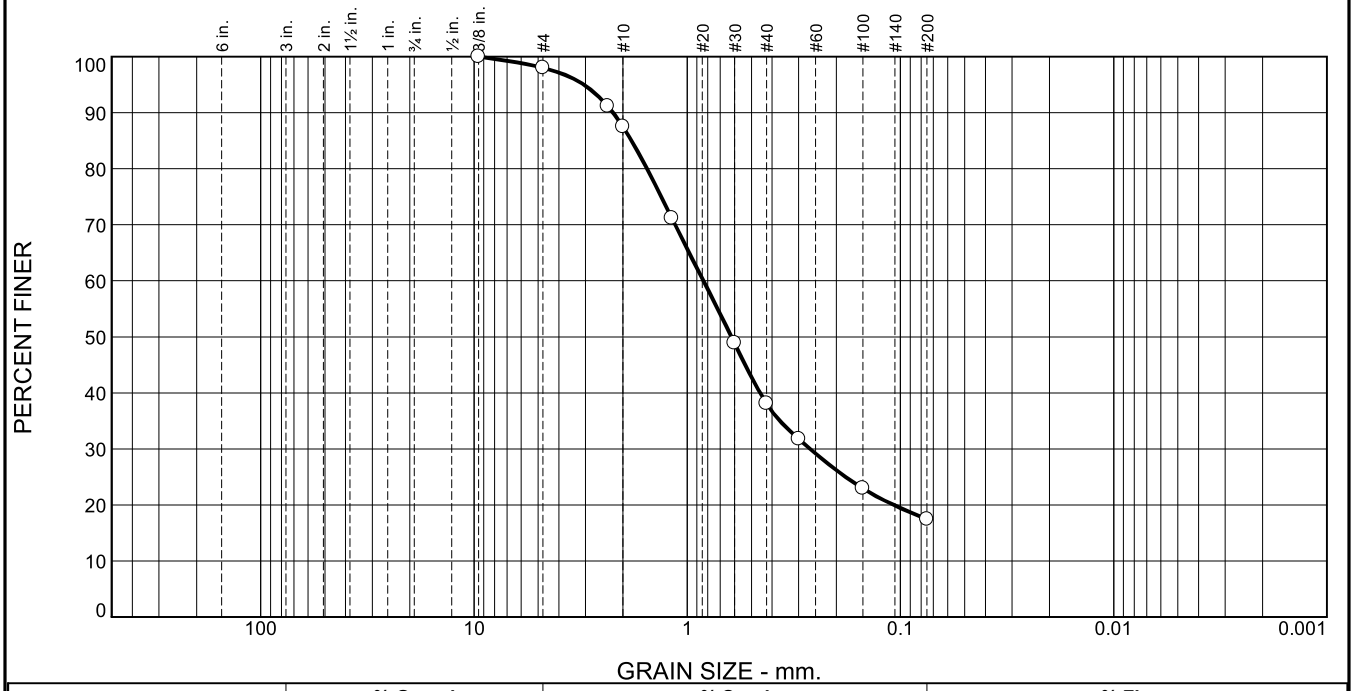
Allied Geotechnical Engineers, Inc.

Santee, CA

Client: RICK ENGINEERING COMPANY
Project: SERRA MESA STORM DRAINS PROJECT

Project No: 178 GS-16-C **Figure** B-4

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.0	10.5	49.4	20.7	17.4	

Test Results (ASTM D 422 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375	100.0		
#4	98.0		
#8	91.2		
#10	87.5		
#16	71.2		
#30	48.9		
#40	38.1		
#50	31.8		
#100	22.9		
#200	17.4		

* (no specification provided)

Material Description

Reddish brown, silty sand with trace of fine gravel

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SM AASHTO (M 145)=

Coefficients

D_{90}= 2.2276 D_{85}= 1.8221 D_{60}= 0.8394
D_{50}= 0.6201 D_{30}= 0.2652 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: **Date Tested:** 10/21/2019

Tested By: B. Hayes

Checked By: Sani Sutanto

Title: Senior Project Manager

Source of Sample: TP4-A **Depth:** 3
Sample Number: 3

Date Sampled:

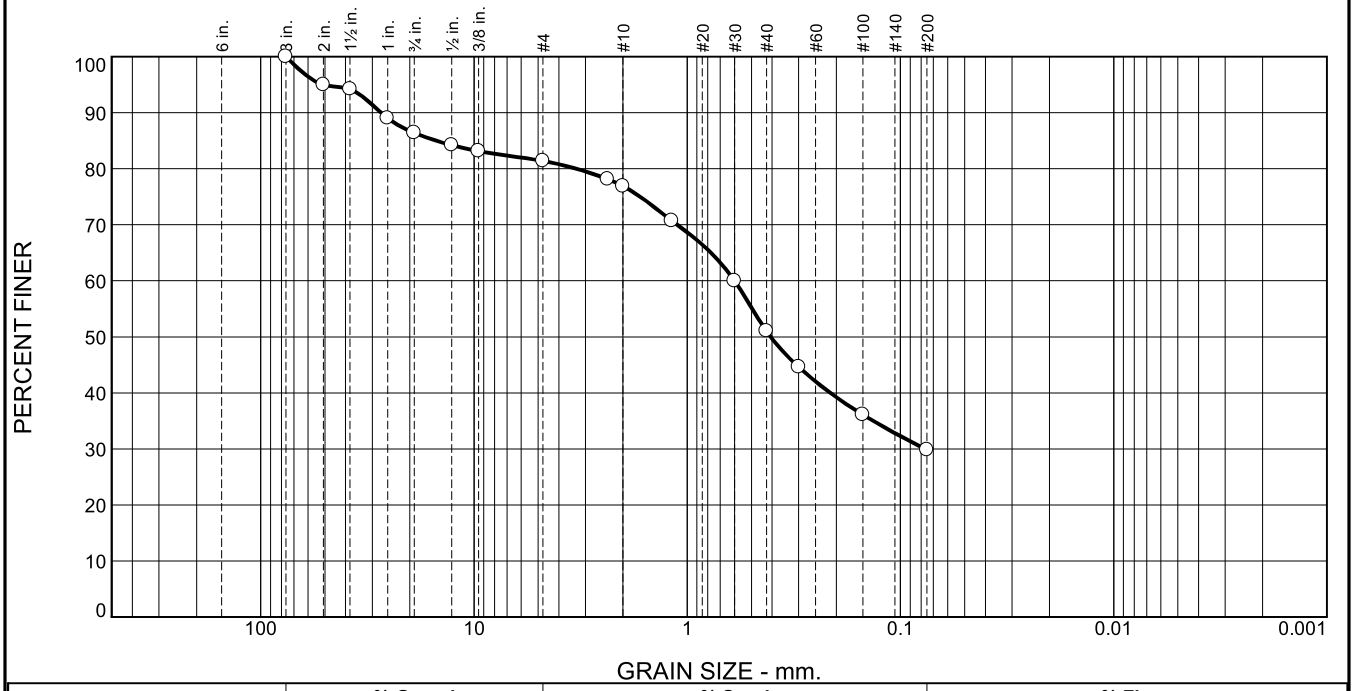
Allied Geotechnical Engineers, Inc.

