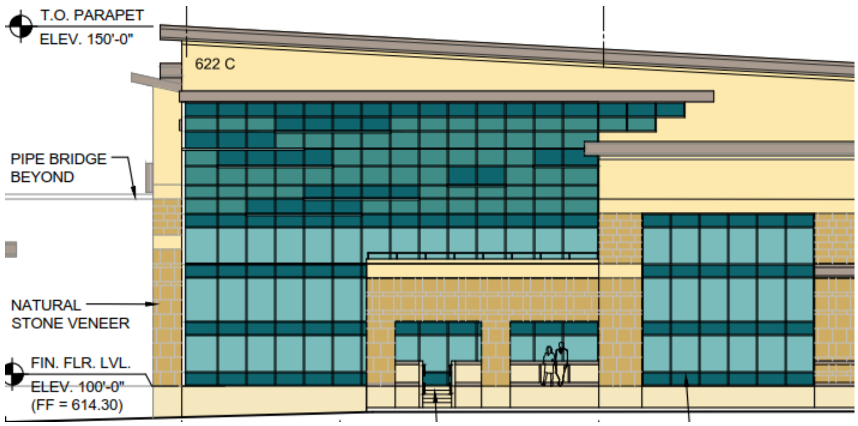
# Hollandia Dairy CUP Modification

# Technical Appendices

## Appendix D

## Geotechnical Investigation

# REPORT
# GEOTECHNICAL INVESTIGATION




Hollandia Dairy Fluid Milk Plant Improvements
622 E. Mission Road, San Marcos, California

**PREPARED FOR**

c/o TFW Construction
7460 Mission Valley Road, Suite 200
San Diego, CA 92108

**PREPARED BY**

NOVA Services, Inc.
4373 Viewridge Avenue, Suite B
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June 18, 2019
NOVA Project 2019039

TFW Construction, Inc.
7460 Mission Valley Road, Suite 200
San Diego, CA 92108

June 18, 2019
Project No. 2019039

Attention: Mr. Ted Weeks IV

Subject: Report
Geotechnical Investigation
Hollandia Dairy Fluid Milk Plant Improvements
622 E. Mission Road, San Marcos, California

Dear Mr. weeks:
NOVA Services, Inc. (NOVA) is pleased to forward herewith the above-referenced report. Work related to this report was completed by NOVA for TFW Construction, Inc. in accordance with the scope of work identified in NOVA's proposal dated February 12, 2019, as authorized on February 19, 2019. A draft of this report was provided on April 18 for your review.

NOVA appreciates the opportunity to be of services to TFW Construction and Hollandia Dairy on this most interesting project. Should you have any questions regarding this report or other matters, please contact the undersigned at (858) 292-7575.

Sincerely,

**NOVA Services, Inc.**

Wail Mokhtar
Project Manager

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# REPORT
# GEOTECHNICAL INVESTIGATION

## Hollandia Dairy Fluid Milk Plant Improvements
## 622 E. Mission Road, San Marcos, California

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# 1.0 INTRODUCTION

## 1.1 Terms of Reference

### 1.1.1 General

This report provides recommendations for the design of foundations and pavements, as well as development of permanent stormwater infiltration Best Management Practices ('BMPs') for the construction of new facilities at the existing Hollandia Dairy Fluid Milk Plant in San Marcos, California.

Work related to this report was completed by NOVA Services, Inc. (NOVA) for TFW Construction, Inc. (TFW) in accordance with the scope of work identified in NOVA's proposal dated February 12, 2019, authorized on February 19, 2019. A draft of this report was provided on April 18, 2019.

The Hollandia Dairy Fluid Milk Plant is located at 622 E. Mission Road in San Marcos, California (hereinafter, also referenced as "the site"). Figure 1-1 depicts the site vicinity.


**Figure 1-1. Vicinity Map**

## 1.2 Objectives, Scope, and Limitations of This Work

### 1.2.1 Objectives

The work reported is intended to characterize the subsurface conditions within the limits of the planned improvements in a manner sufficient to address the following two objectives:

1. Objective 1, Geotechnical. Develop recommendations for geotechnical-related development including foundations, pavements and earthwork.

2. Objective 2, Stormwater. Assess requirements for development of stormwater infiltration Best Management Practices ('stormwater BMPs').

1.2.2 Scope

In order to accomplish the above objectives, NOVA undertook the task-based scope of work described below.

1. Task 1, Background Review. Reviewed readily available background data regarding the site area, including geotechnical reports, topographic maps, geologic data, fault maps and reports, and development plans for the project. Available architectural and information was reviewed.

2. Task 2, Subsurface Exploration. The subsurface exploration included the subtasks listed below.

    - *Subtask 2-1, Reconnaissance*. Prior to undertaking any invasive work, NOVA conducted a site reconnaissance, including layout of exploratory borings used to determine subsurface conditions. Underground Service Alert was notified for underground utility mark-out services.

    - *Subtask 2-2, Permitting and Coordination.* NOVA coordinated with you regarding access for fieldwork. Engineering borings were permitted in accordance with San Diego County Department of Health Services requirements.

    - *Subtask 2-3, Engineering Borings*. A NOVA geologist directed drilling and sampling of four (4) engineering borings to depths of up to 42 feet below existing ground surface (bgs). The borings were drilled using truck mounted hollow stem auger equipment, sampling the borings using ASTM methods.

    - *Subtask 2-4, CPT Soundings*. Six (6) cone penetration test ('CPT', after ASTM D5778) soundings were extended to depths of 30 feet to 43 feet bgs.

    - *Subtask 2-5, Percolation Testing.* Two (2) hollow stem auger borings were drilled at the location of prospective stormwater infiltration BMPs. The borings were converted to percolation test wells, and percolation data obtained in accordance with the San Diego County BMP Design Manual (January 2018 edition).

3. Task 3, Laboratory Testing. Laboratory testing addressed index soil characteristics, as well as testing to address the potential that soils may be corrosive to embedded concrete or metals.

4. Task 4, Engineering Evaluations. The findings of Tasks 1-3 were utilized to support geotechnical evaluations and an assessment of the feasibility of stormwater infiltration.

5. Task 5, Reporting. Submittal of this report completes NOVA's scope of work.

#### 1.2.3 Limitations

The recommendations for design and construction included in this report are not final. These recommendations are developed by NOVA using judgment and opinion and based on the information available at the time of the report. NOVA can finalize its recommendations only by observing actual subsurface conditions revealed during construction. NOVA cannot assume responsibility or liability for the report's recommendations if NOVA does not perform construction observation.

This report does not address any environmental assessment or investigation for the presence or absence of hazardous, toxic or regulated materials in the soil, groundwater, or surface water within or beyond the site.

### 1.3 Understood Use of This Report

NOVA expects that the findings and recommendations provided herein will be utilized by the TFW's Design Team in certain decision-making regarding design and construction of the planned development.

NOVA's recommendations are based on its current understanding and assumptions regarding project development. Effective use of this report by the Design Team should include review by NOVA of the final design. Such review is important for both (i) conformance with the recommendations provided herein, and (ii) consistency with NOVA's understanding of the planned development.

### 1.4 Report Organization

The remainder of this report is organized as abstracted below.

- Section 2 reviews available project information.
- Section 3 describes subsurface exploration.
- Section 4 describes the surface and subsurface conditions.
- Section 5 reviews geologic, soil and siting-related hazards common to this area of San Diego, considering each for its potential to affect the planned improvements.
- Section 6 provides recommendations for earthwork and foundation design.
- Section 7 provides recommendations for development of stormwater infiltration BMPs.
- Section 8 provides recommendations for development of pavements.
- Section 9 provides a list of the principal references utilized in the development of the report.

Figures that directly support discussion in the text are embedded therein. Larger scale plots of subsurface information are provided as Plates immediately following the text of the report.

The report is supported by four appendices. Appendix A provides guidance regarding the use and limitations of the report. Appendix B provide boring logs. Appendix C provides infiltration worksheets. Appendix D provides records of laboratory testing by NOVA.

# 2.0 PROJECT INFORMATION

## 2.1 Site Description

### 2.1.1 Location

The Hollandia Dairy Fluid Milk Plant is located at 622 E. Mission Road in San Marcos, California.

The plant is bounded by existing cut slopes and Mission Hills Court to the north and east, Mission Road to the south, and Mulberry Drive to the west. Figure 2-1 depicts the site vicinity and approximate limits of the Hollandia Dairy Fluid Milk Plant.


**Figure 2-1. Site Location and Limits**

### 2.1.2 Current Site Use

As is evident by review of Figure 2-1, the site is fully developed for industrial use, covered by either structure or pavements. The locations of the new facilities are covered by pavement.

Existing site elevations range from about +610 feet mean sea level (msl) at the southwest corner at Mulberry Drive, to about +630 msl in the northeast corner.

#### 2.1.3 Historic Site Use

Review of historical aerial photography dating to 1938 indicates little development in the site area prior to 1964. Figure 2-2 provides an aerial photograph depicting the area of the dairy in 1964.

**Figure 2-2. 1964 Aerial View of the Site Area**

### 2.2 Planned Development

#### 2.2.1 General

NOVA's understanding of the improvements planned for the plant are based upon review of conceptual level architectural drawings by E. A. Bonelli and Associates, Inc. (reference, *Overall Site Plans: Existing, Phase 1, Phase 1 and 2, and Phase 1, 2 and 3, Hollandia Dairy Fluid Milk Plant*, E. A. Bonelli and Associates, Inc., February 2019, hereinafter 'EAB 2019').

Improvements to the dairy will be completed in three phases, adapting new structures to existing structures, infrastructure and utilities. Proposed phased construction is abstracted below.

- Phase 1: Demolition of existing cooler warehouse buildings, construction of a utility building containing chillers, boilers, electrical, water/air (Phase 1A); construction of a process facility building (Phase 1B); and construction of a metal canopy over a case return loading dock (Phase 1C).

- Phase 2B: Demolition of existing process facility building, construction of a maintenance building, employee welfare building, and a pipe bridge.

- Phase 3: Construction of a process facility building and offices (future).

Figure 3 on the following page is a representation of all phased construction.

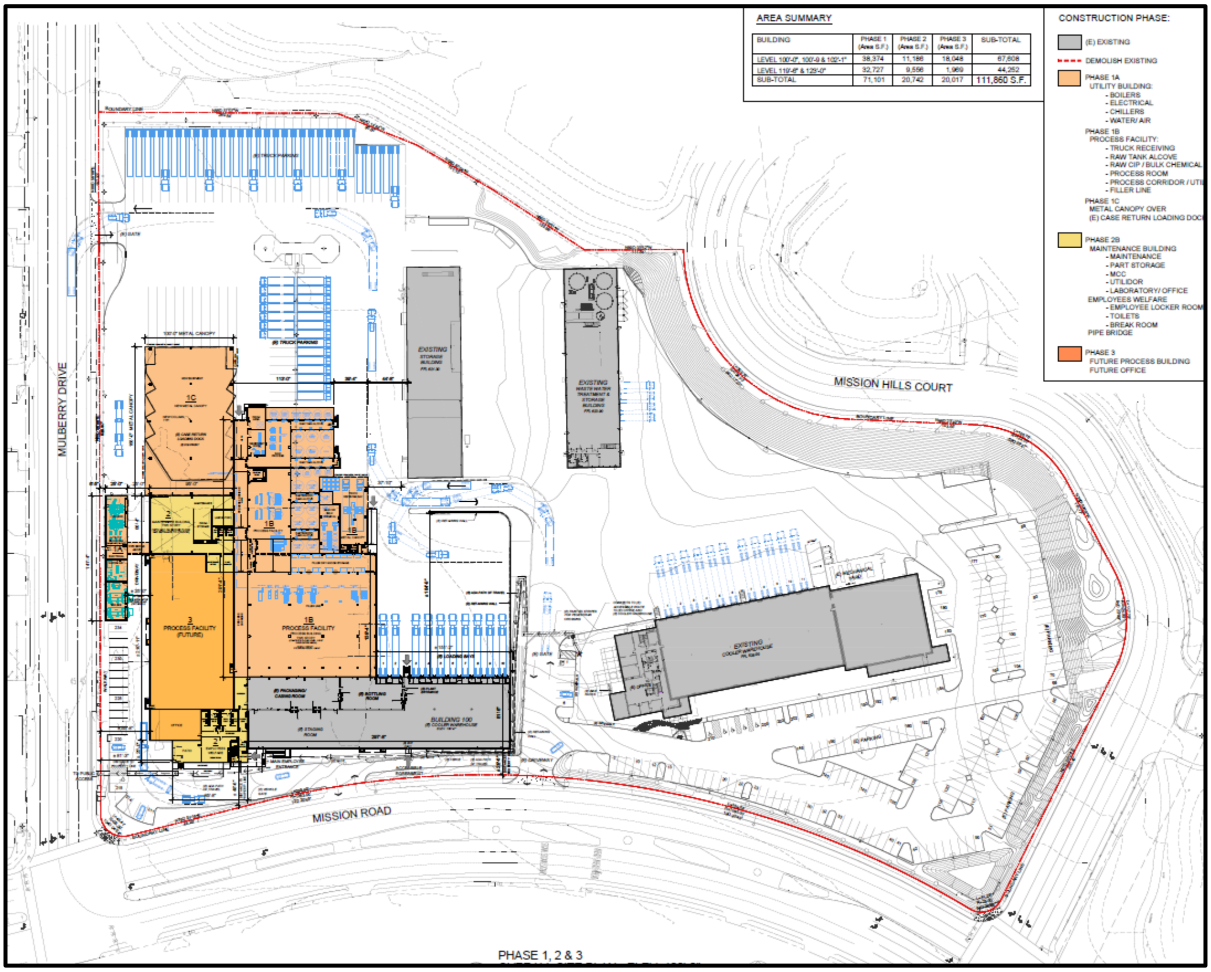


**Figure 2-3. Phased Development Plan**

Figure 2-4 and Figure 2-5 (following page) depict the milk plant at full buildout. As may be seen by review of this figure, design provides for two levels of administrative offices at the west end of the building, with the remainder of the plant developed at a single level. A dock-high fill will allow truck access to the plant.

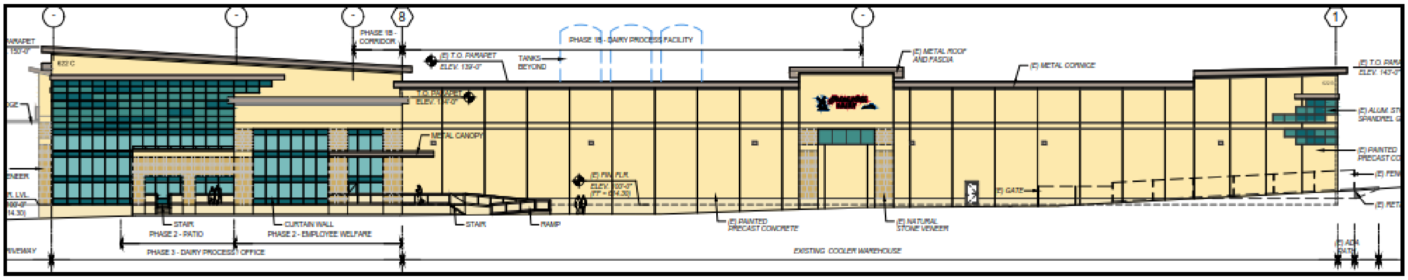

**Figure 2-4. Milk Plant At Full Buildout, Looking North**
(source: EAB 2019)

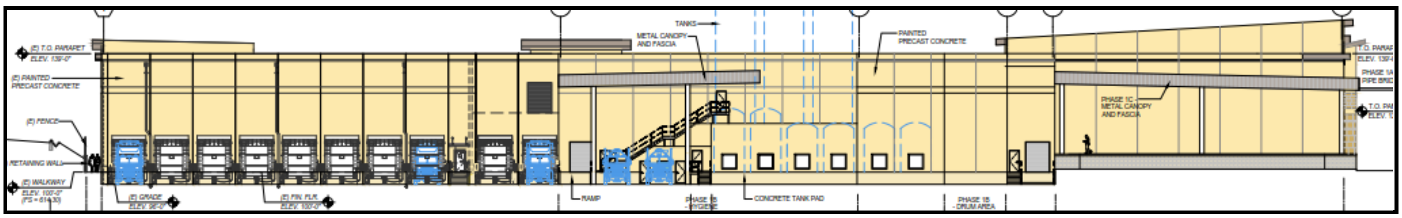

**Figure 2-4. Milk Plant At Full Buildout, Looking South**
(source: EAB 2019)

### 2.2.2 Potential for Earthwork

Based upon review of conceptual architectural drawings, it appears that limited earthwork will be required to achieve design grades for new structures. NOVA's understanding of the planned earthwork in each of the buildings is summarized on Table 2-1 and depicted in Figure 2-6 (following page).

**Table 2-1. Anticipated Earthwork at Each of the Planned Buildings**

| Building | Approximate Building Pad Elevation (feet, msl) | Approx. Thickness of Fill (feet) | Anticipated Construction Phase |
|---|---|---|---|
| Employee Welfare | + 614 | 5 | 2 |
| Process Facility | +614 to +617 | 5 to 7 | 1B and 3 |
| Maintenance Building | +616 | 5 | 2 |
| Utility Building | +614 | Nil to 4 | 1A |

Notes to Table 2-1:
1. Locations of the buildings are shown on Figure 2-3.
2. Pad elevations are estimated
2. Up to 2 feet of cut is anticipated in the northern portion of the process facility near the proposed tanks.