Santee, CA

Client: RICK ENGINEERING COMPANY
Project: SERRA MESA STORM DRAINS PROJECT

Project No: 178 GS-16-C **Figure** B-5

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.6	5.0	4.5	25.9	21.2	29.8	

Test Results (ASTM D 422 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3	100.0		
2	95.0		
1.5	94.2		
1	89.0		
.75	86.4		
0.5	84.2		
0.375	83.2		
#4	81.4		
#8	78.1		
#10	76.9		
#16	70.7		
#30	60.0		
#40	51.0		
#50	44.6		
#100	36.1		
#200	29.8		

* (no specification provided)

Material Description

Gray brown, clayey sand with gravel

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SC AASHTO (M 145)=

Coefficients

D_{90}= 27.2641 D_{85}= 15.0148 D_{60}= 0.5999
D_{50}= 0.4063 D_{30}= 0.0766 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: **Date Tested:** 10/21/2019

Tested By: B. Hayes

Checked By: Sani Sutanto

Title: Senior Project Manager

Source of Sample: TP4-B **Depth:** 2
Sample Number: 2

Date Sampled:

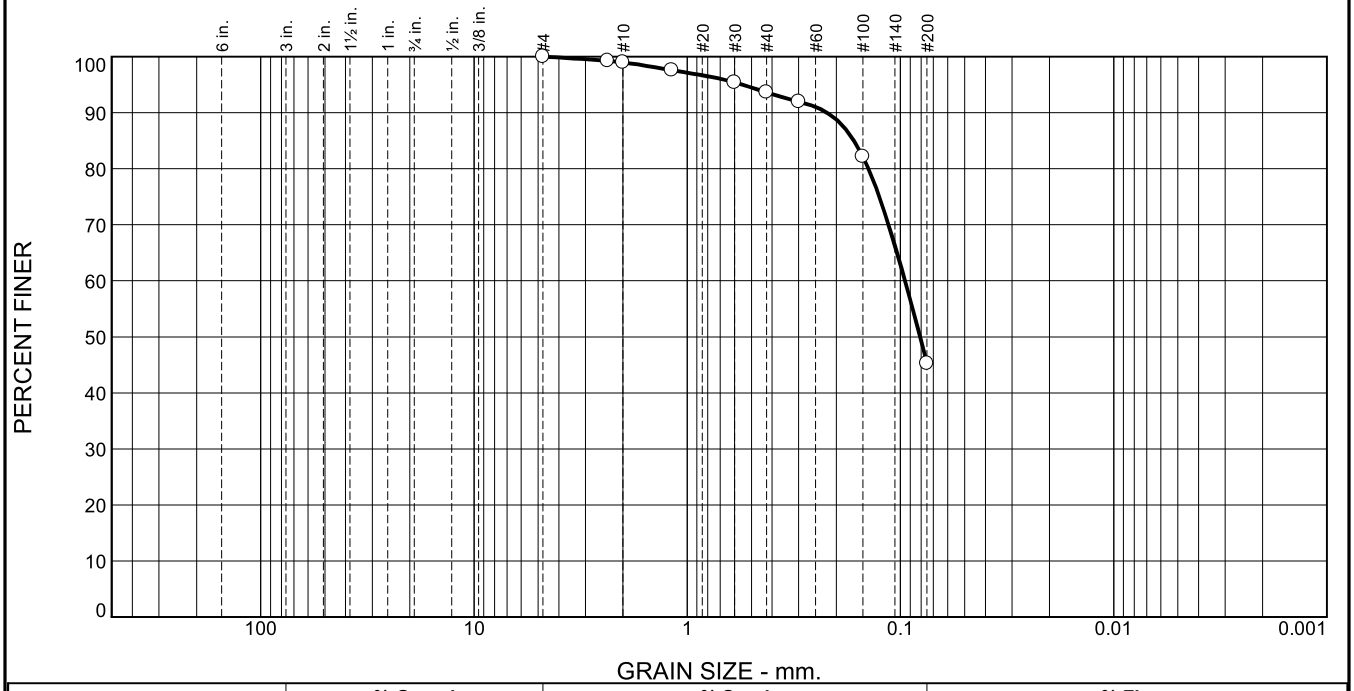
Allied Geotechnical Engineers, Inc.

Santee, CA

Client: RICK ENGINEERING COMPANY
Project: SERRA MESA STORM DRAINS PROJECT

Project No: 178 GS-16-C **Figure** B-6

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.0	5.4	48.3	45.3	

Test Results (ASTM D 422 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#8	99.2		
#10	99.0		
#16	97.6		
#30	95.4		
#40	93.6		
#50	92.0		
#100	82.1		
#200	45.3		

* (no specification provided)

Material Description

Light gray, fine silty sand.