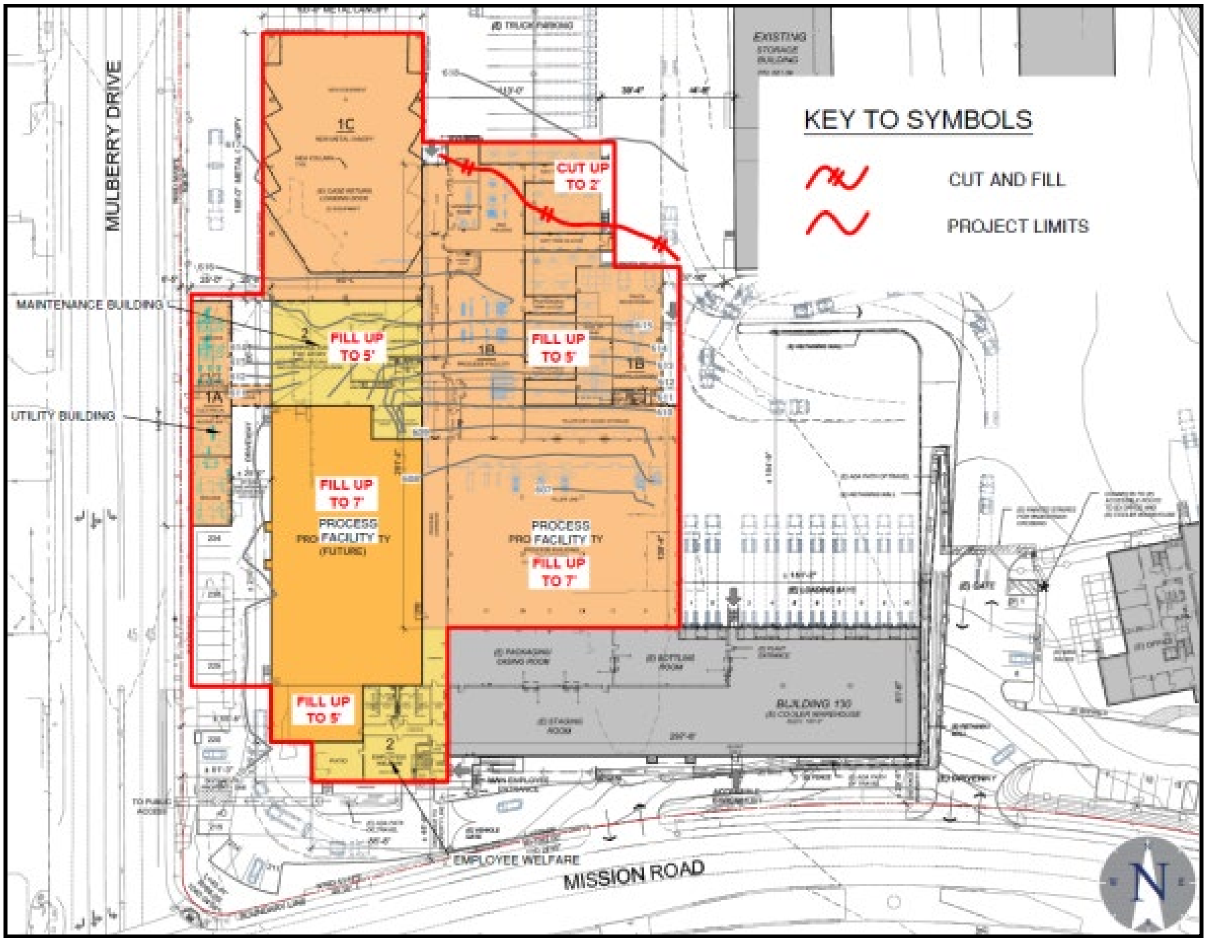


**Figure 2-6. Anticipated Earthwork at Each of the Planned Buildings**
(source: adapted from EAB 2019)

### 2.2.3 Stormwater BMPs

There is no available regarding planning for permanent stormwater infiltration BMPs.

# 3.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

## 3.1 Overview

Four (4) hollow-stem auger borings were drilled on March 9 - 10, 2019. Six (6) cone penetration test soundings ('CPT', after ASTM 5778) were completed on March 9. Two percolation test borings ('P-1' and 'P-2') were drilled on March 10 at the location of prospective stormwater management BMPs. Percolation testing was completed on March 10. Samples collected from the engineering and percolation test borings were returned to NOVA's materials laboratory for inspection and testing.

Figure 3-1 depicts the locations of the separate elements of the subsurface exploration. Plate 1, provided immediately following the text of this report, depicts the above information in larger scale.

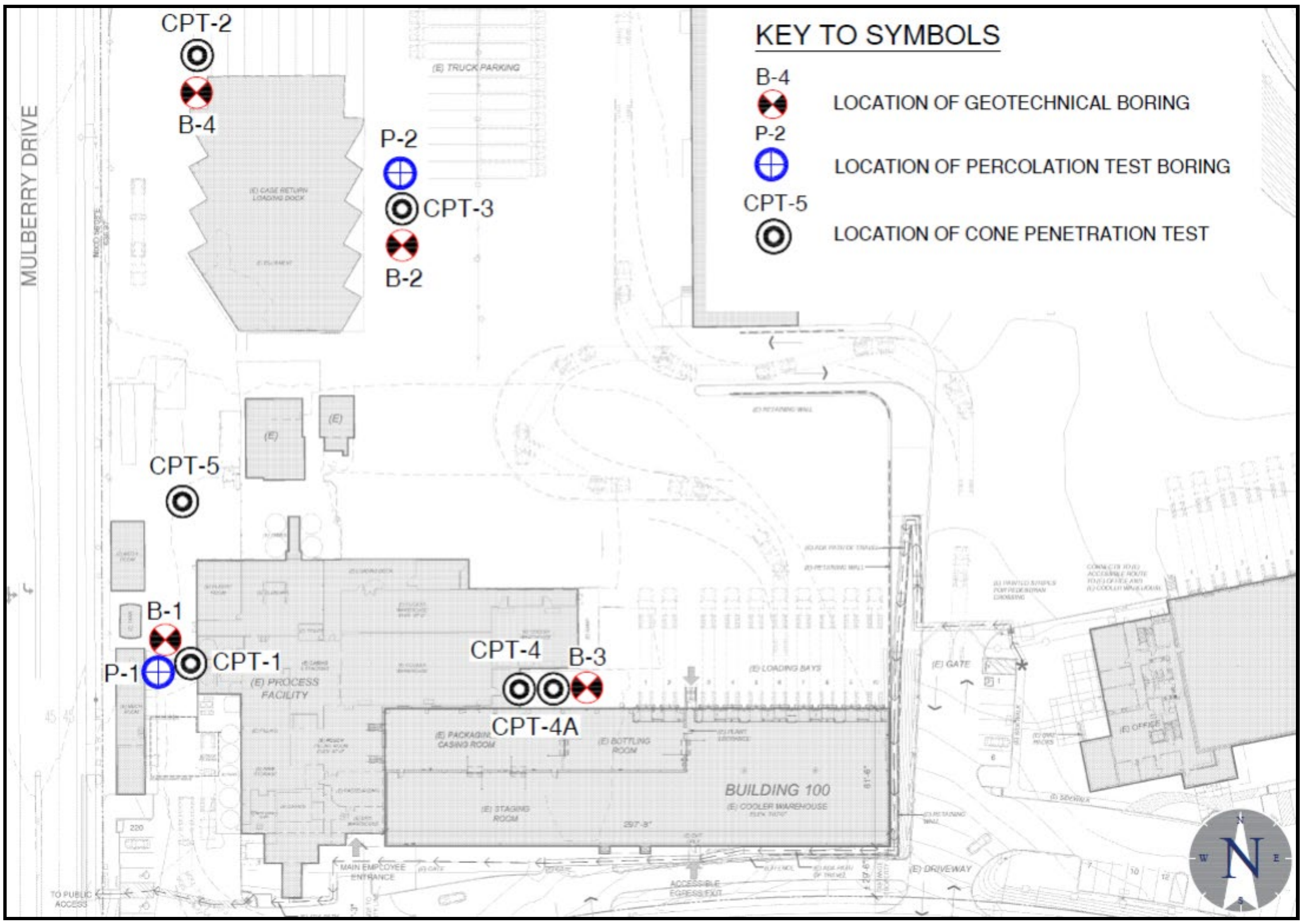


**Figure 3-1. Location of the Engineering Borings, CPT Soundings and Percolation Test Borings**

The following subsections describe the subsurface exploration and related laboratory testing.

## 3.2 Engineering Borings

### 3.2.1 General

Engineering borings were advanced by a truck-mounted drilling rig utilizing hollow stem drilling equipment. The borings were completed under the surveillance of a geologist from NOVA who directed sampling and maintained a log of the subsurface materials that were encountered.

Table 3-1 provides an abstract of the indications of the engineering borings by NOVA.

**Table 3-1. Abstract of the Engineering Borings**

| Boring Reference | Approximate Ground Surface Elevation (feet, msl) | Total Depth Below Ground Surface (feet) | Elevation at Completion (feet, msl) | Depth to Groundwater (feet) |
|---|---|---|---|---|
| B-1 | +611.3 | 42.2 | +569.1 | 8 |
| B-2 | +616.1 | 39.1 | +577 | 7.1 |
| B-3 | +610.3 | 29.7 | +580.6 | 4.1 |
| B-4 | +616.1 | 16.5 | +599.6 | 7 (estimated) |

Notes:
1. B-1, B-2 and B-3 extended to formational granitics (Mzu). B-4 terminated in the Older Alluvium (Qoa)
2. Groundwater was encountered in borings B-1, B-2 and B-3. Groundwater level estimated at B-4

Figure 3-2 (following page) depicts drilling operations in March 2019.

### 3.2.2 Sampling

Both disturbed and relatively undisturbed samples were recovered from the borings, sampling of soils is described below.

1. The Modified California sampler ('ring sampler', after ASTM D 3550) was driven using a 140-pound hammer falling for 30 inches with a total penetration of 18 inches, recording blow counts for every 6 inches of penetration.

2. The Standard Penetration Test sampler ('SPT', after ASTM D 1586) was driven in the same manner as the ring sampler, recording blow counts in the same fashion. SPT blow counts for the final 12 inches of penetration comprise the SPT 'N' value, an index of soil consistency.

3. Bulk samples were recovered from the upper 5 feet of the subsurface, providing composite samples for testing of soil moisture and density relationships and corrosivity.

Logs of the borings are provided in Appendix B.

### 3.2.3 Closure

On completion, the borings were backfilled with cuttings. The area of each boring was cleaned and left as close to the original condition as practical.

**Figure 3-2. Drilling Operations on March 10, 2019**

## 3.3 CPT Soundings

### 3.3.1 General

Six (6) Cone Penetrometer Test soundings ('CPT', after ASTM 5778) were completed to depths of between 29.9 feet and 43.3 feet bgs on March 9, 2019 at locations indicated on Figure 3-1.

Table 3-2 (following page) abstracts the CPT soundings.

**Table 3-2. Abstract of the CPT Soundings**

| Sounding | Approx. Elevation (feet, msl) | Total Depth (feet) | Tip Elevation (feet, msl) |
|---|---|---|---|
| CPT-1 | +611.3 | 40.7 | +570.6 |
| CPT-2 | +616.1 | 43.3 | +572.8 |
| CPT-3 | +616.1 | 43.0 | +573.1 |
| CPT-4 | +610.3 | 29.9 | +580.4 |
| CPT-4A | +610.3 | 30.1 | +580.2 |
| CPT-5 | +611.3 | 39.8 | +571.5 |

Both the CPT soundings and the borings indicate the subsurface is dominated by finer grained soils. Subsurface exploration using the cone penetration test (CPT) allows development of a continuous profile of the subsurface, useful for more detailed characterization of the soils. Figure 3-3 reproduces the profile develop by CPT-1, from which it can be seen that silts and clays dominate the near subsurface.

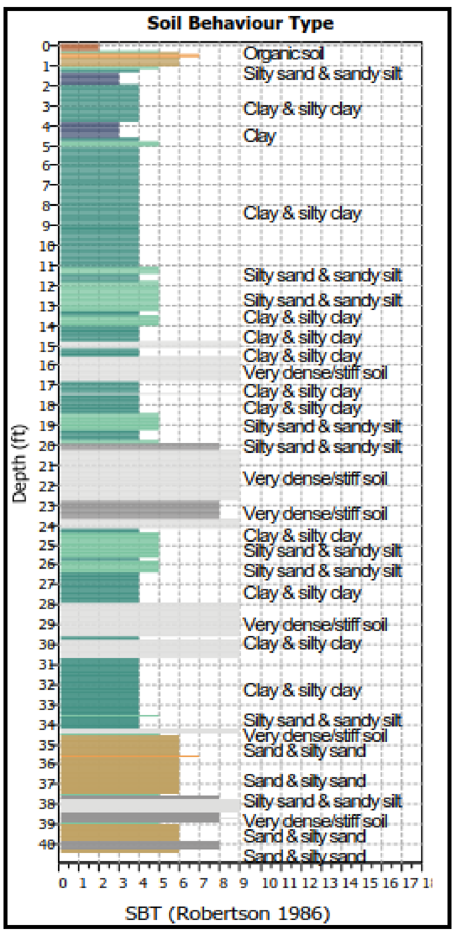


**Figure 3-3. Subsurface Profile at CPT-1**

### 3.3.2 Stiffness of the Soils

Finer grained soils are often associated with lesser strength and higher compressibility, commonly making these soils unfavorable for development of foundations. However, the data obtained from the subsurface exploration indicates the soils at this site are relatively stiffer.

Figure 3-4 is a graphic that provides the variation of constrained soil modulus (M), shear modulus (G) and shear strength with depth, all indicators of soil stiffness.

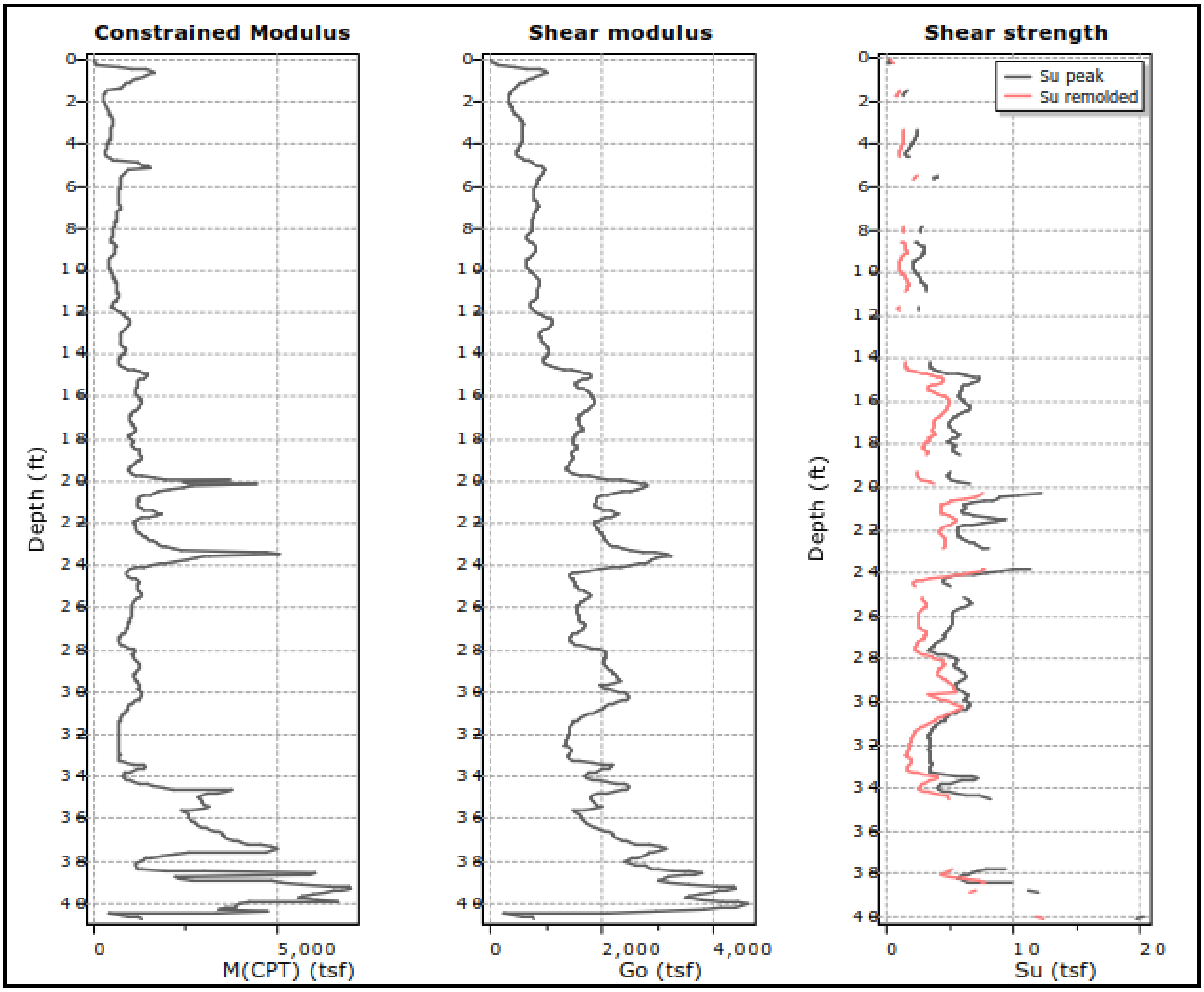


**Figure 3-4. Stiffness of the Subsurface**

## 3.4 Percolation Testing

### 3.4.1 General

NOVA directed the excavation and construction of two (2) percolation test borings following the recommendations for percolation testing presented in the latest edition of the City of San Marcos <u>BMP Design Manual</u>. The locations of these borings are shown in Figure 3-1.

### 3.4.2 Drilling

The borings were drilled with a truck mounted 8-inch hollow stem auger to approximate depths of 5 feet bgs. Field measurements were taken to confirm that the borings were excavated to approximately 8-inches in diameter. The borings were logged by a NOVA geologist, who observed and recorded exposed soil cuttings and the boring conditions.

The borings were logged by a NOVA geologist, who observed and recorded exposed soil cuttings and boring conditions. Logs of the percolation test borings are provided in Appendix B.

### 3.4.3 Conversion to Percolation Wells

Once a boring was drilled to the desired depth, the boring was converted to percolation test well by placing an approximately 2-inch layer of ¾-inch gravel on the bottoms, then extending 3-inch diameter Schedule 40 perforated PVC pipe to the ground surface. The ¾-inch gravel was used to partially fill the annular space around the perforated pipe below the existing finish grade to minimize the potential of soil caving.

### 3.4.4 Percolation Testing

The percolation test wells were each pre-soaked by filling the wells with water to at least 5 times the well radius. The pre-soak water did not percolate at least 6 inches into the soil unit within 25 minutes; therefore, the holes were filled to the ground surface elevation and testing commenced the following day, within a 26-hour window.

Water levels were then recorded every 30 minutes for six hours (minimum of 12 readings), or until the water percolation stabilized after each reading. The water level was raised to close to the previous water level to maintain a near constant head before subsequent readings.

Table 3-3 abstracts the indications of the percolation testing.

**Table 3-3. Abstract of the Percolation Testing**

| Boring | Approx. Elevation (feet, msl) | Total Depth (feet) | Approximate Percolation Test Elev. (feet, msl) | Percolation Rate (in/hour) [2] | Subsurface Units Tested[1] |
|---|---|---|---|---|---|
| P-1 | +611.3 | 5 | +606.3 | 0.24 | Qya |
| P-2 | +616.1 | 5 | +611.1 | 0.48 | Qya |

Notes:
1. The referenced geologic unit is Younger Alluvium (Qya).
2. Table addresses field-measured percolation rate. Section 7 addresses design infiltration rates.

### 3.4.5 Closure

At the conclusion of the percolation testing, the upper sections of the PVC pipe were removed and the resulting holes backfilled with soil cuttings to match the existing surfacing.

## 3.5 Laboratory Testing

### 3.5.1 General

Soil samples recovered from the engineering borings were transferred to NOVA's geotechnical laboratory where a geotechnical engineer reviewed the soil samples and the field logs. Representative soil samples were selected and tested in NOVA's materials laboratory to check visual classifications and to determine pertinent engineering properties. The laboratory program included visual classifications of all soil samples as well as index and expansivity testing in general accordance with ASTM standards.

Records of the geotechnical laboratory testing are provided in Appendix D.

### 3.5.2 Compaction

A single composite sample of the sandy fraction of near-surface soil was tested to determine the moisture-density characteristics during compaction after ASTM D1557 (the 'modified Proctor'). This testing indicated a maximum dry unit weight of 127 pounds per cubic foot at an optimum moisture content of 7.7%.

### 3.5.3 Plasticity and Expansion Potential

The visual classifications were supplemented by index testing to determine plasticity.

Testing of the near surface clayey soils after ASTM D 4829 to determine Expansion Index, as well as visual inspection of samples recovered by NOVA, indicates the soil is moderately to highly expansive. Testing of a samples of the younger alluvium indicated EI = 71, suggesting 'Medium' expansion potential.

The sample with EI = 71 had a related Atterberg Liquid Limit (LL) of LL = 31 and Plasticity Index (PI) of PI = 15, suggestive of a lower plasticity clay ('CL,' after ASTM 2487).

### 3.5.4 Gradation

Table 3-4 (following page) summarizes the results of gradation testing of soils recovered from the borings.

### 3.5.5 Corrosion Potential

Resistivity, sulfate content and chloride contents were determined to estimate the potential corrosivity of on-site soils. These chemical tests were performed on a representative sample of the near-surface soils by Clarkson Laboratory and Supply, Inc.

The testing indicated low levels of soluble sulfates and chlorides in soils. Section 6 discusses the indications of the chemical testing.

**Table 3-4. Abstract of the Soil Gradation Testing**

| Boring | Depth (feet) | Percent Finer Than the U.S. No. 200 Sieve (0.074 m) | Soil Classification |
|---|---|---|---|
| B-1 | 2.5 | 77 | CL |
| B-1 | 12 – 15 | 70 | CL |
| B-1 | 20 | 38 | SM |
| B-1 | 25 | 52 | ML |
| B-1 | 30 | 45 | SM-ML |
| B-1 | 35 | 25 | SM |
| B-1 | 40 | 14 | SM |
| B-2 | 10 | 71 | ML-CL |
| B-2 | 15 | 49 | SM-ML |
| B-2 | 20 | 89 | ML |
| B-2 | 30 | 51 | ML |
| B-2 | 38 | 21 | SM |
| B-3 | 1 | 30 | SC |
| B-3 | 7.5 | 78 | CL |
| B-3 | 15 | 72 | CL |
| B-4 | 1 | 55 | CL |
| B-4 | 5 | 77 | ML-CL |

# 4.0 SITE CONDITIONS

## 4.1 Geologic Setting

### 4.1.1 Regional

The project area is located in the coastal portion of the Peninsular Range geomorphic province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California. The province varies in width from approximately 30 to 100 miles.

This area of the Province has undergone several episodes of marine inundation and subsequent marine regression (coastline changes) throughout the last 54 million years. These events have resulted in the deposition of a thick sequence of marine and nonmarine sedimentary rocks on the basement igneous rocks of the Southern California Batholith and metamorphic rocks.

Gradual emergence of the region from the sea occurred in Pleistocene time, and numerous wave-cut platforms, most of which were covered by relatively thin marine and nonmarine terrace deposits, formed as the sea receded from the land. Accelerated fluvial erosion during periods of heavy rainfall, along with the lowering of base sea level during Quaternary times, resulted in the rolling hills, mesas and deeply incised canyons which characterize the landforms in western San Diego County.

### 4.1.2 Site Specific

The site is underlain by a sequence of artificial fill, Younger Alluvium (Qya), Older Alluvium (Qoa), and Jurassic-aged metavolcanic bedrock (Mzu) of the Santiago Peak Volcanics.

The alluvium is comprised of a variety of finer grained sands, silts and clays. These soils are gray, grey-brown, brown and orange in color. Generally, the younger alluvium forms a 5-foot to 8-foot thick cap over the older alluvium.

The metavolcanic bedrock is known as the Santiago Peak Volcanics. NOVA anticipates that the upper portions of the bedrock may be weathered, with the rock becoming denser with depth. Refusal to the hollow stem auger drilling tools on hard, dense rock occurred in the borings at depths of 16 feet to 42 feet below ground surface.

Figure 4-1(following page) reproduces geologic mapping of the near surface geology in the vicinity of the site. As may be seen by review of this mapping, younger alluvium (Qya) is mapped as the surface shall geologic unit in the site area. The metavolcanic bedrock rises to the surface just north and east of the site. This bedrock is buried beneath the above- described alluvial sequence within the site limits.

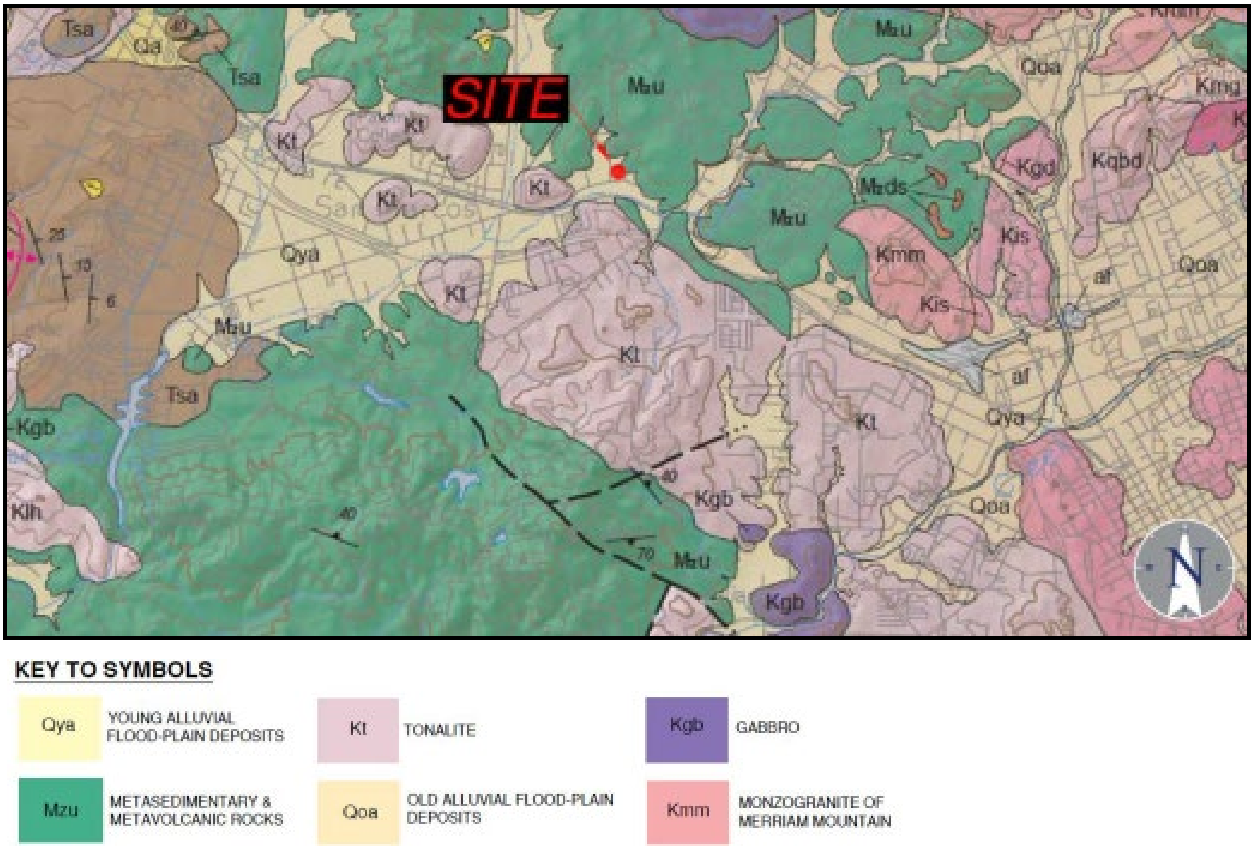


**Figure 4-1. Geologic Mapping of the Site Vicinity**

## 4.2 Site Conditions

### 4.2.1 Surface

The developed site area is relatively level, covered with structures and pavements. Existing site elevations range from about +610 feet mean sea level (msl) at the southwest corner at Mulberry Drive, to about +630 msl in the northeast corner. This 20 foot elevation differential occurs over a distance of about 800 feet, a surface gradient of about 3.5%.

### 4.2.2 Subsurface

The borings indicate that the site is underlain by a sequence of fill and naturally occurring soils that may be characterized for the purposes of this report as below.

The subsurface disclosed by the engineering borings may be generalized to occur as follows:

1. Unit 1, Fill. Fill was encountered in B-3 to a depth of 5.5 feet bgs. The fill is comprised of sandy silt of firm to stiff consistency. Because no records exist regarding the placement of this fill, the fill is considered ‘undocumented,’ and at risk for wide variations in quality. Figure 4-2 (following page) depicts a sample of this soil unit.

**Figure 4-2. Unit 1 Undocumented Clayey Fill**

2. Unit 2, Alluvium. This unit is comprised of a thin strata of Younger Alluvium (Qya) that extends to about 7.5 feet bgs, below which occur deposits of Older Alluvium (Qoa) to depths of up to 41 feet bgs. The younger alluvium is brown to dark brown, characteristically silty and clayey, of stiff consistency. The older alluvium is comprised of reddish silts, silty sands and silty clays.

   Figure 4-3 (following page) depicts the alluvium.

3. Unit 3, Metavolcanics. All of the borings encountered hard metavolcanic rock beneath the alluvium. Drilling met refusal on this unit when encountered, and few samples could be recovered.

**Figure 4-3. Unit 2 Older Alluvium (Qoa)**

### 4.2.3 Groundwater

Static

Groundwater was encountered in the borings at depths of 4 feet to 8 feet below ground surface, about El +606 to +608 feet msl.

Perched

Infiltrating storm water from prolonged wet periods can 'perch' atop localized zones of lower permeability soil that exist above the static groundwater level. Localized perched groundwater conditions may also develop once development completes and landscape irrigation commences.

No perched groundwater was observed during the work reported herein.

### 4.2.4 Surface Water

No surface water was evident on the site at the time of NOVA's subsurface exploration. NOVA did not observe any visual evidence of seeps, springs, erosion, staining, discoloration, etc. that would indicate recent problems with surface water.

# 5.0 REVIEW OF GEOLOGIC, SOIL AND SITING HAZARDS

## 5.1 Overview

This section provides a review of geologic, soil and siting-related hazards common to this region of California, considering each for its potential to affect the planned development. The primary hazard identified by this review is the risk for moderate-to-severe ground shaking in response to a large-magnitude earthquake during the lifetime of the planned development.

While there is no risk of liquefaction or related seismic phenomena, strong ground motion could affect the site. This circumstance is common to all civil works in this area of California.

The following subsections describe NOVA's review of soil and geologic hazards.

## 5.2 Geologic Hazards

### 5.2.1 Strong Ground Motion

The site is not located within a currently designated Alquist-Priolo Earthquake Zone. No known active faults are mapped on the site area. The nearest known active faults are within the Oceanside section of the Newport-Inglewood-Rose Canyon fault system, aligned offshore approximately 13.3 miles west of the site. This system has the potential to be a source of strong ground motion.

The seismicity of the site was evaluated utilizing a web-based analytical tool provided by the The American Society of Civil Engineers (ASCE). This evaluation shows the site may be subjected to a Magnitude 7 seismic event, with a corresponding risk-based Peak Ground Acceleration ($PGA_M$) of $PGA_M \sim 0.43$ g.

### 5.2.2 Fault Rupture

No evidence of faulting was observed during NOVA's geologic reconnaissance of the site. No faulting is otherwise mapped within a mile of the site. Because of the lack of known active faults on the site, the potential for surface rupture at the site is considered low. Shallow ground rupture due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

As is discussed above, the nearest known active fault is the Newport-Inglewood-Rose Canyon fault system, more than 13 miles west of the site. Figure 5-1 (following page) reproduces published mapping of active faulting in the site vicinity


**Figure 5-1. Active Faulting in the Site Vicinity**

### 5.2.3 Landslide

As used herein, 'landslide' describes downslope displacement of a mass of rock, soil, and/or debris by sliding, flowing, or falling. Such mass earth movements are greater than about 10 feet thick and larger than 300 feet across. Landslides typically include cohesive block glides and disrupted slumps that are formed by translation or rotation of the slope materials along one or more slip surfaces.

The causes of classic landslides start with a preexisting condition- characteristically, a plane of weak soil or rock- inherent within the rock or soil mass. Thereafter, movement may be precipitated by earthquakes, wet weather, and changes to the structure or loading conditions on a slope (e.g., by erosion, cutting, filling, release of water from broken pipes, etc.).

Associated with this assessment, NOVA completed review of published information regarding historical landslides and the risk of landsliding in the site vicinity. Figure 5-2 (following page) reproduces the findings of that work, from which it can be seen that (i) the site itself is marginally susceptible to landsliding; and, (ii) there are no mapped historic landslides in the site area.