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SM AASHTO (M 145)=

Coefficients

D_{90}= 0.2205 D_{85}= 0.1651 D_{60}= 0.0952
D_{50}= 0.0809 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: **Date Tested:** 10/21/2019

Tested By: B. Hayes

Checked By: Sani Sutanto

Title: Senior Project Manager

Source of Sample: TP5-A **Depth:** 4
Sample Number: 3

Date Sampled: 10/15/19

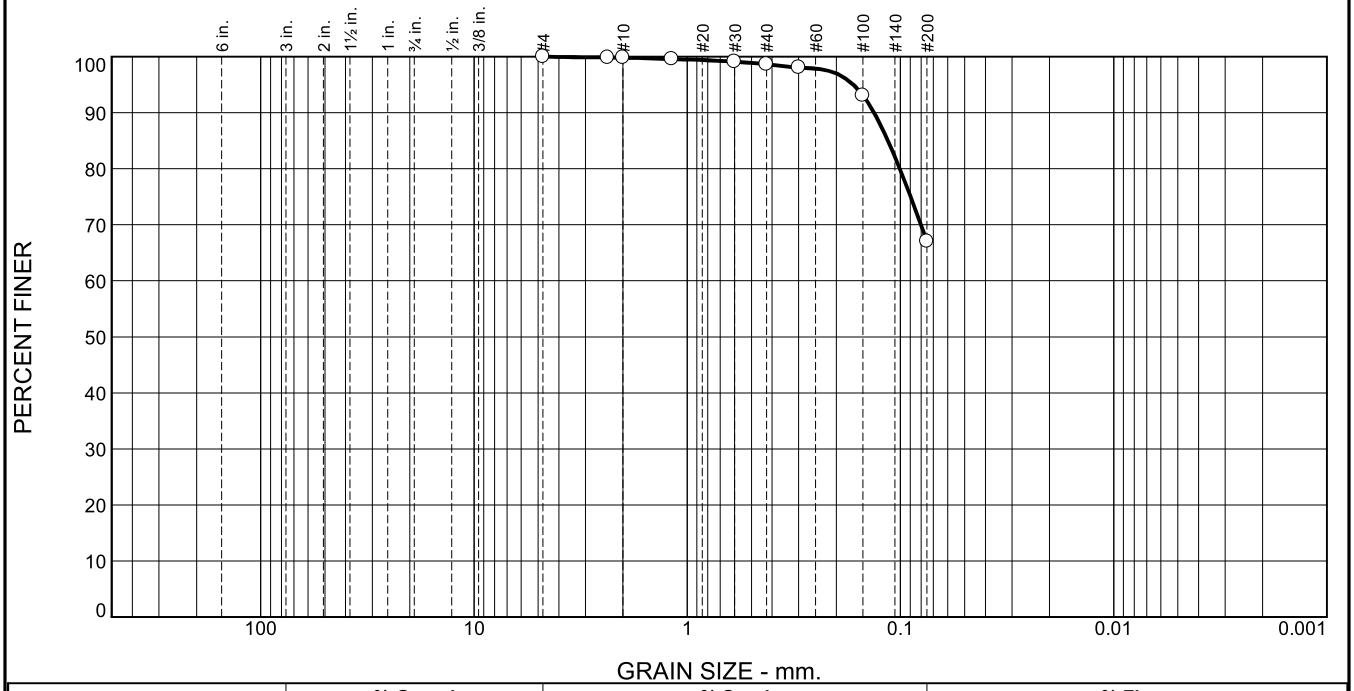
Allied Geotechnical Engineers, Inc.

Santee, CA

Client: RICK ENGINEERING COMPANY
Project: SERRA MESA STORM DRAINS PROJECT

Project No: 178 GS-16-C **Figure** B-7

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	1.2	31.6	67.0	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#8	99.8		
#10	99.8		
#16	99.6		
#30	99.1		
#40	98.6		
#50	98.0		
#100	93.1		
#200	67.0		

* (no specification provided)

Material Description

Pale yellow, fine-grained sandy silt.

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= ML AASHTO (M 145)=

Coefficients

D_{90}= 0.1331 D_{85}= 0.1145 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: **Date Tested:** 10/05/2019

Tested By: B. Hayes

Checked By: Sani Sutanto

Title: Senior Project Manager

Source of Sample: TP6-A **Depth:** 3
Sample Number: 3

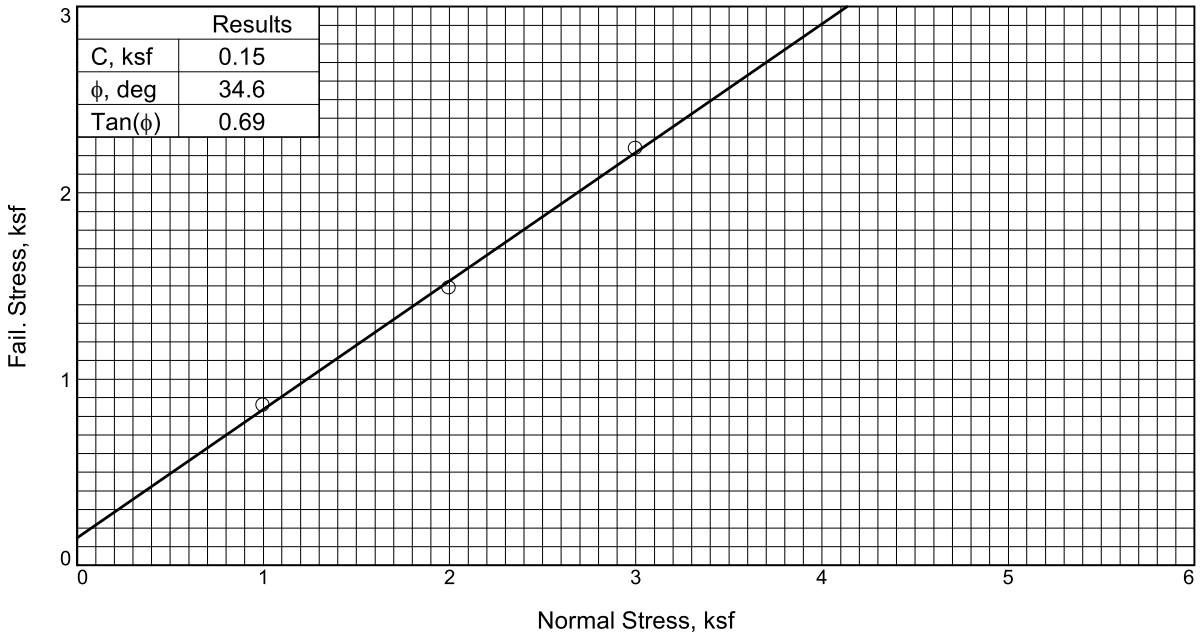
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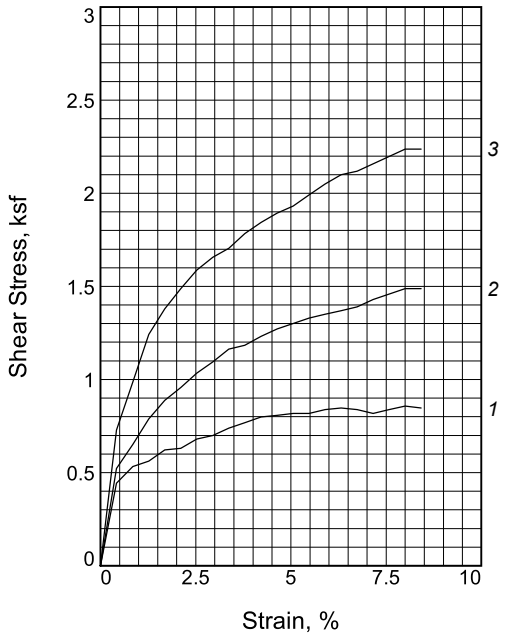
Allied Geotechnical Engineers, Inc.