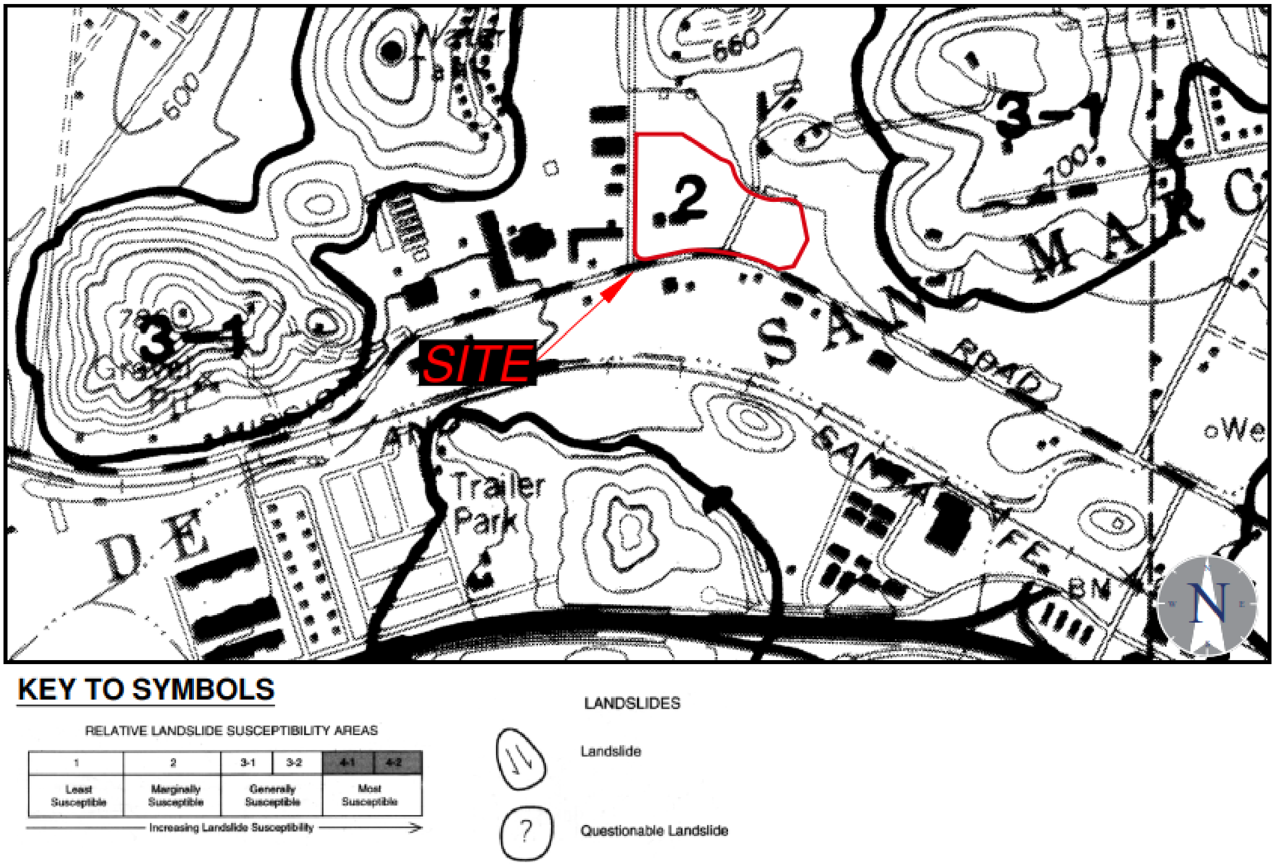


**Figure 5-2. Landslide Risk in the Site Area**
(source: adapted from Tan 1995)

In consideration of the level ground at and around the site, review of published information regarding NOVA considers the landslide hazard at the site to be 'negligible' for the site and the surrounding area.

## 5.3 Soil Hazards

### 5.3.1 Embankment Stability

As used herein, 'embankment stability' is intended to mean the safety of localized natural or man-made embankments against failure. Unlike landslides described above, embankment stability can include smaller scale slope failures such as erosion-related washouts and more subtle, less evident processes such as soil creep.

No new slopes are planned as part of the future site development. There are no existing slopes on the site. There is no concern regarding embankment stability at this site.

### 5.3.2 Seismic

<u>Liquefaction</u>

'Liquefaction' refers to the loss of soil strength during a seismic event. The phenomenon is observed in areas that include geologically 'younger' soils (i.e., soils of Holocene age), shallow water table (less than about 60 feet depth), and cohesionless (i.e., sandy and silty) soils of looser consistency. The seismic ground motions increase soil water pressures, decreasing grain-to-grain contact among the soil particles, which causes the soils to lose strength.

Resistance of a soil mass to liquefaction increases with increasing density, plasticity (associated with clay-sized particles), geologic age, cementation, and stress history. The cemented, very dense and geologically 'older' subsurface units at this site have no potential for liquefaction.

Seismically Induced Settlement

Apart from liquefaction, a strong seismic event can induce settlement within loose to moderately dense, unsaturated granular soils. The soils of Unit 2 are sufficiently cemented and dense that these soils will not be prone to seismic settlement.

Lateral Spreading

Lateral spreading is a phenomenon in which large blocks of intact, non-liquefied soil move downslope on a liquefied soil layer. Lateral spreading is often a regional event. For lateral spreading to occur, a liquefiable soil zone must be laterally continuous and unconstrained, free to move along sloping ground. Due to the absence of a potential for liquefaction and relatively flat surrounding topography, there is no potential for lateral spreading.

### 5.3.3 Expansive Soil

Expansive soils are characterized by their ability to undergo significant volume changes (shrinking or swelling) due to variations in moisture content¸ the magnitude of which is related to both clay content and plasticity index. These volume changes can be damaging to structures. Nationally, the annual value of real estate damage caused by expansive soils is exceeded only by that caused by termites.

As is discussed in Section 3, the soils have been characterized by testing to determine Expansion Index ('EI' after ASTM D 4829). Originally developed in Orange County in the 1960s, EI is a basic soil index property, comparable to indices such as the Atterberg limits of soils. EI is adopted by the 2016 California Building Code ('CBC', Section 1803.5.3) for characterization of expansive soils. The listing below tabulates the qualitative descriptors of expansion potential based upon EI.

**Table 5-1. Qualitative Descriptors Of Expansion Potential Based Upon EI**

| Expansion Index ('EI'), ASTM D 4829 | Expansion Potential, ASTM D 4829 | Expansion Classification, 2016 CBC |
|---|---|---|
| 0 to 20 | Very Low | Non-Expansive |
| 21 to 50 | Low | Expansive |
| 51 to 90 | Medium | |
| 91 to 130 | High | |
| >130 | Very high | |

Testing of the Unit 2 younger alluvium, as well as visual inspection of samples recovered by NOVA, indicates that this soil has 'Medium' expansion potential.

#### 5.3.4 Hydro-Collapsible Soils

Hydro-collapsible soils are common in the arid climates of the western United States in specific depositional environments- principally, in areas of young alluvial fans, debris flow sediments, and loess (wind-blown sediment) deposits. These soils are characterized by low *in situ* density, low moisture contents, and relatively high unwetted strength.

The consistency and geologic age of the Unit 1 and Unit 2 soils are such that these soils are not potentially hydro-collapsible.

#### 5.3.5 Undocumented Fill

Records are not available regarding the placement of the Unit 1 fill, such that this fill is considered 'undocumented,' subject to wide variations in quality and potentially compressible.

Section 6 discusses design to adapt to the undocumented fill.

#### 5.3.6 Corrosive Soils

Chemical testing of the near-surface soils indicates the soils contain low concentrations of soluble sulfates and chlorides. Section 6 addresses this consideration in more detail.

### 5.4 Siting Hazards

#### 5.4.1 Effect on Adjacent Properties

The proposed project will not affect the structural integrity of adjacent properties or existing public improvements and street right-of-ways located adjacent to the site if the recommendations of this report are incorporated into project design.

#### 5.4.2 Flood

The site is located within a FEMA-designated flood zone, Flood Map No. 06073C0794G dated May 16, 2012. The site area is designated "Zone X," an area of minimal flood hazard.

Figure 5-3 reproduces flood mapping by FEMA of the site area.

#### 5.4.3 Tsunami and Seiche

Tsunami describes a series of fast-moving, long period ocean waves caused by earthquakes or volcanic eruptions. The altitude and distance of the site from the ocean preclude this threat.

Seiches are standing waves that develop in an enclosed or partially enclosed body of water such as lakes or reservoirs. Harbors or inlets can also develop seiches.

The site is not located near a body of water that could generate a seiche.


**Figure 5-3. Flood Mapping of the Site Area**
(source: adapted from FEMA Flood Map 06073C0794G, Revised May 16, 2012)

# 6.0 EARTHWORK AND FOUNDATIONS

## 6.1 Overview

### 6.1.1 Review of Site Hazards

Section 5 provides review of geologic, soil and siting-related hazards that may affect the planned development. The primary hazard identified by that review is that the site is at risk for moderate-to-severe ground shaking in response to large-magnitude earthquakes during the lifetime of the planned development.

While there is no risk of liquefaction or seismic phenomena related to liquefaction, strong ground motion could affect the site. This circumstance is common to all civil works in this area of California. Section 6.2 provides seismic design parameters.

### 6.1.2 Foundation System

Based upon the indications of the field and laboratory data developed for this work, it is the opinion of NOVA that the site is suitable for development of the planned structures on shallow foundations provided the geotechnical recommendations described herein are followed. Founded as such, the project will not affect the structural integrity of adjacent properties or existing public improvements and street right-of-ways located adjacent to the site.

### 6.1.3 Review and Surveillance

The subsections following provide geotechnical recommendations for the planned development as it is now understood. It is intended that these recommendations provide sufficient geotechnical information to develop the project in general accordance with 2016 California Building Code (CBC) requirements.

NOVA should be given the opportunity to review the grading plan, foundation plan, and geotechnical-related specifications as they become available to confirm that the recommendations presented in this report have been incorporated into the plans prepared for the project.

All earthwork related to site and foundation preparation should be completed under the observation of NOVA.

## 6.2 Seismic Design Parameters

### 6.2.1 Site Class

The Site Class was determined using site-specific boring data and geologic knowledge, with reference to ASCE 7-10, Table 20.3-1. Based on this information, the site is classified as Site Class C per ASCE 7-10, Table 20.3-1.

### 6.2.2 Seismic Design Parameters

Table 6-1 (following page) provides seismic design parameters for the site in accordance with 2016 CBC and mapped spectral acceleration parameters.

**Table 6-1. Seismic Design Parameters, ASCE 7-10**

| Parameter | Value |
|---|---|
| Site Soil Class | C |
| Site Latitude (decimal degrees) | 32.82217 |
| Site Longitude (decimal degrees) | -117.13107 |
| Site Coefficient, $F_a$ | 1.003 |
| Site Coefficient, $F_v$ | 1.420 |
| Mapped Short Period Spectral Acceleration, $S_S$ | 0.992 |
| Mapped One-Second Period Spectral Acceleration, $S_1$ | 0.380 |
| Short Period Spectral Acceleration Adjusted For Site Class, $S_{MS}$ | 0.995 |
| One-Second Period Spectral Acceleration Adjusted For Site Class, $S_{M1}$ | 0.540 |
| Design Short Period Spectral Acceleration, $S_{DS}$ | 0.663 |
| Design One-Second Period Spectral Acceleration, $S_{D1}$ | 0.360 |

Source: *ASCE 7 Hazard Tool*, found at https://asce7hazardtool.online/

## 6.3 Corrosivity and Sulfates

### 6.3.1 General

Electrical resistivity, chloride content, and pH level are all indicators of the soil's tendency to corrode ferrous metals or to attack to embedded concrete. Chemical testing was performed on a representative sample of the near surface soils. The results of the testing are tabulated in Table 6-2.

**Table 6-2. Summary of Corrosivity Testing of the Near Surface Soil**

| Parameter | Units | Value |
|---|---|---|
| pH | standard unit | 8.6 |
| Resistivity | Ohm-cm | 1,000 |
| Water Soluble Chloride | ppm | 75 |
| Water Soluble Sulfate | ppm | 93 |

### 6.3.2 Metals

Caltrans considers a soil to be corrosive if one or more of the following conditions exist for representative soil and/or water samples taken at the site:

- chloride concentration is 500 parts per million (ppm) or greater;
- sulfate concentration is 2,000 ppm (0.2%) or greater; or,
- the pH is 5.5 or less.

Based on the Caltrans criteria, the on-site soils would not be considered 'corrosive' to buried metals.

In addition to the above parameters, the risk of soil corrosivity buried metals is considered by determination of electrical resistivity (ρ). Soil resistivity may be used to express the corrosivity of soil only in unsaturated soils. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of DC electrical current from the metal into the soil. As the resistivity of the soil decreases, the corrosivity generally increases. A common qualitative correlation (cited in Romanoff 1989, NACE 2007) between soil resistivity and corrosivity to ferrous metals is tabulated below.

**Table 6-3. Soil Resistivity and Corrosion Potential**

| **Minimum Soil Resistivity** (Ω-cm) | **Qualitative Corrosion Potential** |
|---|---|
| 0 to 2,000 | Severe |
| 2,000 to 10,000 | Moderate |
| 10,000 to 30,000 | Mild |
| Over 30,000 | Not Likely |

Despite the relatively benign environment for corrosivity indicated by pH and water-soluble chlorides, the resistivity testing suggests that design should consider that the soils may be moderately corrosive to embedded ferrous metals.

Typical recommendations for mitigation of such corrosion potential in embedded ferrous metals include:

- a high-quality protective coating such as an 18-mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar;
- electrical isolation from above grade ferrous metals and other dissimilar metals by means of dielectric fittings in utilities and exposed metal structures breaking grade; and,
- steel and wire reinforcement within concrete having contact with the site soils should have at least 2 inches of concrete cover.

If extremely sensitive ferrous metals are expected to be placed in contact with the site soils, it may be desirable to consult a corrosion specialist regarding choosing the construction materials and/or protection design for the objects of concern.

#### 6.3.3 Sulfate Attack

As shown in Table 6-2, the soil sample indicated water-soluble sulfate ($SO_4$) content of 93 parts per million (‘ppm,’ 0.009% by weight). With $SO_4 < 0.10$ percent by weight, the American Concrete Institute (ACI) 318-08 considers a soil to have no potential (S0) for sulfate attack.

Table 6-4 (following page) reproduces the Exposure Categories considered by ACI.

**Table 6-4. Exposure Categories and Requirements for Water-Soluble Sulfates**

| Exposure Category | Class | Water-Soluble Sulfate ($SO_4$) In Soil (percent by weight) | Cement Type (ASTM C150) | Max Water-Cement Ratio | Min. $f'_c$ (psi) |
|---|---|---|---|---|---|
| Not Applicable | S0 | $SO_4 < 0.10$ | - | - | - |
| Moderate | S1 | $0.10 \leq SO_4 < 0.20$ | II | 0.50 | 4,000 |
| Severe | S2 | $0.20 \leq SO_4 \leq 2.00$ | V | 0.45 | 4,500 |
| Very severe | S3 | $SO_4 > 2.0$ | V + pozzolan | 0.45 | 4,500 |

Adapted from: ACI 318-08, Building Code Requirements for Structural Concrete

6.3.4 Limitations

Testing to determine several chemical parameters that indicate a potential for soils to be corrosive to construction materials are traditionally completed by the Geotechnical Engineer, comparing testing results with a variety of indices regarding corrosion potential.

Like most geotechnical consultants, NOVA does not practice in the field of corrosion protection, since this is not specifically a geotechnical issue. Should you require more information, a specialty corrosion consultant should be retained to address these issues.

## 6.4 Earthwork

6.4.1 General

As is noted in Section 2, no detailed structural or civil- related design information is available at this time. However, based upon the known condition of the site and the design concept that is currently considered, NOVA expects that earthwork will be limited to (i) preparation of building pads; and, (ii) excavations for foundations and utilities.

Earthwork should be performed in accordance with Section 300 of the most recent approved edition of the "*Standard Specifications for Public Works Construction*" and "*Regional Supplement Amendments*."

6.4.2 Site Preparation

Prior to the start of earthwork, the site should be cleared of structures and existing pavement. The deleterious materials should be disposed of in approved off-site locations.

At the outset of site work, the Contractor should establish Construction BMPs to prevent erosion of graded/excavated areas until such time as permanent drainage and erosion control measures have been installed. Any existing utilities which are to be abandoned should either be (i) excavated and the trenches backfilled; or, (ii) the lines completely filled with sand-cement slurry.

6.4.3 Select Fill

Material Requirements

Any fill used to support structures should be 'select.' Select Fill should be a mineral soil free of organics with the characteristics listed below:

- free of organics, with at least 40 percent by weight finer than ¼-inches in size and,
- maximum particle size of 3 inches; and,
- expansion index (EI) less than 50 (i.e., EI < 50, after ASTM D 4829).

Compaction Requirements

All fill should be compacted to a minimum of 90 percent relative compaction after ASTM D1557 (the 'modified Proctor') following moisture conditioning to at least 2% above the optimum moisture content.

Fill should be placed in loose lifts no thicker than the ability of the compaction equipment to thoroughly densify the lift. For most self-propelled construction equipment, this will limit loose lifts to on the order of 10-inches or less. Lift thickness for hand-operated equipment (tampers, walked behind compactors, etc.) will be limited to on the order of 4 inches or less.

### 6.4.4 Excavation Characteristics

The Unit 1 fill and Unit 2 alluvium will be readily excavated by earthwork equipment usual for construction of this nature.

### 6.4.5 Remedial Grading

General

Earthwork operations should provide for removal of the Unit 1 fill and Unit 2 alluvial soils to competent soils, or to a depth of approximately 3 feet below the bottom of new foundations. The exposed bottom of removals should be scarified, moisture conditioned and compacted to a minimum of 90% relative compaction after ASTM D 1557 (the 'modified Proctor'). The resultant excavation should be backfilled with soils that meet the Select Fill criteria of Section 6.4.3. Due to their clayey and expansive nature, some of the Unit 2 soils may not be suitable for reuse within the upper three feet beneath the bottom of foundations. As an alternative, a controlled low strength material ('CLSM', sometimes referenced as 'flowable fill') can be used. NOVA should evaluate materials to be used as fill prior to placement or importing.

Remedial grading that encroaches upon existing buildings may be performed in slot cuts. The slot cuts should be 10 feet wide or less, as measured along the length of the existing building. Slots should be completely backfilled prior to opening an adjoining section. The process of slot cutting may include alternating a 10-0foot wide section of the slot cut adjacent to a 10-foot wide intact and un-excavated section of the existing surface. It should be noted that the width of the recommended slot cuts may be revised and reduced by NOVA based upon the conditions exposed during construction.

Stabilization Contingency

It is possible that removals may be associated with the exposure of wet soils at the bottom of the excavations. Construction should plan for the contingency that in certain instances the near surface saturated soils or areas of purchase, seeping water may require use of ground stabilization to provide a base for subsequent backfilling. In such instances these areas may be stabilized by use of 12 inches of ¾ inch crushed rock or aggregate based placed over a biaxial geo-grid such as

Tensar BX 1100, or equivalent. The crushed rock should be covered with a segregation geotextile-a nonwoven fabric such as Mirafi 140N, or equivalent.

CLSM

Over excavated areas or other excavations can be backfilled up to the bottom of the design footing elevation with a CLSM that develops a minimum unconfined compressive strength of 40 psi. A two sack slurry mix should meet this criterion.

If employed, the CLSM should conform to material requirements identified in Section 19-3 of the Caltrans Standard Specifications (latest edition). The Caltrans specification for the gradation of CLSM aggregate is reproduced on below as Table 6-5.

**Table 6-5. Gradation for CLSM Fill Aggregate**

| U.S. Standard Sieve Size | Percent Passing by Weight, |
|---|---|
| 1½ inch | 100 |
| 1 inch | 80 to 100 |
| ¾ inch | 60 to 100 |
| 3/8 inch | 50 to 100 |
| No. 4 | 40 to 80 |
| No. 8 | 10 to 40 |

Source: Caltrans 2015, Section 19-3.02G

#### 6.4.6 Maintenance of Moisture in Soils During Construction

The subgrade moisture condition of the building pad and foundation soils must be maintained at least 2% above optimum moisture content up to the time of concrete.

#### 6.4.7 Trenching and Backfilling for Utilities

Excavation for utility trenches must be performed in conformance with OSHA regulations contained in 29 CFR Part 1926.

Utility trench excavations have the potential to degrade the properties of the adjacent soils. Utility trench walls that are allowed to move laterally will reduce the bearing capacity and increase settlement of adjacent footings and overlying slabs.

Backfill for utility trenches is as important as the original subgrade preparation or engineered fill placed to support either a foundation or slab. Backfill for utility trenches must be placed to meet the project specifications for the engineered fill of this project. Unless otherwise specified, the backfill for the utility trenches should be placed in 4 to 6 inch loose lifts and compacted to a minimum of 90 percent relative compaction after ASTM D 1557 (the ‘modified Proctor’) at soil moisture at least +2 percent of the optimum moisture content. Up to 4 inches of bedding material placed directly under the pipes or conduits placed in the utility trench can be compacted to 90 percent relative compaction with respect to the Modified Proctor.

6.4.8 Flatwork

Prior to casting exterior flatwork, the upper 12" of subgrade soils should be removed and replaced with "Select" fill, moisture conditioned and recompacted, as recommended in Section 6.4.4. Concrete slabs for pedestrian traffic should be at least four (4) inches thick.

## 6.5 Shallow Foundations

6.5.1 General

6.5.2 Settlement of Footings

The stiffer subsurface is favorable for development of shallow foundations for support of light to moderate loads. Figure 6-1 (following page) depicts the estimated performance of a 6 foot x 6 foot footing bearing at 3 kips per square foot on the near surface alluvial soils. As may be seen by review of this graphic, settlement of this footing will be low.

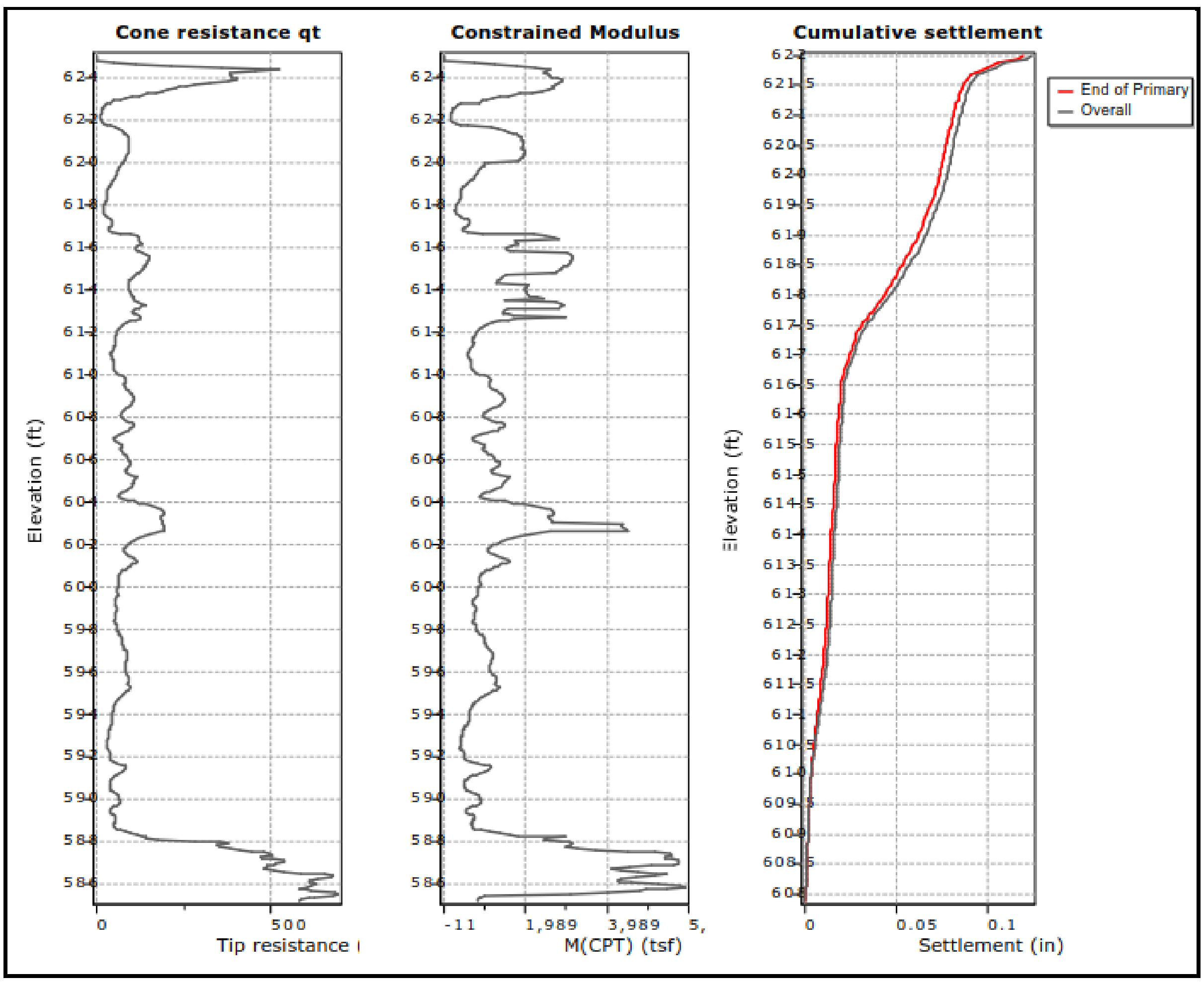


**Figure 6-1. Settlement of a 6-Foot Square footing Bearing at 3 ksf**

The following subsections detail recommendations for shallow foundations.

### 6.5.3 Isolated and Continuous Foundations

Isolated and continuous footings, may be employed as described below.

Isolated Foundations
Isolated foundations for interior columns may be designed for an allowable contact stress of 3,500 psf. This value may be increased by one-third for transient loads such as wind and seismic. These foundation units should have a minimum width of 30 inches, embedded a minimum of 24 inches below lowest adjacent grade, including a minimum embedment of 12 inches into sound Unit 2 alluvium.

Continuous Foundations
Continuous foundations may be designed for an allowable contact stress of 3,000 psf, for footings with a minimum of 18 inches in width and embedded 24 inches below lowest adjacent grade with an overall minimum embedment of 12 inches into sound Unit 2 sandstone. This bearing value may be increased by one-third for transient loads such as wind and seismic.

Resistance to Lateral Loads
Lateral loads to shallow foundations cast 'neat' against Unit 2 alluvium may be resisted by passive earth pressure against the face of the footing, calculated as a fluid density of 250 psf per foot of depth, neglecting the upper 1-foot of soil below surrounding grade in this calculation. If the footing is in the interior of the building, with slab on grade on both sides of the column footing or wall footing, the upper 1-foot should not be neglected. Additionally, a coefficient of friction of 0.35 between soil and the concrete base of the footing may be used with dead loads.

Settlement
Supported as recommended above, the structure will settle on the order of 0.5 inch. This movement will occur elastically, as dead load (DL) and permanent live loads (LL) are applied. In usual circumstance, about 70% of this settlement will occur during the construction period. Angular distortion due to differential settlement of adjacent, unevenly loaded footings should be less than 1 inch in 40 feet (i.e., $\Delta/L$ less than 1:480).

### 6.5.4 Tank Pads

Foundations for tank pads may be developed in a manner similar to that for shallow foundations described above. Ring walls for tanks will behave as continuous foundations and may be designed as described in Section 6.5.3. A mat foundation for tank pad will behave in a manner similar to that for an isolated foundation and may be designed as described in Section 6.5.3.

The lateral resistance for tank pads will develop in a manner similar to that for other shallow foundations and should be calculated as described in Section 6.5.3.

### 6.5.5 Conventionally Reinforced Concrete Slab

Modulus of Subgrade Reaction

The ground level of the structures may employ conventional on-grade (ground-supported) slab designed using a modulus of subgrade reaction (k) of 90 pounds per cubic inch (i.e., k = 90 pci).

The recommended value of 90 pci reflects judgment by NOVA regarding the expected soil-slab interaction for a variety of interior slab sizes. NOVA recognizes that this value is low relative to that often recommended for slabs in this region. The value reflects guidance for estimation of k for clayey soils published in '*Evaluation of Coefficients of Subgrade Reaction*' [1] and adapted by NAVFAC and other design standard guidance.

The referenced guidance correlates estimates of k for foundation design with that the values for $k_1$ developed from plate load tests on a variety of soils, calculating $k$ for the foundation soil mass ($k_s$) using the expression below. The modulus of subgrade reaction is not a property of a soil, as its value is also a function of the foundation size. The expression has the effect of reducing the modulus of subgrade reaction as the footing size increases. The recommended values of $k_1$ for clays that are provided in the above-cited reference are summarized on the table below.

**Table 6-6. Values of $k_1$ for 1 ft² Plates Bearing on Clay (lbs/in³)**

| Parameter | Stiff | Very Stiff | Hard |
|---|---|---|---|
| Range of $k_1$, square plates | 55 - 115 | 115 – 230 | > 230 |
| Proposed values, square plates | 85 | 170 | 350 |

The reference recommends that the values for $k_1$ on Table 6-6 be reduced to account for the size of the loaded area for foundations other than square, calculating $k$ for the foundation soil mass ($k_s$) using the expression below.

$$k_1 = k_1\left(\frac{1+0.5}{1.5\,l}\right)$$

where,

$l$ = ratio of the slab length to the slab width

$k_1$ = the modulus for a 1 ft.² plate from Table 6-6

Slab Thickness

The actual slab thickness and reinforcement should be designed by the Structural Engineer. NOVA recommends the slab be a minimum 5 inches thick, reinforced by at least #3 bars placed at 16 inches on center each way within the middle third of the slabs by supporting the steel on chairs or concrete blocks ("dobies").

Crack Control

Minor cracking of concrete after curing due to drying and shrinkage is normal. Cracking is aggravated by a variety of factors, including high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due during curing.

[1] Terzaghi, Karl, *Evaluating Coefficients of Subgrade Reaction,* Geotechnique, Vol 5, 1955, pp 297-326. See also, Department of the Navy, NAVFAC DM 7.01, *Soil Mechanics,* Washington DC, September 1986, Fig 6, p 7.1-219.

The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or 'weakened plane' joints at frequent intervals. Joints should be laid out to form approximately square panels and never exceeding a length to width ratio of 1.5 to 1. Proper joint spacing and depth are essential to effective control of random cracking. Joints are commonly spaced at distances equal to 24 to 30 times the slab thickness. Joint spacing that is greater than 15 feet should include the use of load transfer devices (dowels or diamond plates). Contraction/ control joints should be established to a depth of ¼ the slab thickness as depicted in Figure 6-2.

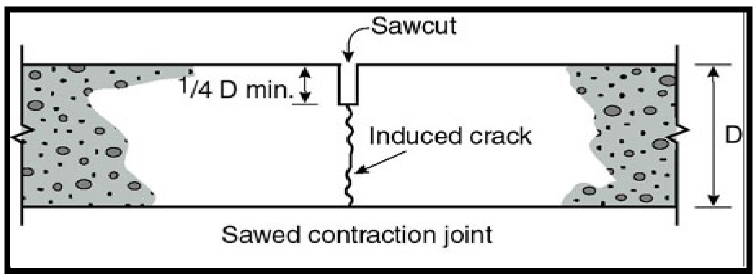


**Figure 6-2. Sawed Contraction Joint**

## 6.6 Underslab Vapor Retarder

### 6.6.1 General

Soil moisture vapor that penetrates ground-supported concrete slabs can result in damage to moisture-sensitive floors, some floor sealers, or sensitive equipment in direct contact with the floor. It is not the responsibility of the geotechnical consultant to provide recommendations for vapor retarders to address this concern. This responsibility usually falls to the Architect. Decisions regarding the appropriate vapor retarder are principally driven by the nature of the building space above the slab, floor coverings, anticipated penetrations, concerns for mold or soil gas, and a variety of other environmental, aesthetic and materials factors known only to the Architect.

A variety of specialty polyethylene (polyolefin)-based vapor retarding products are available to retard moisture transmission into and through concrete slabs. This remainder of this section provides an overview of design and installation guidance, and considers the use of vapor retarders in the building construction in the San Diego area.

### 6.6.2 Guidance Documentation

Detail to support selection of vapor retarders and to address the issue of moisture transmission into and through concrete slabs is provided in a variety of publications by the American Society for Testing and Materials (ASTM) and the American Concrete Institute (ACI). A partial listing of those publications is provided below.

- ASTM E1745-97 (2009). *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*

- ASTM E154-88 (2005). *Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Walls, or as Ground Cover*

- ASTM E96-95 (2005). *Standard Test Methods for Water Vapor Transmission of Materials*

- ASTM E1643-98 (2009). *Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs*

- ACI 302.2R-06. *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials*

#### 6.6.3 Design

Vapor retarders employed for ground supported slabs in the San Diego are commonly specified as minimum 10 mil polyolefin plastic that conforms to the requirements of ASTM E1745 as a Class A vapor retarder (i.e., a maximum vapor permeance of 0.1 perms, minimum 45 lb/in tensile strength and 2,200 grams puncture resistance). Among the commercial products that meet this requirement are the series of Yellow Guard® vapor retarders vended by Poly-America, L.P.; the Perminator® products by W. R. Meadows; and, Stego®Wrap products by Stego Industries, LLC.

The person responsible for design of the vapor barrier should consult with product vendors to ensure selection of the vapor retarder that best meets the project requirements. For example, concrete slabs with particularly sensitive floor coverings may require lower permeance or other performance-related factors than are specified by the ASTM E1745 class rating.

#### 6.6.4 Installation

The performance of vapor retarders is particularly sensitive to the quality of installation. Installation should be performed in accordance with the vendor's recommendations under full-time surveillance.

### 6.7 Control of Moisture Around Foundations

#### 6.7.1 General

Design for the structure should include care to control accumulations of moisture around and below foundations. Such design will require coordination from among the Design Team; at a minimum to include the Architect, the Civil Engineer, and the Landscape Architect.

#### 6.7.2 Erosion and Moisture Control During Construction

Surface water should be controlled during construction, via berms, gravel/sandbags, silt fences, straw wattles, siltation basins, positive surface grades, or other methods to avoid damage to the finish work or adjoining properties. The Contractor should take measures to prevent erosion of graded areas until such time as permanent drainage and erosion control measures have been installed. After grading, all excavated surfaces should exhibit positive drainage and eliminate areas where water might pond.

#### 6.7.3 Design

Civil, structural, architectural and landscaping design for the areas around foundations should be undertaken with a view to the maintenance of an environment that encourages constant moisture conditions in the foundation soils following construction. Roof and surface drainage, landscaping, and utility connections should be designed to limit the potential for infiltration and/or releases of moisture beneath structures. In particular, rainfall to roofs should be collected in gutters and discharged in a controlled manner through downspouts designed to drain away from foundations. Downspouts, roof drains or scuppers should discharge into splash blocks to slabs or paving sloped away from buildings.

Proper surface drainage will be required to minimize the potential of water seeking the level of the bearing soils under foundations and pavements. In areas where sidewalks or paving do not immediately adjoin the structure, protective slopes should be provided with a minimum grade (away from the structure) of approximately 3 percent for at least 5 feet from perimeter walls. A minimum gradient of 1 percent is recommended in hardscape areas. Drainage should be directed to approved drainage facilities.

### 6.7.4 Utilities

Design for Differential Movement

Underground piping within or near structures should be designed with flexible couplings to accommodate both ground and slab movement so that minor deviations in alignment do not result in breakage or distress. Utility knockouts should be oversized to accommodate the potential for differential movement between foundations and the surrounding soil.

Backfill Above Utilities.

Excavations for utility lines, which extend under or near structural areas should be properly backfilled and compacted. Utilities should be bedded and backfilled with approved granular soil to a depth of at least one foot over the pipe. This backfill should be uniformly watered and compacted to a firm condition for pipe support. Backfill above the pipe zone should meet the requirements for Select Fill, placed to at least 90% relative compaction at 2% above optimum.

## 6.8 Deep Foundations

### 6.8.1 General

Heavier column loads and uplift loads the structures may be accommodated by the use of deep foundations. A variety of driven and drilled piles are available in this regard.