Santee, CA

Client: RICK ENGINEERING COMPANY
Project: SERRA MESA STORM DRAINS PROJECT

Project No: 178 GS-16-C **Figure** B-8





Sample No.		1	2	3
Initial	Water Content, %	11.5	11.3	10.9
	Dry Density, pcf	97.8	97.6	98.1
	Saturation, %	43.4	42.3	41.3
	Void Ratio	0.7112	0.7142	0.7052
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	24.5	25.3	23.8
	Dry Density, pcf	97.8	97.6	98.1
	Saturation, %	92.5	94.9	90.6
	Void Ratio	0.7112	0.7142	0.7052
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		0.86	1.49	2.24
Strain, %		8.0	8.0	8.0
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.007	0.007	0.007

Sample Type: Remold

Description: Gray brown, clayey sand with gravel and cobble.

Specific Gravity= 2.68

Remarks: Remold at 80 percent of laboratory maximum density.

Figure B-9

Client: RICK ENGINEERING COMPANY

Project: SERRA MESA STORM DRAINS PROJECT

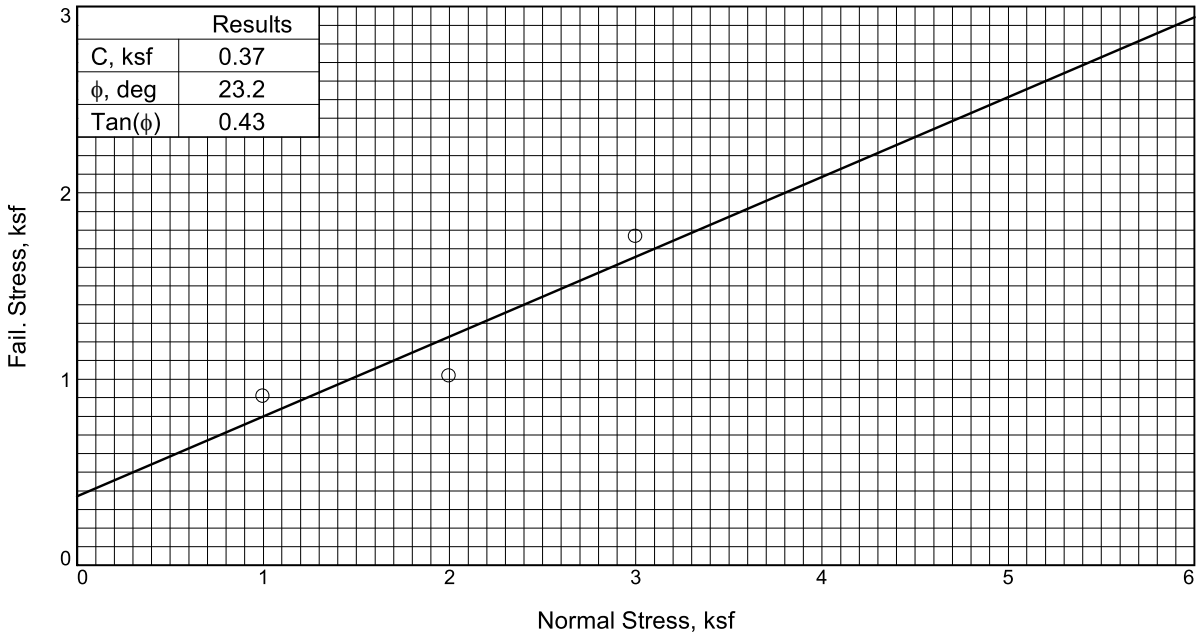
Source of Sample: TP1-B **Depth:** 3'

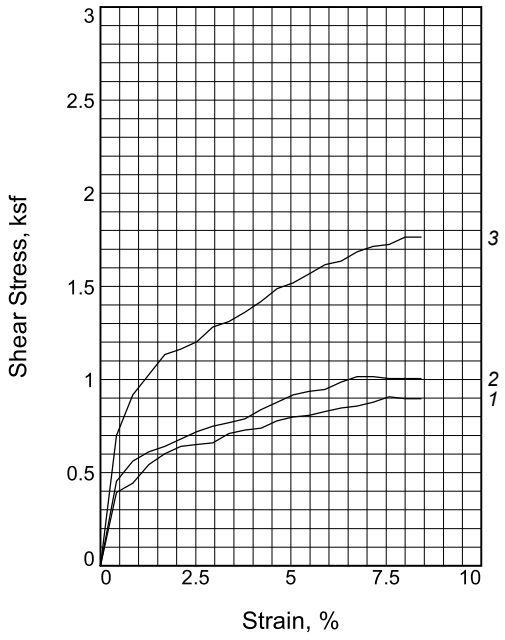
Sample Number: 2

Proj. No.: 178 GS-16-C **Date Sampled:**

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Tested By: B. Hayes **Checked By:** Sani Sutanto





Sample No.		1	2	3
Initial	Water Content, %	9.0	11.2	8.7
	Dry Density, pcf	108.0	100.2	103.5
	Saturation, %	43.9	44.6	37.9
	Void Ratio	0.5490	0.6700	0.6159
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	20.8	25.2	22.3
	Dry Density, pcf	108.0	100.2	103.5
	Saturation, %	101.4	100.9	97.2
	Void Ratio	0.5490	0.6700	0.6159
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		0.91	1.02	1.76
Strain, %		7.6	6.7	8.0
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.007	0.007	0.007

Sample Type: In-situ
Description: Red brown, silty to clayey sand.

Specific Gravity= 2.68
Remarks:

Client: RICK ENGINEERING COMPANY

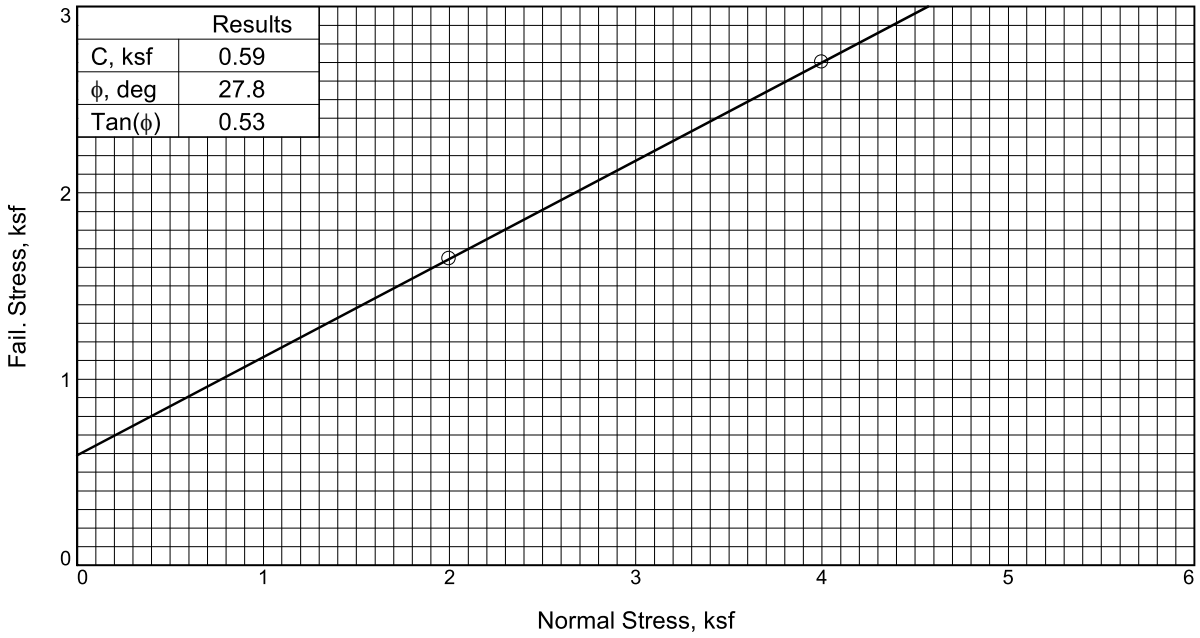
Project: SERRA MESA STORM DRAINS PROJECT

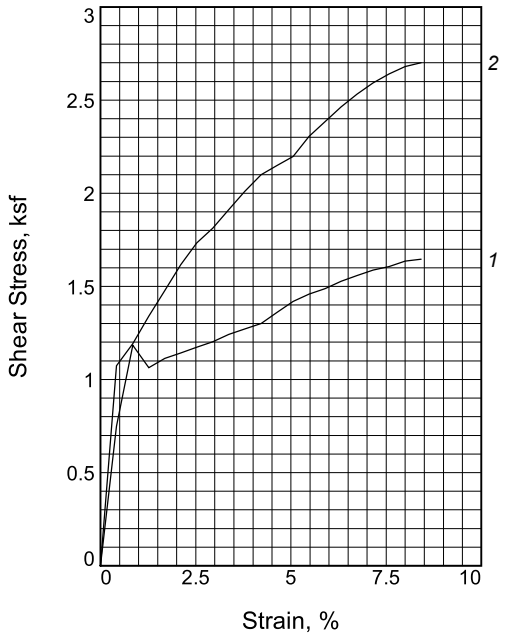
Source of Sample: TP2-A **Depth:** 4.5'
Sample Number: 5
Proj. No.: 178 GS-16-C **Date Sampled:**

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Figure B-10

Tested By: B. Hayes **Checked By:** Sani Sutanto





Sample No.		1	2
Initial	Water Content, %	6.4	6.4
	Dry Density, pcf	98.4	96.1
	Saturation, %	24.4	23.3
	Void Ratio	0.7007	0.7418
	Diameter, in.	2.38	2.38
	Height, in.	1.00	1.00
At Test	Water Content, %	16.9	23.7
	Dry Density, pcf	98.4	96.1
	Saturation, %	64.5	85.7
	Void Ratio	0.7007	0.7418
	Diameter, in.	2.38	2.38
	Height, in.	1.00	1.00
Normal Stress, ksf		2.00	4.00
Fail. Stress, ksf		1.65	2.70
Strain, %		8.4	8.4
Ult. Stress, ksf			
Strain, %			
Strain rate, in./min.		7.000	0.007

Sample Type: In-situ
Description: Reddish brown, silty sand with trace of fine gravel

Specific Gravity= 2.68
Remarks:

Figure B-11

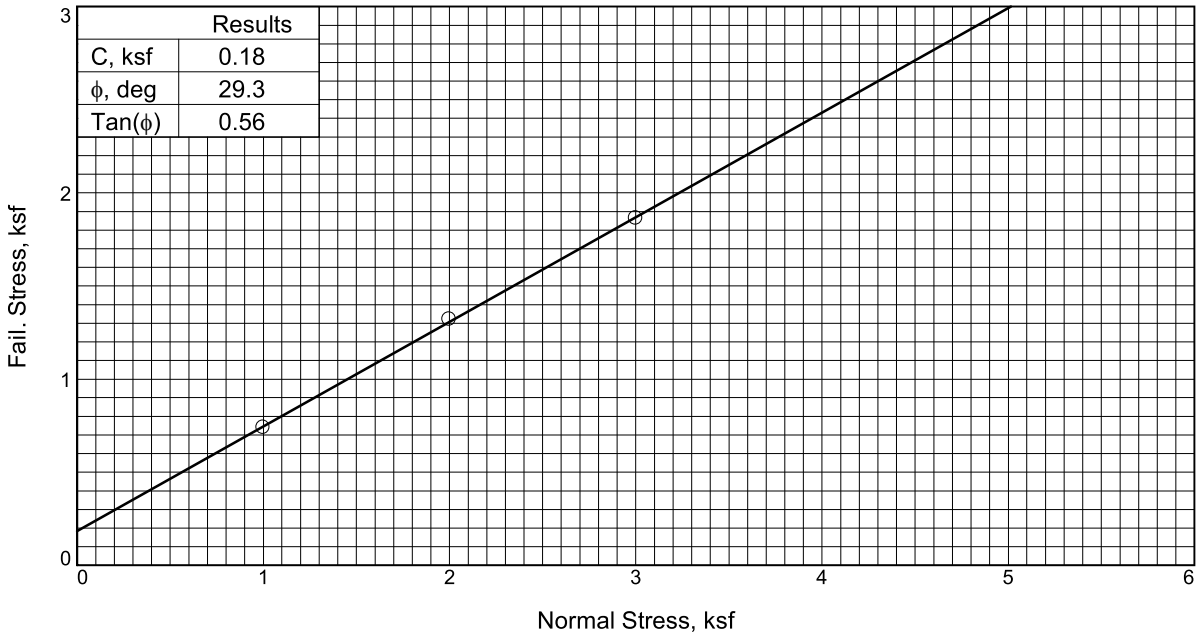
Client: RICK ENGINEERING COMPANY

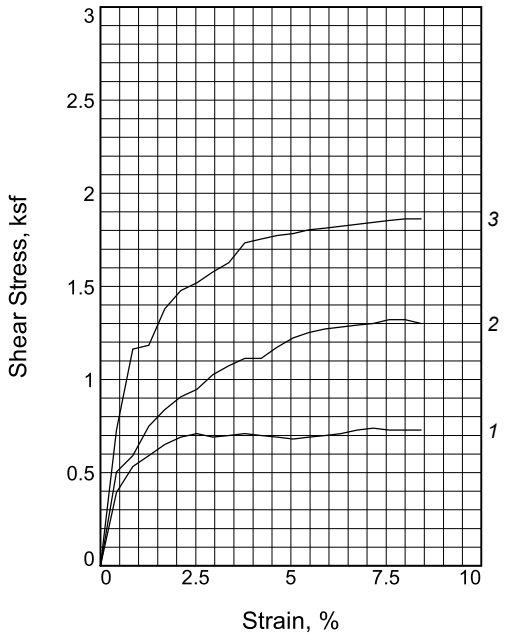
Project: SERRA MESA STORM DRAINS PROJECT

Source of Sample: TP4-A **Depth:** 3
Sample Number: 3
Proj. No.: 178 GS-16-C **Date Sampled:**

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Tested By: B. Hayes **Checked By:** Sani Sutanto





Sample No.		1	2	3
Initial	Water Content, %	11.5	10.1	11.3
	Dry Density, pcf	99.7	100.7	100.2
	Saturation, %	45.3	40.8	45.3
	Void Ratio	0.6787	0.6615	0.6700
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	24.7	23.7	23.4
	Dry Density, pcf	99.7	100.7	100.2
	Saturation, %	97.4	96.2	93.7
	Void Ratio	0.6787	0.6615	0.6700
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		0.74	1.32	1.86
Strain, %		7.2	7.6	8.0
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.007	0.007	0.007

Sample Type: Remold
Description: Gray brown, clayey sand with gravel

Specific Gravity= 2.68
Remarks: Remold at 80 percent of laboratory maximum density.

Figure B-12

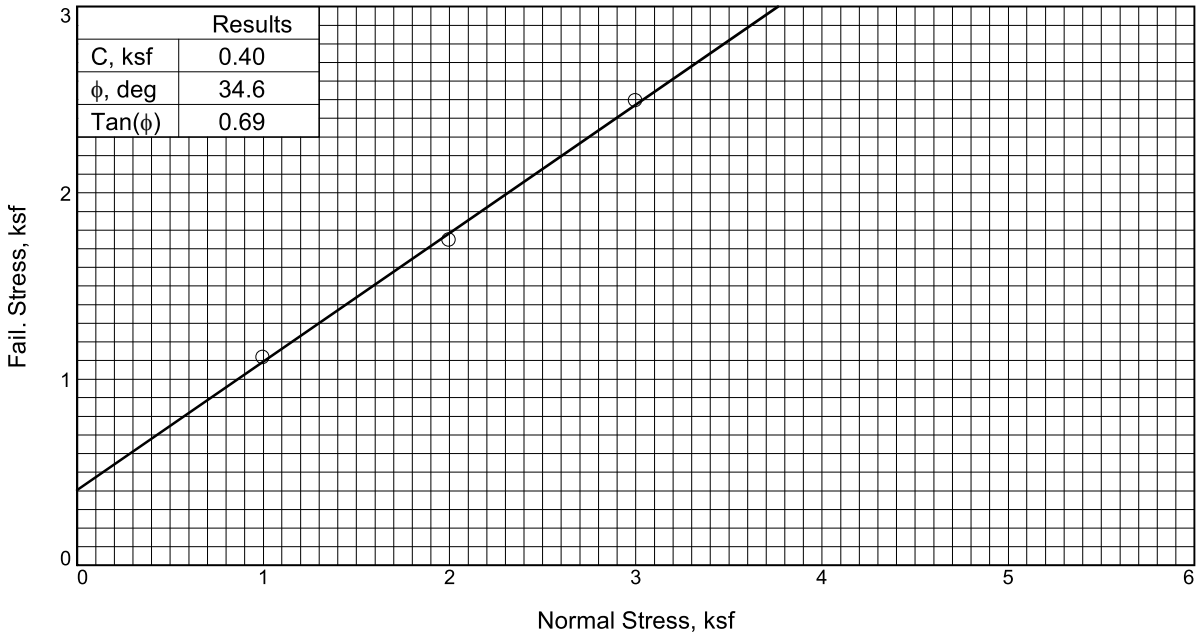
Client: RICK ENGINEERING COMPANY

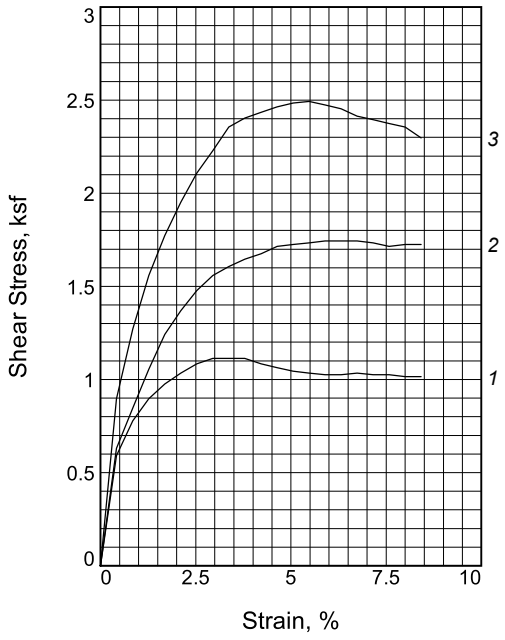
Project: SERRA MESA STORM DRAINS PROJECT

Source of Sample: TP4-B **Depth:** 2
Sample Number: 2
Proj. No.: 178 GS-16-C **Date Sampled:**

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Tested By: B. Hayes **Checked By:** Sani Sutanto





Sample No.		1	2	3
Initial	Water Content, %	15.5	15.1	13.3
	Dry Density, pcf	97.1	96.7	97.4
	Saturation, %	57.4	55.5	49.8
	Void Ratio	0.7233	0.7294	0.7172
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	24.5	25.1	23.7
	Dry Density, pcf	97.1	96.7	97.4
	Saturation, %	90.9	92.1	88.7
	Void Ratio	0.7233	0.7294	0.7172
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		1.11	1.74	2.49
Strain, %		2.9	5.9	5.5
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.007	0.007	0.007

Sample Type: In-situ
Description: Light gray, fine silty sand.