A variety of deep foundation options are available. The selection of a preferred deep foundation should be based on expected performance and cost. Based upon successful previous experience in the San Diego area, NOVA expects that auger drilled cast-in-place piles (‘ACIPs’ or ‘auger cast piles,’ or ‘CFA piles’) will prove most economical on a basis of cost and performance. The most ubiquitous deep foundation type in the San Diego area, ACIPs have the advantages of being relatively economical to install, will develop higher capacities in the Unit 3 granitics, and will be easily adaptable to changing site conditions. Very significantly, these piles are not associated with ground vibrations that are common to many pile types, making these piles attractive for use within the near proximity of existing structures.

Figure 6-3 (following page) describes NOVA’s the axial capacity of a 24 inch diameter drilled pile, depicting the accumulation of pile capacity with depth below ground surface. As may be seen by review of this graphic, substantial deep foundation capacity is available at this site.

Piles should be drilled/embedded through the Unit 1 and Unit 2 soils, extending at least 5 pile diameters into sound Unit 3 granitics. Based on the indications of the field exploration, Unit 3 will be encountered within about 10 feet below existing site grades.

### 6.8.2 Axial Capacity

Table 6-7 provides unit allowable end bearing and soil friction values for determination of the vertical capacities of deep foundations.

**Table 6-7. Unit Pile Capacities Note 1**

| Soil Unit | Description | Allowable Unit Pile Capacities (psf) | | |
|---|---|---|---|---|
| | | Side Note 2 (compression) | Tip Note 3 (compression) | Side Note 2 (uplift) |
| 1,2 | Fill/Alluvium | 150 | 0 | 50 |
| 3 | Volcanics | 400 Note 4 | 8,000 Note 4 | 350 |

Note 1: capacities on this table may be increased by 1/3 for transitory loads such as wind and seismic
Note 2: calculations of side resistance should ignore the contribution of the upper 2.5 pile diameters
Note 3: all piles should be embedded a minimum of 5 diameters into Unit 3
Note 4: unit value assumes a minimum embedment of 5 pile diameters into Unit 3
Note 5: allowable capacities provide for a factor of safety (F) of F= 2.5 in compression

### 6.8.3 Lateral Response

The lateral response of piles embedded in Unit 1 fill and Unit 2 alluvium fill should assume k = 130 pounds per cubic inch (pci) for the Unit 1 Fill/Alluvium a value characteristic of a medium dense sandy soil. The lateral resistance to piles embedded in the Unit 3 volcanics may be calculated assuming k = 300 pci. NOVA recommends use of these values in calculations of the lateral response of drilled piles to shear and moment.

Estimates of lateral pile analyses have been completed assuming a pile cap/grade beam design that approximates ‘fixed head’ conditions (i.e., the top of the individual pile is allowed to translate but not rotate), with piles spaced at 3 pile diameters center to center. Table 6-8 tabulates lateral capacities for individual pile units, assuming 0.2” as a limiting allowable lateral translation.

**Table 6-8. Lateral Capacity of Individual Drilled Piles, Top Deflection = 0.2 Inch**

| Pile Diameter (inches) | Top Shear (V, kips) | Lateral Deflection at Top (inches) |
|---|---|---|
| 12 | 12 | 0.2 |
| 16 | 20 | 0.2 |
| 24 | 35 | 0.2 |

### 6.8.4 Construction Considerations

Minimum Concrete Strength

Drilled piles should be installed under the observation of the GEOR.

A minimum concrete strength of 4,000 psi is recommended for construction of the drilled piles. Concrete properties are critical in installing piles that will perform satisfactorily. The concrete should include additives that will adequately control setting shrinkage and must be fluid enough to be pumped easily and must flow without excessive pressure losses.

NOVA recommends that at least one set of four ASTM C 31 cylinder specimens be cast per every 50 cubic yards of concrete placed as pile caps, in order to verify achievement of the design compressive strength.

Placing Concrete

Concrete pressure should be monitored during pumping. Concrete should be continuously placed under sufficient head to prevent suction from developing as the augers are withdrawn from the borehole. Suction could cause the soil to mix with the concrete, loss of bearing, or hole collapse. A head of at least 10 feet of concrete above the injection point should be maintained at all times to help prevent collapse.

Auger withdrawal rate should not exceed 10 feet per minute. Sudden pulls of the auger, which may cause "bottlenecking" or collapse of the hole should be avoided. Pile reinforcing may consist of bundled steel rods, rolled steel sections, or reinforcing bar cages as determined by the Structural Engineer and as necessary to satisfy 2016 CBC requirements. All reinforcing should be installed before the concrete sets up, normally within 10 minutes of auger withdrawal, centering the reinforcing steel in the hole with centering devices.

The volume of concrete placed into each pile should be recorded and compared to the theoretical volume of pile by the testing representative. Where the ratio of actual volume to theoretical volume is less than 1.2, the pile will need to be re-drilled unless otherwise directed by the GEOR.

Quality Assurance

The drilled pile installation contractor should be required to use state-of-the-practice construction/ installation techniques to optimize Quality Control (QC) And Quality Assurance (QA).

As an option to evaluate the integrity of constructed piles after installation, access tubes may be cast as part of the drilled pile. Attached to the reinforcing steel, access tubes should consist of 2-inch i.d. or larger Schedule 40 PVC pipes extending the length of the pile. The lower ends of the access tubes should be plugged to keep out concrete/concrete. At the time of construction, the tubes should be filled with water to stabilize the temperature of the pipes to keep from deep bonding from the concrete.

## 6.9 Retaining Walls

### 6.9.1 Wall Loads

Smaller walls may be developed; for example, retaining walls around proposed BMPs. Static lateral earth pressures are provided for these walls on Table 6-9 (following page) as equivalent fluid weights, in psf/foot of wall height or pounds per cubic foot (pcf).

It is expected that cantilevered retaining walls will be less than 8 feet in height. Seismic lateral loads may be ignored for these walls.

**Table 6-9. Lateral Earth Pressures to Retaining Walls**

| Loading Condition | Equivalent Fluid Density (pcf) for Approved Backfill[A, B,C] |
|---|---|
| Active (wall movement allowed) | 35 |
| "At Rest" (no wall movement) | 60 |
| 'Passive" (wall movement toward the soils) | 250 |

Note A: 'approved' means Select Fill with EI < 30 after ASTM D4829 and approved by the Geotechnical Engineer.
Note B: assumes level backfill and appropriate wall drainage.
Note C: The values on Table 6-9 do not contain a factor of safety (F).

#### 6.9.1 Retaining Wall Foundations

Retaining walls may be supported on continuous foundations designed as described in Section 6.5.

#### 6.9.2 Foundation Uplift

A soil unit weight of 125 pcf may be assumed for calculating the weight of soil over the wall footing.

#### 6.9.3 Resistance to Lateral Loads

Lateral loads to wall foundations will be resisted by a combination of frictional and passive resistance as described in Section 6.5.

#### 6.9.4 Wall Drainage

The recommended equivalent fluid pressures provided in the preceding subsection assume that constantly functioning drainage systems are installed between walls and soil backfill to prevent the uncontrolled buildup of hydrostatic pressures and lateral stresses in excess of those stated.

Design for wall drainage may include the use of pre-engineered wall drainage panels or a properly compacted granular free-draining backfill material (EI <30).

The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall.

### 6.10 Temporary Slopes

Temporary slopes may be required for excavations during grading. All temporary excavations should comply with local safety ordinances. The safety of all excavations is solely the responsibility of the Contractor and should be evaluated during construction as the excavation progresses.

Based on the data interpreted from the borings, the design of temporary slopes in Unit 2 may assume California Occupational Safety and Health Administration (Cal/OSHA) Soil Type C for planning purposes. Temporary slopes in the Unit 3 formational soils may be excavated no steeper than ¾: 1 (horizontal: vertical).

# 7.0 STORMWATER INFILTRATION

## 7.1 Overview

Based upon the indications of the field exploration and laboratory testing reported herein, NOVA has evaluated the site as abstracted below after guidance contained in the latest edition of the City of San Marcos BMP Design Manual (hereafter, 'the BMP Manual').

Section 3.4 provides a description of the field work undertaken to complete percolation testing. Figure 3-1 depicts the location of the testing. This section provides the results of that testing and related recommendations for management of stormwater in conformance with the BMP Manual.

As is well-established by the BMP Manual, the feasibility of stormwater infiltration is principally dependent on geotechnical and hydrogeologic conditions at the project site. In consideration of the negligible infiltration rates and the increased risk of geotechnical hazards as a result of stormwater infiltration (see Section 7.2 and Section 7.3), NOVA concludes that the site is not feasible for development of permanent stormwater infiltration BMPs.

This section provides NOVA's assessment of the feasibility of stormwater infiltration BMPs utilizing the information developed by the field exploration described in Section 3, as well as other elements of the site assessment.

## 7.2 Infiltration Rates

### 7.2.1 General

The percolation rate of a soil profile is not the same as its infiltration rate ('I'). Therefore, the measured/calculated field percolation rate was converted to an estimated infiltration rate utilizing the Porchet Method in accordance with guidance contained in the BMP Manual. Table 7-1 provides a summary of the infiltration rates determined by the percolation testing.

**Table 7-1. Infiltration Rates Determined by Percolation Testing**

| Boring | Approximate Ground Elevation (feet, msl) | Depth of Test (feet) | Approximate Test Elevation (feet, msl) | Infiltration Rate (inches/hour) | Design Infiltration Rate (in/hour, F=2*) |
|---|---|---|---|---|---|
| P-1 | +611.3 | 5 | +606.3 | 0.01 | 0.00 |
| P-2 | +616.1 | 5 | +611.1 | 0.02 | 0.01 |

Notes: (1) 'F' indicates 'Factor of Safety' (2) elevations are approximate and should be reviewed

### 7.2.2 Design Infiltration Rate

As may be seen by review of Table 1, a factor of safety (F) is applied to the infiltration rate (I) determined by the percolation testing. This factor of safety, at least F = 2 in local practice, considers the nature and variability of subsurface materials, as well as the natural tendency of infiltration structures to become less efficient with time. The calculated infiltration rates after applying F = 2 are I = 0.00 and I = 0.01 inches per hour for P-1 and P-2, respectively.

## 7.3 Review of Geotechnical Feasibility Criteria

### 7.3.1 Overview

Section C.2 of Appendix C of the BMP Manual provides seven factors that should be considered by the stormwater professional while assessing the feasibility of infiltration related to subsurface conditions. These factors are listed below.

- C.2.1 Soil and Geologic Conditions
- C.2.2 Settlement and Volume Change
- C.2.3 Slope Stability
- C.2.4 Utilities
- C.2.5 Groundwater Mounding
- C.2.6 Retaining Walls and Foundations
- C.2.7 Other Factors

The above feasibility criteria are reviewed in the following subsections

### 7.3.2 Soil and Geologic Conditions

The soil borings, CPT soundings and percolation test borings completed for this assessment disclose the sequence of soil units described below.

1. Unit 1, Fill. Fill was encountered in B-3 at a depth of 5.5 feet bgs. The fill is comprised of sandy silt of firm to stiff consistency. Because no records exist regarding placement of this fill, the fill is considered 'undocumented' and at risk for wide variations in quality.

2. Unit 2, Alluvium. This unit is comprised of a thin strata of Younger Alluvium (Qya) that extends to about 7.5 feet bgs, below which occur deposits of Older Alluvium (Qoa) to depths of up to 41 feet bgs. The younger alluvium is brown to dark brown, characteristically silty and clayey, of stiff consistency. The older alluvium is comprised of reddish silts, silty sands and silty clays.

3. Unit 3, Metavolcanics. Hard metavolcanic rock occurs beneath the alluvium. Drilling met refusal on this unit when encountered, and few samples could be recovered.

### 7.3.3 Settlement and Volume Change

Unit 2 alluvium includes expansive clay with the potential to swell upon wetting and shrink upon drying. Introduction of water to this unit will create damaging foundation movement.

### 7.3.4 Slope Stability

There are no slopes on-site, nor are any soil embankments planned for the new development. As a consequence, BMPs should not be sited within 50 feet of an existing slope.

### 7.3.5 Utilities

Stormwater infiltration BMPs should not be sited within 10 feet of underground utilities.

#### 7.3.6 Groundwater Mounding

In consideration of the low measured percolation rates, it is likely that groundwater mounding will occur if stormwater infiltration is attempted in any scale. Groundwater mounding will effect damaging volume changes to soils, affecting utilities, pavements, flat work, and foundations.

#### 7.3.7 Retaining Walls and Foundations

Stormwater infiltration BMPs should not be sited within 10 feet of retaining walls and foundations.

#### 7.3.8 Other Factors

The location near boring B-3 is overlain by 5.5 feet of Unit 1 undocumented fill. Extension of a BMP to natural soil at this location or other areas of considerable fill depth may prove infeasible.

The depth to groundwater ranges from 4 feet to 8 feet below ground surface. The depth to groundwater below the base of stormwater infiltration BMPs must be greater than 10 feet.

### 7.4 Suitability of the Site for Stormwater Infiltration

In consideration of the foregoing, it is the judgment of NOVA that the site is not suitable for development of stormwater infiltration BMPs.

This judgment is based upon consideration of the variety of factors detailed above, most significantly (i) the low design basis infiltration rate (I) of I = 0.00 to I = 0.01 inches per hour; (ii) the high potential for groundwater mounding; and, (iii) the potential for volume change of the expansive Unit 2 clays due to the introduction of stormwater.

# 8.0 PAVEMENT DESIGN

## 8.1 General

The structural design of pavement sections depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. For the purposes of the preliminary evaluation provided in this section, NOVA has assumed a Traffic Index (TI) of 5.0 for passenger car parking, and 6.0 for the driveways. These traffic indices should be confirmed by the project civil engineer prior to final design.

## 8.2 Drainage

Control of surface drainage is important to the design and construction of pavements. Standing water that develops either on the pavement surface or within the base course can soften the subgrade and create other problems related to the deterioration of the pavement. Good drainage should minimize the risk of the subgrade materials becoming saturated and weakened over a long period of time.

The following recommendations should be considered to limit the amount of excess moisture, which can reach the subgrade soils:

- maintain surface gradients at a minimum 2% grade away from the pavements;
- compact utility trenches for landscaped areas to the same criteria as the pavement subgrade;
- seal all landscaped areas in or adjacent to pavements to minimize or prevent moisture migration to subgrade soils;
- planters should not be located next to pavements (otherwise, subdrains should be used to drain the planter to appropriate outlets);
- place compacted backfill against the exterior side of curb and gutter; and,
- concrete curbs bordering landscaped areas should have a deepened edge to provide a cutoff for moisture flow beneath pavements (generally, the edge of the curb can be extended an additional twelve inches below the base of the curb).

Preventative maintenance should be planned and provided for in the ownership of all pavements. Preventative maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Preventative maintenance consists of both localized maintenance (e.g. crack sealing and patching) and global maintenance (e.g. surface sealing). Preventative maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements.

## 8.3 Subgrade Preparation

### 8.3.1 Subgrade Preparation

Grading for paved areas should consist of removing and replacing the upper 1 foot below subgrade levels. The upper 6" of the subgrade exposed by this excavation should be moisture conditioned to at least 2% above the optimum moisture content, then densified/compacted to a minimum 90% relative compaction after ASTM D 1557 (the 'modified Proctor'). Prior to replacement, the removed soils should be moisture conditioned to at least 2% above the optimum moisture content. The soils should then be replaced at a minimum 90% relative compaction after ASTM D 1557.

#### 8.3.2 Proof Rolling

After the completion of subgrade preparation, areas to receive pavements should be proof-rolled. A loaded dump truck or similar should be used to aid in identifying localized soft or unsuitable material.

Any soft or unsuitable materials encountered during this proof-rolling should be removed, replaced with an approved backfill, and compacted.

#### 8.3.3 Timely Pavement Construction

Construction should be managed such that preparation of the subgrade immediately precedes placement of the base course. Proper drainage of the paved areas should be provided to reduce moisture infiltration to the subgrade.

#### 8.3.4 Surveillance

The preparation of roadway and parking area subgrades should be observed on a full-time basis by a representative of NOVA to confirm that any unsuitable materials have been removed and that the subgrade is suitable for support of the proposed driveways and parking areas after ASTM D1557.

### 8.4 Flexible Pavements

Provided the subgrade in paved areas is prepared per the recommendations in Section 8.3, an R-value of 12 may be used for design. An estimated TI = 7 should be used for design of driveways and delivery routes. Table 8-1 provides recommended sections for flexible pavements.

**Table 8-1. Preliminary Recommendations for Flexible Pavements**

| Area | Estimated Subgrade R-Value | Traffic Index | Asphalt Thickness (in) | Base Course Thickness (in) |
|---|---|---|---|---|
| Auto Driveways/Roadways | 12 | 7.0 | 4.0 | 15.0 |

The above sections assume properly prepared subgrade consisting of at least 12 inches of subgrade compacted to a minimum of 95% relative compaction. The aggregate base, Caltrans Class II aggregate base or similar, should also be placed at a minimum 95% relative compaction. Construction materials (asphalt and aggregate base) should conform to the current Standard Specifications for Public Works Construction (Green Book).

Note that the recommended pavement sections are for planning purposes only. Additional R-value testing should be performed on actual soils at the design subgrade levels to confirm the pavement design.

### 8.5 Rigid Pavements

The flexible pavements may not be adequate for truck loading and turnaround areas. In this event, NOVA recommends that a rigid concrete pavement section be provided.