Specific Gravity= 2.68
Remarks:

Figure B-13

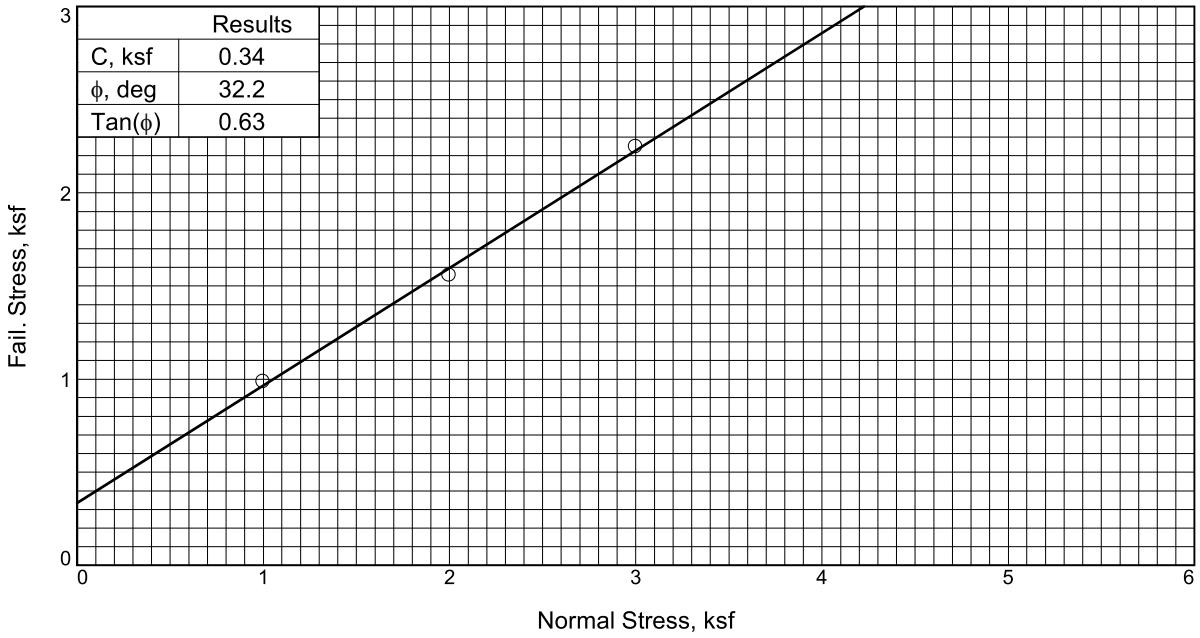
Client: RICK ENGINEERING COMPANY

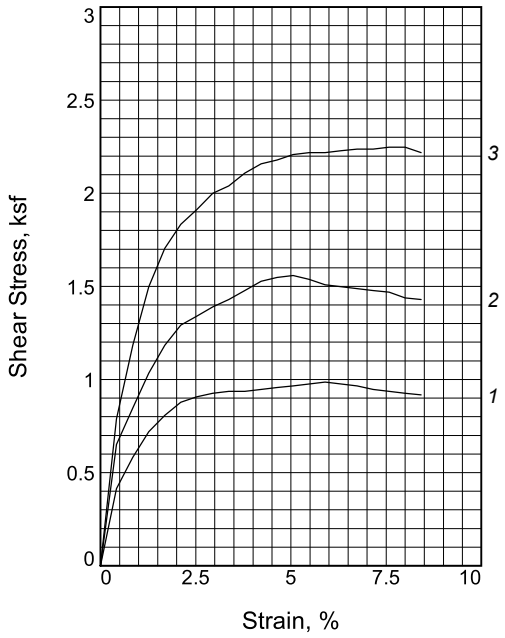
Project: SERRA MESA STORM DRAINS PROJECT

Source of Sample: TP5-A **Depth:** 4
Sample Number: 3
Proj. No.: 178 GS-16-C **Date Sampled:**

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Tested By: B. Hayes **Checked By:** Sani Sutanto





Sample No.		1	2	3
Initial	Water Content, %	16.4	17.2	17.2
	Dry Density, pcf	90.3	91.3	91.4
	Saturation, %	51.5	55.5	55.6
	Void Ratio	0.8530	0.8320	0.8303
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	27.8	28.8	27.4
	Dry Density, pcf	90.3	91.3	91.4
	Saturation, %	87.4	92.8	88.4
	Void Ratio	0.8530	0.8320	0.8303
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		0.99	1.56	2.25
Strain, %		5.9	5.1	7.6
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.007	0.007	0.007

Sample Type: In-situ
Description: Pale yellow, fine-grained sandy silt.

Specific Gravity= 2.68
Remarks:

Client: RICK ENGINEERING COMPANY

Project: SERRA MESA STORM DRAINS PROJECT

Source of Sample: TP6-A **Depth:** 3
Sample Number: 3
Proj. No.: 178 GS-16-C **Date Sampled:**

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Figure B-14

Tested By: B. Hayes **Checked By:** Sani Sutanto

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

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

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: October 10, 2019
Purchase Order Number: 178 GS-16-C
Sales Order Number: 45863
Account Number: ALLG

To:

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

Laboratory Number: SO7544-1 Customers Phone: 619-449-5900
Fax: 619-449-5902
Sample Designation:

One soil sample received on 10/04/19 at 3:05pm,
taken from Serra Mesa Storm Drain Project No 178 GS-16-C
marked as TP-1B-1@0'-1'.