The rigid pavement section should consist of 7 inches of concrete over a 6-inch base course. The aggregate base materials should be placed at a minimum 95% relative compaction. The concrete should be obtained from a mix design that conforms with the minimum properties shown on Table 8-2 (following page).

**Table 8-2. Recommendations for Concrete Pavements**

| Property | Recommended Requirement |
|---|---|
| Compressive Strength @ 28 days | 3,250 psi minimum |
| Strength Requirements | ASTM C94 |
| Minimum Cement Content | 5.5 sacks/cu. yd. |
| Cement Type | Type V Portland |
| Concrete Aggregate | ASTM C33 |
| Aggregate Size | 1-inch maximum |
| Maximum Water Content | 0.5 lb/lb of cement |
| Maximum Allowable Slump | 4 inches |

Longitudinal and transverse joints should be provided as needed in concrete pavements for expansion/contraction and isolation. Sawed joints should be cut within 24-hours of concrete placement, and should be a minimum of 25% of slab thickness plus 1/4 inch. All joints should be sealed to prevent entry of foreign material and doweled where necessary for load transfer. Where dowels cannot be used at joints accessible to wheel loads, pavement thickness should be increased by 25 percent at the joints and tapered to regular thickness in 5 feet.

# 9.0 REFERENCES

## 9.1 Site Specific

### 9.1.1 Architectural

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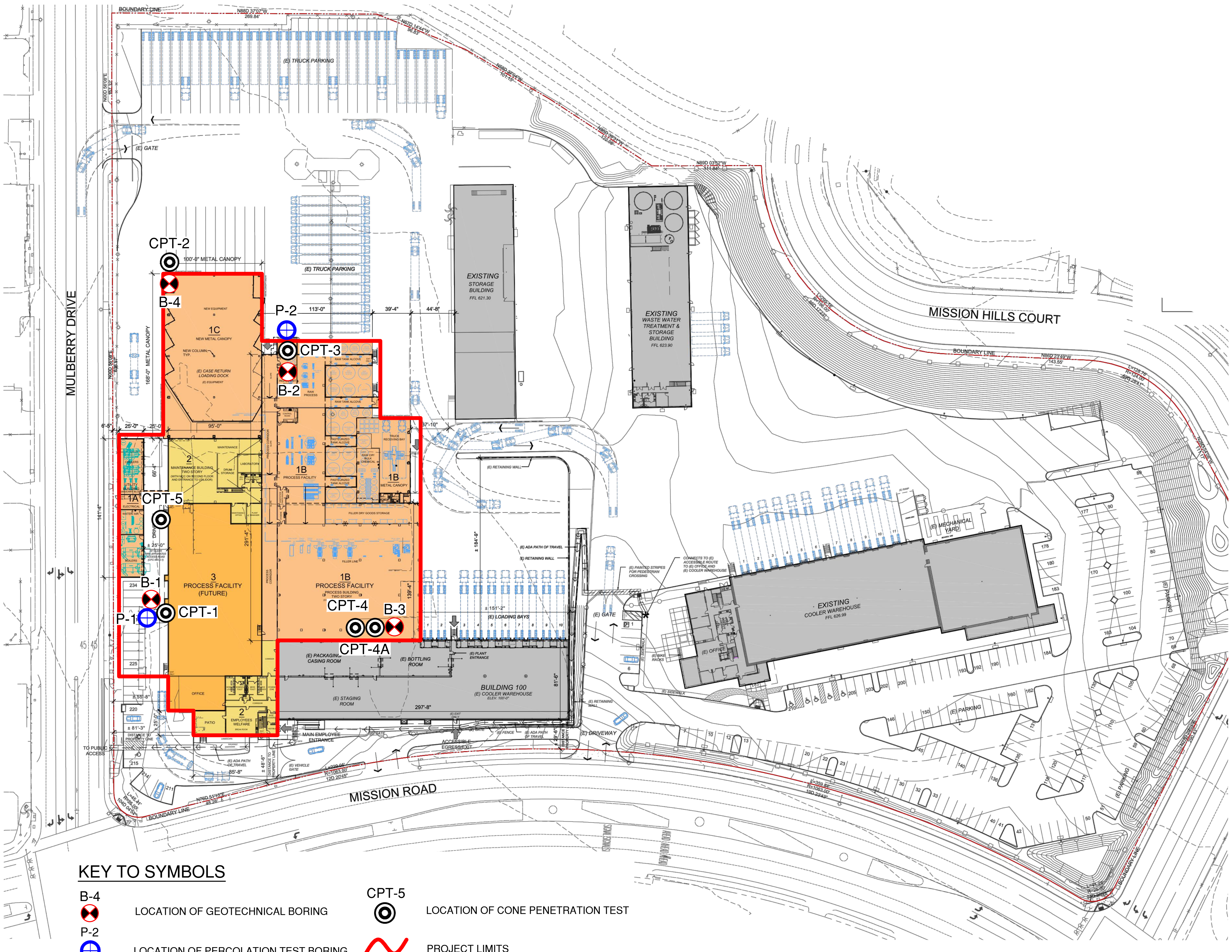
# PLATES

NOVA

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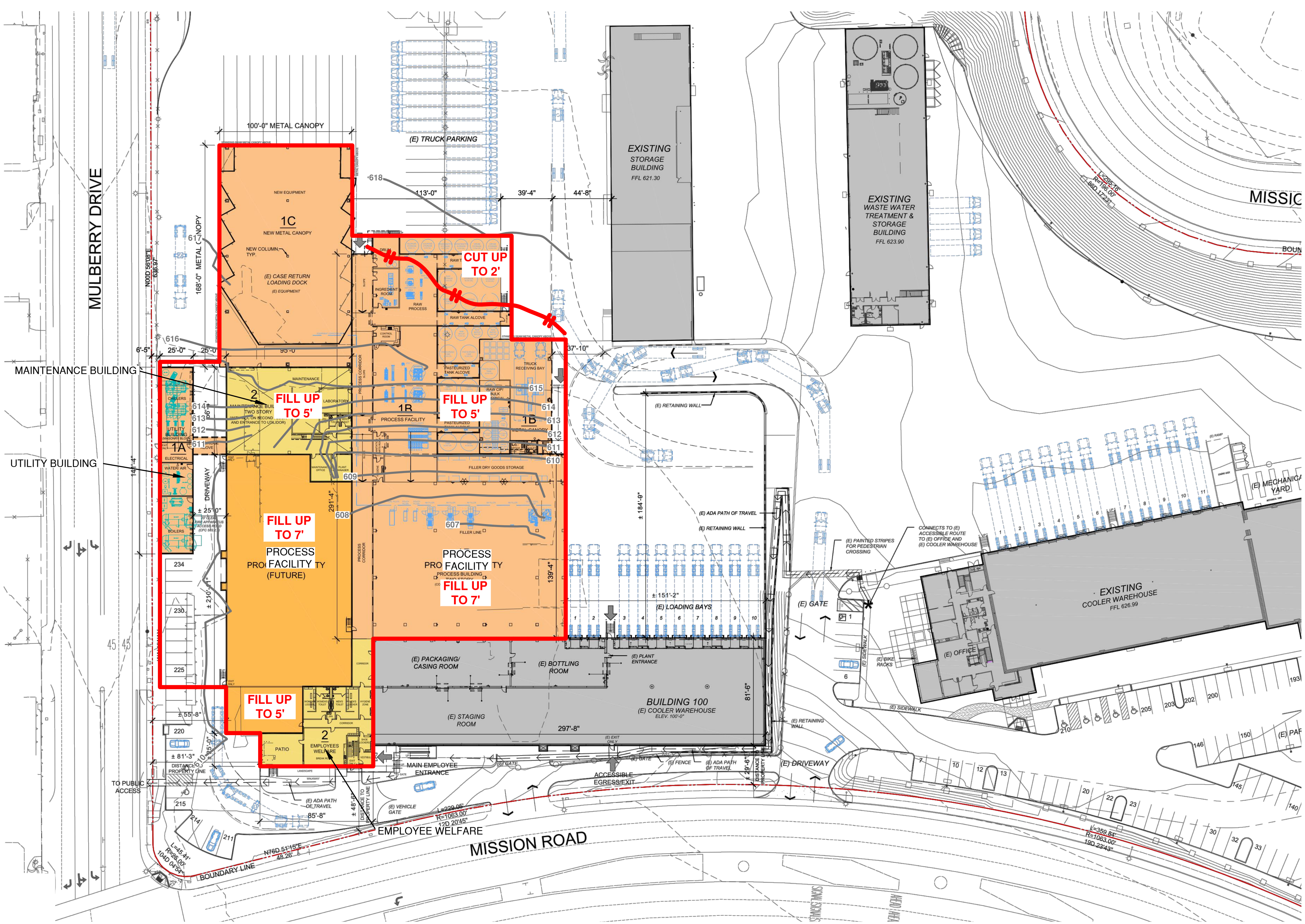
# HOLLANDIA DAIRY
## 622 E. MISSION ROAD
## SAN MARCOS, CALIFORNIA

| | |
|---|---|
| PROJECT NO: | 2019039 |
| DATE: | APR 2019 |
| DRAWN BY: | DTW |
| REVIEWED BY: | WM |

SUBSURFACE INVESTIGATION MAP

0 100' 200'

PLATE 1




## KEY TO SYMBOLS

CUT AND FILL

PROJECT LIMITS

*NOTE: ANTICIPATED CUTS AND FILLS TO ACHIEVE PLANNED FINISH GRADES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# HOLLANDIA DIARY
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

| | |
|---|---|
| PROJECT NO: | 2019039 |
| DATE: | APR 2019 |
| DRAWN BY: | DTW |
| REVIEWED BY: | WM |

PLANNED EARTHWORK

0 80' 160'

PLATE 2

# APPENDIX A

# USE OF THE GEOTECHNICAL REPORT

NOVA

# Important Information About Your

# Geotechnical Engineering Report

*Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.*

*The following information is provided to help you manage your risks.*

## Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

## Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

## A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

## Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

## Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

## A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

## A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

## Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

## Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

## Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

## Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

## Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

## Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org



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# APPENDIX B

## Logs of Borings

NOVA

# BORING LOG B-1

| | | | |
|---|---|---|---|
| **DATE EXCAVATED:** | MARCH 9, 2019 | **EQUIPMENT:** | D50 |
| **EXCAVATION DESCRIPTION:** | 8-INCH DIAMETER AUGER BORING | **GPS COORD.:** | N/A |
| **GROUNDWATER DEPTH:** | 8 FT | **ELEVATION:** | ± 611.3 FT MSL |

**LAB TEST ABBREVIATIONS**

| | |
|---|---|
| CR | CORROSIVITY |
| MD | MAXIMUM DENSITY |
| DS | DIRECT SHEAR |
| EI | EXPANSION INDEX |
| AL | ATTERBERG LIMITS |
| SA | SIEVE ANALYSIS |
| RV | RESISTANCE VALUE |
| CN | CONSOLIDATION |
| SE | SAND EQUIVALENT |

| DEPTH (FT) | SOIL CLASS. (USCS) | BLOWS PER 12-INCHES | **SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)* | LABORATORY | REMARKS |
|---|---|---|---|---|---|
| 0 | | | *6 INCHES OF ASPHALT CONCRETE OVER 6 INCHES OF BASE* | | |
| | CL | 15 | **YOUNGER ALLUVIUM (Qya):** *SILTY CLAY; DARK GRAY BROWN MOTTLED LIGHT GRAY, DAMP, STIFF* | CR AL SA | |
| | | 11 | | | |
| 5 | | 22 | *DARK GRAY BROWN TO DARK GRAY* | | |
| | CL | 13 | **OLDER ALLUVIUM (Qoa):** *SILTY CLAY; BROWN TO GRAY BROWN, DAMP, STIFF* | | |
| 10 | | 24 | *VERY STIFF* | | |
| | | 12 | *STIFF* | AL SA | |
| 15 | ML | 21 | *CLAYEY SILT; ORANGE BROWN, WET, MEDIUM DENSE-VERY STIFF, FINE TO MEDIUM GRAINED* | | |
| 20 | SM | 18 | *SILTY SAND; ORANGE BROWN, SATURATED, MEDIUM DENSE, FINE TO COARSE GRAINED* | SA | |
| 25 | ML | 18 | *SANDY SILT; ORANGE BROWN, MOIST TO WET, VERY STIFF, FINE TO MEDIUM GRAINED SAND* *LIGHT BROWN* | SA | |
| 30 | | | | | |

## KEY TO SYMBOLS

| | | | |
|---|---|---|---|
| ▼/▽ | GROUNDWATER / STABILIZED | # | ERRONEOUS BLOW COUNT |
| ⊠ | BULK SAMPLE | * | NO SAMPLE RECOVERY |
| ◸ | SPT SAMPLE ( ASTM D1586) | ——— | GEOLOGIC CONTACT |
| ■ | CAL. MOD. SAMPLE (ASTM D3550) | — — — | SOIL TYPE CHANGE |

**HOLLANDIA DAIRY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

NOVA

| | | | |
|---|---|---|---|
| LOGGED BY: | DM | DATE: | APR 2019 |
| REVIEWED BY: | BMH | PROJECT NO.: | 2019039 |

APPENDIX B.1

# CONTINUED BORING LOG B-1

| | | | |
|---|---|---|---|
| **DATE EXCAVATED:** | MARCH 9, 2019 | **EQUIPMENT:** | D50 |
| **EXCAVATION DESCRIPTION:** | 8-INCH DIAMETER AUGER BORING | **GPS COORD.:** | N/A |
| **GROUNDWATER DEPTH:** | 8 FT | **ELEVATION:** | ± 611.3 FT MSL |

**LAB TEST ABBREVIATIONS**

| | |
|---|---|
| CR | CORROSIVITY |
| MD | MAXIMUM DENSITY |
| DS | DIRECT SHEAR |
| EI | EXPANSION INDEX |
| AL | ATTERBERG LIMITS |
| SA | SIEVE ANALYSIS |
| RV | RESISTANCE VALUE |
| CN | CONSOLIDATION |
| SE | SAND EQUIVALENT |

| DEPTH (FT) | GRAPHIC LOG | BULK SAMPLE | CAL/SPT SAMPLE | SOIL CLASS. (USCS) | BLOWS PER 12-INCHES | SOIL DESCRIPTION<br>*SUMMARY OF SUBSURFACE CONDITIONS*<br>*(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)* | LABORATORY | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 30 | | | | SM-ML | 20 | **OLDER ALLUVIUM (Qoa):** *SILTY SAND-SANDY SILT; ORANGE BROWN, MOIST TO WET, MEDIUM DENSE OR VERY STIFF, FINE GRAINED* | SA | |
| 35 | | | | | 24 | | | |
| | | | | SM | | *SILTY SAND; BROWN TO BLACK, SATURATED, MEDIUM DENSE, FINE TO COARSE GRAINED* | SA | |
| | | | | | 13 | | | |
| 40 | | | | | | *BROWN* | SA | |
| | | | | SC | 50/4"<br>50/2" | **METAVOLCANIC ROCK (Mzu):** *CLAYEY SANDSTONE; GRAY MOTTLED ORANGE, DAMP TO MOIST, HARD* | | *WEATHERED* |
| | | | | | | *BORING TERMINATED AT 42.2 FT DUE TO REFUSAL. GROUNDWATER ENCOUNTERED AT 15 FT. GROUNDWATER STABILIZED AT 8 FT. NO CAVING.* | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |
| 55 | | | | | | | | |
| 60 | | | | | | | | |

## KEY TO SYMBOLS

| | | | |
|---|---|---|---|
| ▼/▽ | GROUNDWATER / STABILIZED | # | ERRONEOUS BLOW COUNT |
| ⊠ | BULK SAMPLE | * | NO SAMPLE RECOVERY |
| ◪ | SPT SAMPLE ( ASTM D1586) | ——— | GEOLOGIC CONTACT |
| ■ | CAL. MOD. SAMPLE (ASTM D3550) | — — — | SOIL TYPE CHANGE |

**HOLLANDIA DAIRY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

| | | | |
|---|---|---|---|
| LOGGED BY: | DM | DATE: | APR 2019 |
| REVIEWED BY: | BMH | PROJECT NO.: | 2019039 |

NOVA

# BORING LOG B-2

| | | | |
|---|---|---|---|
| **DATE EXCAVATED:** | MARCH 9, 2019 | **EQUIPMENT:** | D50 |
| **EXCAVATION DESCRIPTION:** | 8-INCH DIAMETER AUGER BORING | **GPS COORD.:** | N/A |
| **GROUNDWATER DEPTH:** | 7.1 FT | **ELEVATION:** | ± 616.1 FT MSL |

**LAB TEST ABBREVIATIONS**

| | |
|---|---|
| CR | CORROSIVITY |
| MD | MAXIMUM DENSITY |
| DS | DIRECT SHEAR |
| EI | EXPANSION INDEX |
| AL | ATTERBERG LIMITS |
| SA | SIEVE ANALYSIS |
| RV | RESISTANCE VALUE |
| CN | CONSOLIDATION |
| SE | SAND EQUIVALENT |

| DEPTH (FT) | SOIL CLASS. (USCS) | BLOWS PER 12-INCHES | SOIL DESCRIPTION<br>*SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)* | LABORATORY | REMARKS |
|---|---|---|---|---|---|
| 0 | | | *4 INCHES OF ASPHALT CONCRETE OVER 6 INCHES OF BASE* | | |
| | CL | | **YOUNGER ALLUVIUM (Qya):** *SANDY CLAY; DARK GRAY, DAMP, STIFF, TRACE GRAVEL* | EI<br>AL | *71 MEDIUM* |
| | | 12 | *BROWN* | | |
| 5 | ML-CL | 57 | **OLDER ALLUVIUM (Qoa):** *SILTY CLAY-CLAYEY SILT; RED BROWN MOTTLED GRAY, DAMP, HARD* | | |
| 10 | | 14 | *GRAY BROWN, DAMP TO MOIST, STIFF* | SA | |
| | | | *STIFF* | | |
| 15 | SM-ML | 24 | *SILTY SAND-SANDY SILT; BROWN MOTTLED GRAY-BROWN, MOIST, MEDIUM DENSE-VERY STIFF, FINE TO MEDIUM GRAINED, TRACE COARSE SAND* | SA | |
| 20 | ML | 11 | *CLAYEY SILT; BROWN, MOIST, STIFF* | SA | |
| 25 | | 22 | *RED BROWN, TRACE COARSE SAND* | | |
| | | | *LIGHT BROWN* | | |
| 30 | | | | | |