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

pH 6.1

Water Added (ml)	Resistivity (ohm-cm)
10	5800
5	2400
5	1400
5	940
5	820
5	780
5	780
5	800
5	810

10 years to perforation for a 16 gauge metal culvert.
13 years to perforation for a 14 gauge metal culvert.
18 years to perforation for a 12 gauge metal culvert.
23 years to perforation for a 10 gauge metal culvert.
28 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.025% (250ppm)

Water Soluble Chloride Calif. Test 422 0.023% (240ppm)

Bicarbonate (as $CaCO_3$) 14ppm
(on a 1:3 water extraction)

Rosa Bernal
RMB/ilv

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

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

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: October 10, 2019
Purchase Order Number: 178 GS-16-C
Sales Order Number: 45863
Account Number: ALLG

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

Laboratory Number: SO7544-2 Customers Phone: 619-449-5900
Fax: 619-449-5902
Sample Designation:
--
One soil sample received on 10/04/19 at 3:05pm,
taken from Serra Mesa Storm Drain Project No 178 GS-16-C
marked as TP-2A-3@3'-3.5'.

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

pH 6.0

Water Added (ml)	Resistivity (ohm-cm)
10	20000
10	3400
5	2000
5	1800
5	1500
5	1700
5	1900

14 years to perforation for a 16 gauge metal culvert.
19 years to perforation for a 14 gauge metal culvert.
26 years to perforation for a 12 gauge metal culvert.
33 years to perforation for a 10 gauge metal culvert.
40 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.003% (33ppm)

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

Bicarbonate (as $CaCO_3$) 18ppm
(on a 1:3 water extraction)

Rosa Bernal
RMB/ilv

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

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

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: October 10, 2019
Purchase Order Number: 178 GS-16-C
Sales Order Number: 45863
Account Number: ALLG

To:

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

Laboratory Number: SO7544-3 Customers Phone: 619-449-5900
Fax: 619-449-5902
Sample Designation:

One soil sample received on 10/04/19 at 3:05pm,
taken from Serra Mesa Storm Drain Project No 178 GS-16-C
marked as TP-3A-3@3'-4'

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

pH 4.9

Water Added (ml)	Resistivity (ohm-cm)
10	2000
5	1100
5	980
5	690
5	650
5	670
5	690

Less than 5 years to perforation for a 16 gauge metal culvert.
Less than 5 years to perforation for a 14 gauge metal culvert.
5 years to perforation for a 12 gauge metal culvert.
7 years to perforation for a 10 gauge metal culvert.
8 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.025% (250ppm)
Water Soluble Chloride	Calif. Test 422	0.031% (310ppm)
Bicarbonate (as $CaCO_3$) (on a 1:3 water extraction)		8ppm

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Date: October 10, 2019
Purchase Order Number: 178 GS-16-C
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Account Number: ALLG

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

Laboratory Number: SO7544-4 Customers Phone: 619-449-5900
Fax: 619-449-5902
Sample Designation:
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One soil sample received on 10/04/19 at 3:05pm,
taken from Serra Mesa Storm Drain Project No 178 GS-16-C
marked as TP-4A-4@3.5'-4'.

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

pH 5.4

Water Added (ml)	Resistivity (ohm-cm)
10	4500
5	2000
5	1500
5	1200
5	1300
5	1600

10 years to perforation for a 16 gauge metal culvert.
12 years to perforation for a 14 gauge metal culvert.
17 years to perforation for a 12 gauge metal culvert.
22 years to perforation for a 10 gauge metal culvert.
27 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.005% (45ppm)

Water Soluble Chloride Calif. Test 422 0.006% (64ppm)

Bicarbonate (as $CaCO_3$) 12ppm
(on a 1:3 water extraction)

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Date: October 10, 2019
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Account Number: ALLG

To:

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

Laboratory Number: SO7544-5 Customers Phone: 619-449-5900
Fax: 619-449-5902
Sample Designation:

One soil sample received on 10/04/19 at 3:05pm,
taken from Serra Mesa Storm Drain Project No 178 GS-16-C
marked as TP-5A-4@4.5'-5'.

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

pH 6.6

Water Added (ml)	Resistivity (ohm-cm)
10	6400
5	2200
5	1700
5	1900
5	2200

20 years to perforation for a 16 gauge metal culvert.
26 years to perforation for a 14 gauge metal culvert.
36 years to perforation for a 12 gauge metal culvert.
46 years to perforation for a 10 gauge metal culvert.
56 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.005% (48ppm)

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

Bicarbonate (as $CaCO_3$) 12ppm
(on a 1:3 water extraction)

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Date: October 10, 2019
Purchase Order Number: 178 GS-16-C
Sales Order Number: 45863
Account Number: ALLG

To:

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

Laboratory Number: SO7544-6 Customers Phone: 619-449-5900
Fax: 619-449-5902
Sample Designation:

One soil sample received on 10/04/19 at 3:05pm,
taken from Serra Mesa Storm Drain Project No 178 GS-16-C
marked as TP-6A-4@3.5'-4'.

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

pH 4.6

Water Added (ml)	Resistivity (ohm-cm)
10	960
5	550
5	360
5	300
5	310
5	340

Less than 5 years to perforation for a 16 gauge metal culvert.
Less than 5 years to perforation for a 14 gauge metal culvert.
Less than 5 years to perforation for a 12 gauge metal culvert.
Less than 5 years to perforation for a 10 gauge metal culvert.
Less than 5 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.048% (480ppm)

Water Soluble Chloride Calif. Test 422 0.080% (800ppm)

Bicarbonate (as $CaCO_3$) 4ppm
(on a 1:3 water extraction)

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Date: October 10, 2019
Purchase Order Number: 178 GS-16-C
Sales Order Number: 45863
Account Number: ALLG

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

Laboratory Number: SO7544-7 Customers Phone: 619-449-5900
Fax: 619-449-5902
Sample Designation:
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One soil sample received on 10/04/19 at 3:05pm,
taken from Serra Mesa Storm Drain Project No 178 GS-16-C
marked as TP-7B-4@4'-5'.

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

pH 5.7

Water Added (ml)	Resistivity (ohm-cm)
10	6300
5	3500
5	3100
5	2700
5	2900
5	3200

17 years to perforation for a 16 gauge metal culvert.
22 years to perforation for a 14 gauge metal culvert.
31 years to perforation for a 12 gauge metal culvert.
39 years to perforation for a 10 gauge metal culvert.
48 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.007% (66ppm)

Water Soluble Chloride Calif. Test 422 0.006% (64ppm)

Bicarbonate (as $CaCO_3$) 4ppm
(on a 1:3 water extraction)

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