## KEY TO SYMBOLS

| | |
|---|---|
| GROUNDWATER / STABILIZED | # ERRONEOUS BLOW COUNT |
| BULK SAMPLE | * NO SAMPLE RECOVERY |
| SPT SAMPLE ( ASTM D1586) | GEOLOGIC CONTACT |
| CAL. MOD. SAMPLE (ASTM D3550) | SOIL TYPE CHANGE |

**HOLLANDIA DAIRY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

| | | | |
|---|---|---|---|
| LOGGED BY: | DM | DATE: | APR 2019 |
| REVIEWED BY: | BMH | PROJECT NO.: 2019039 | |

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APPENDIX B.3

# CONTINUED BORING LOG B-2

**DATE EXCAVATED:** MARCH 9, 2019 **EQUIPMENT:** D50

**EXCAVATION DESCRIPTION:** 8-INCH DIAMETER AUGER BORING **GPS COORD.:** N/A

**GROUNDWATER DEPTH:** 7.1 FT **ELEVATION:** ± 616.1 FT MSL

**LAB TEST ABBREVIATIONS**

| | |
|---|---|
| CR | CORROSIVITY |
| MD | MAXIMUM DENSITY |
| DS | DIRECT SHEAR |
| EI | EXPANSION INDEX |
| AL | ATTERBERG LIMITS |
| SA | SIEVE ANALYSIS |
| RV | RESISTANCE VALUE |
| CN | CONSOLIDATION |
| SE | SAND EQUIVALENT |

| DEPTH (FT) | GRAPHIC LOG | BULK SAMPLE | CAL/SPT SAMPLE | SOIL CLASS. (USCS) | BLOWS PER 12-INCHES | SOIL DESCRIPTION<br>*SUMMARY OF SUBSURFACE CONDITIONS*<br>*(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)* | LABORATORY | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 30 | | | | ML | 16 | **OLDER ALLUVIUM (Qoa):** *(CONTINUED) CLAYEY SILT; RED BROWN, MOIST, STIFF* | SA | |
| | | | | SM | | *SILTY SAND; RED BROWN TO BROWN, SATURATED, DENSE, FINE TO MEDIUM GRAINED* | | |
| 35 | | | | | 57 | | | |
| | | | | SM | 50/6"<br>50/1" | **METAVOLCANIC ROCK (Mzu):** *SILTY SANDSTONE; GRAY, WET, VERY DENSE, SCATTERED ROCK FRAGMENTS* | SA | *WEATHERED* |
| 40 | | | | | | *BORING TERMINATED AT 39.1 FT DUE TO REFUSAL. GROUNDWATER ENCOUNTERED AT 15 FT. GROUNDWATER STABILIZED AT 7.1 FT. NO CAVING.* | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |
| 55 | | | | | | | | |
| 60 | | | | | | | | |

## KEY TO SYMBOLS

| | | | |
|---|---|---|---|
| ▼/▽ | GROUNDWATER / STABILIZED | # | ERRONEOUS BLOW COUNT |
| ⊠ | BULK SAMPLE | * | NO SAMPLE RECOVERY |
| ◸ | SPT SAMPLE ( ASTM D1586) | ——— | GEOLOGIC CONTACT |
| ■ | CAL. MOD. SAMPLE (ASTM D3550) | — — — | SOIL TYPE CHANGE |

**HOLLANDIA DAIRY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

LOGGED BY: DM DATE: APR 2019
REVIEWED BY: BMH PROJECT NO.: 2019039

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APPENDIX B.4

# BORING LOG B-3

| | | | |
|---|---|---|---|
| **DATE EXCAVATED:** | MARCH 9, 2019 | **EQUIPMENT:** | D50 |
| **EXCAVATION DESCRIPTION:** | 8-INCH DIAMETER AUGER BORING | **GPS COORD.:** | N/A |
| **GROUNDWATER DEPTH:** | 4.1 FT | **ELEVATION:** | ± 610.3 FT MSL |

**LAB TEST ABBREVIATIONS**

| | |
|---|---|
| CR | CORROSIVITY |
| MD | MAXIMUM DENSITY |
| DS | DIRECT SHEAR |
| EI | EXPANSION INDEX |
| AL | ATTERBERG LIMITS |
| SA | SIEVE ANALYSIS |
| RV | RESISTANCE VALUE |
| CN | CONSOLIDATION |
| SE | SAND EQUIVALENT |

| DEPTH (FT) | GRAPHIC LOG | BULK SAMPLE | CAL/SPT SAMPLE | SOIL CLASS. (USCS) | BLOWS PER 12-INCHES | SOIL DESCRIPTION<br>*SUMMARY OF SUBSURFACE CONDITIONS*<br>*(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)* | LABORATORY | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | SC | 10 | **FILL (Qaf):** *CLAYEY SAND; BROWN MOTTLED LIGHT BROWN, DAMP TO MOIST, LOOSE, SCATTERED GRAVEL, TRACE ASPHALT CONCRETE DEBRIS, TRACE PLYWOOD* | MD<br>AL<br>SA | *7.8% 127.2pcf* |
| | | | | | 9 | *GRAY BROWN MOTTLED RED BROWN* | | |
| 5 | | | | CL | 17 | **OLDER ALLUVIUM (Qoa):** *SILTY CLAY; RED BROWN, DAMP, VERY STIFF, TRACE BLACK STAINING* | SA | |
| | | | | | 14 | *LIGHT BROWN, STIFF* | | |
| 10 | | | | | 49 | *LIGHT BROWN MOTTLED RED, HARD* | | |
| | | | | | | *ORANGE BROWN MOTTLED LIGHT BROWN, MOIST, VERY STIFF* | | |
| 15 | | | | | 18 | *VERY STIFF* | SA | |
| 20 | | | | ML | 10 | *CLAYEY SILT; LIGHT BROWN, DAMP, STIFF* | | |
| | | | | | | *SCATTERED ROCK FRAGMENTS, HARDER DRILLING* | | |
| 25 | | | | | 26 | *LIGHT BROWN MOTTLED GRAY BROWN, VERY STIFF, TRACE COARSE GRAINED SAND, TRACE YELLOW STAINING* | | |
| | | | | | | **METAVOLCANIC ROCK (Mzu):** *CLAYEY SANDSTONE; GRAY, WET, VERY DENSE, SOME ROCK FRAGMENTS* | | |
| 30 | | | | SC | 50/2" | *BORING TERMINATED AT 29.7 FT DUE TO REFUSAL. GROUNDWATER ENCOUNTERED AT 15 FT. GROUNDWATER STABILIZED AT 4.1 FT. NO CAVING.* | | |

## KEY TO SYMBOLS

| | | | |
|---|---|---|---|
| ▼/▽ | GROUNDWATER / STABILIZED | # | ERRONEOUS BLOW COUNT |
| ⊠ | BULK SAMPLE | * | NO SAMPLE RECOVERY |
| ◸ | SPT SAMPLE ( ASTM D1586) | —— | GEOLOGIC CONTACT |
| ■ | CAL. MOD. SAMPLE (ASTM D3550) | – – – | SOIL TYPE CHANGE |

**HOLLANDIA DAIRY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

NOVA

| | | | |
|---|---|---|---|
| LOGGED BY: | DM | DATE: | APR 2019 |
| REVIEWED BY: | BMH | PROJECT NO.: 2019039 | APPENDIX B.5 |

# BORING LOG B-4

| | | | |
|---|---|---|---|
| **DATE EXCAVATED:** | MARCH 9, 2019 | **EQUIPMENT:** | D50 |
| **EXCAVATION DESCRIPTION:** | 8-INCH DIAMETER AUGER BORING | **GPS COORD.:** | N/A |
| **GROUNDWATER DEPTH:** | GROUNDWATER NOT ENCOUNTERED | **ELEVATION:** | ± 616.1 FT MSL |

**LAB TEST ABBREVIATIONS**

| | |
|---|---|
| CR | CORROSIVITY |
| MD | MAXIMUM DENSITY |
| DS | DIRECT SHEAR |
| EI | EXPANSION INDEX |
| AL | ATTERBERG LIMITS |
| SA | SIEVE ANALYSIS |
| RV | RESISTANCE VALUE |
| CN | CONSOLIDATION |
| SE | SAND EQUIVALENT |

| DEPTH (FT) | GRAPHIC LOG | BULK SAMPLE | CAL/SPT SAMPLE | SOIL CLASS. (USCS) | BLOWS PER 12-INCHES | SOIL DESCRIPTION<br>*SUMMARY OF SUBSURFACE CONDITIONS*<br>*(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)* | LABORATORY | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | *3 INCHES OF ASPHALT CONCRETE* | | |
| | | | | CL | 7 | **YOUNGER ALLUVIUM (Qya):** *SILTY CLAY; DARK GRAY TO BLACK MOTTLED BROWN; DAMP, SOFT TO FIRM, TRACE GRAVEL* | RV<br>SA | |
| | | | | | | *BLACK, FIRM* | | |
| | | | | | 28 | *BROWN* | | |
| | | | | | | *DARK GRAY, VERY STIFF* | | |
| 5 | | | | ML-CL | 29 | **OLDER ALLUVIUM (Qoa):** *CLAYEY SILT-SILTY CLAY; ORANGE BROWN, DRY, VERY STIFF* | | |
| | | | | | | *DAMP, TRACE BLACK STAINING* | SA | |
| 10 | | | | | 10 | *LIGHT BROWN, TRACE BLACK STAINING* | | |
| 15 | | | | | 17 | *LIGHT GRAY BROWN MOTTLED ORANGE* | | |
| | | | | | | *BORING TERMINATED AT 16.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.* | | |
| 20 | | | | | | | | |
| 25 | | | | | | | | |
| 30 | | | | | | | | |

## KEY TO SYMBOLS

| | | | |
|---|---|---|---|
| ▼/▽ | GROUNDWATER / STABILIZED | # | ERRONEOUS BLOW COUNT |
| ⊠ | BULK SAMPLE | * | NO SAMPLE RECOVERY |
| ⧄ | SPT SAMPLE ( ASTM D1586) | ——— | GEOLOGIC CONTACT |
| ■ | CAL. MOD. SAMPLE (ASTM D3550) | — — — | SOIL TYPE CHANGE |

**HOLLANDIA DAIRY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

| | | | |
|---|---|---|---|
| LOGGED BY: | DM | DATE: | APR 2019 |
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NOVA

APPENDIX B.6

# PERCOLATION BORING LOG P-1

| | | | |
|---|---|---|---|
| **DATE EXCAVATED:** | MARCH 9, 2019 | **EQUIPMENT:** | D50 |
| **EXCAVATION DESCRIPTION:** | 8-INCH DIAMETER AUGER BORING | **GPS COORD.:** | N/A |
| **GROUNDWATER DEPTH:** | GROUNDWATER NOT ENCOUNTERED | **ELEVATION:** | ± 611.3 FT MSL |

**LAB TEST ABBREVIATIONS**

| | |
|---|---|
| CR | CORROSIVITY |
| MD | MAXIMUM DENSITY |
| DS | DIRECT SHEAR |
| EI | EXPANSION INDEX |
| AL | ATTERBERG LIMITS |
| SA | SIEVE ANALYSIS |
| RV | RESISTANCE VALUE |
| CN | CONSOLIDATION |
| SE | SAND EQUIVALENT |

| DEPTH (FT) | GRAPHIC LOG | BULK SAMPLE | CAL/SPT SAMPLE | SOIL CLASS. (USCS) | BLOWS PER 12-INCHES | **SOIL DESCRIPTION**<br>*SUMMARY OF SUBSURFACE CONDITIONS*<br>*(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)* | LABORATORY | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | *6 INCHES OF ASPHALT CONCRETE OVER 6 INCHES OF BASE* | | |
| | | | | CL | | **YOUNGER ALLUVIUM (Qya):** *SILTY CLAY; DARK GRAY BROWN MOTTLED LIGHT GRAY, DAMP, STIFF* | | |
| 5 | | | | | | *BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION WELL.* | | |
| 10 | | | | | | | | |
| 15 | | | | | | | | |
| 20 | | | | | | | | |
| 25 | | | | | | | | |
| 30 | | | | | | | | |

**KEY TO SYMBOLS**

| | | | |
|---|---|---|---|
| ▼/▽ | GROUNDWATER / STABILIZED | # | ERRONEOUS BLOW COUNT |
| ⊠ | BULK SAMPLE | * | NO SAMPLE RECOVERY |
| ◱ | SPT SAMPLE ( ASTM D1586) | ——— | GEOLOGIC CONTACT |
| ■ | CAL. MOD. SAMPLE (ASTM D3550) | — — — | SOIL TYPE CHANGE |

**HOLLANDIA DAIRY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

| | | | |
|---|---|---|---|
| LOGGED BY: | DM | DATE: | APR 2019 |
| REVIEWED BY: | BMH | PROJECT NO.: 2019039 | |

NOVA

APPENDIX B.7

# PERCOLATION BORING LOG P-2

**DATE EXCAVATED:** MARCH 9, 2019 **EQUIPMENT:** D50

**EXCAVATION DESCRIPTION:** 8-INCH DIAMETER AUGER BORING **GPS COORD.:** N/A

**GROUNDWATER DEPTH:** GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 616.1 FT MSL

| LAB TEST ABBREVIATIONS | |
|---|---|
| CR | CORROSIVITY |
| MD | MAXIMUM DENSITY |
| DS | DIRECT SHEAR |
| EI | EXPANSION INDEX |
| AL | ATTERBERG LIMITS |
| SA | SIEVE ANALYSIS |
| RV | RESISTANCE VALUE |
| CN | CONSOLIDATION |
| SE | SAND EQUIVALENT |

| DEPTH (FT) | GRAPHIC LOG | BULK SAMPLE | CAL/SPT SAMPLE | SOIL CLASS. (USCS) | BLOWS PER 12-INCHES | SOIL DESCRIPTION<br>*SUMMARY OF SUBSURFACE CONDITIONS*<br>*(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)* | LABORATORY | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | *4 INCHES OF ASPHALT CONCRETE OVER 6 INCHES OF BASE* | | |
| | | | | CL | | **YOUNGER ALLUVIUM (Qya):** *SANDY CLAY; DARK GRAY, DAMP, STIFF, TRACE GRAVEL* | | |
| 5 | | | | | | *BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION WELL.* | | |
| 10 | | | | | | | | |
| 15 | | | | | | | | |
| 20 | | | | | | | | |
| 25 | | | | | | | | |
| 30 | | | | | | | | |

## KEY TO SYMBOLS

| | | | |
|---|---|---|---|
| ▼/▽ | GROUNDWATER / STABILIZED | # | ERRONEOUS BLOW COUNT |
| ⊠ | BULK SAMPLE | * | NO SAMPLE RECOVERY |
| ◸ | SPT SAMPLE ( ASTM D1586) | ——— | GEOLOGIC CONTACT |
| ■ | CAL. MOD. SAMPLE (ASTM D3550) | — — — | SOIL TYPE CHANGE |

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SAN MARCOS, CALIFORNIA

NOVA

| | | | |
|---|---|---|---|
| LOGGED BY: | DM | DATE: | APR 2019 |
| REVIEWED BY: | BMH | PROJECT NO.: 2019039 | APPENDIX B.8 |

# APPENDIX C

## Infiltration Worksheets

NOVA

**Worksheet C.4-1: Categorization of Infiltration Feasibility Condition**

| Categorization of Infiltration Feasibility Condition | | Worksheet C.4-1 | |
|---|---|---|---|
| **Part 1 - Full Infiltration Feasibility Screening Criteria**<br>**Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?** | | | |
| Criteria | Screening Question | **Yes** | **No** |
| 1 | **Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D. | | **X** |
| Provide basis:<br>*The infiltration rate of the existing soils for locations P-1 and P-2, based on the on-site infiltration study was calculated to be less than 0.5 inches per hour (P-1=0.00 and P-2=0.01, and inches per hour) after applying a minimum factor of safety (F) of F=2.*<br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability. | | | |
| 2 | **Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2. | | **X** |
| Provide basis:<br>*No. See Criterion 1.*<br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability. | | | |

| Worksheet C.4-1 Page 2 of 4 | | | |
|---|---|---|---|
| Criteria | Screening Question | **Yes** | **No** |
| 3 | **Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3. | | |
| Provide basis:<br>*Water contamination was not evaluated by NOVA Services.*<br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability. | | | |
| 4 | **Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3. | | |
| Provide basis:<br>*The potential for water balance was not evaluated by NOVA Services.*<br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability. | | | |
| **Part 1 Result*** | If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration**<br><br>If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2 | *Proceed to Part 2* | |

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

| Worksheet C.4-1 Page 3 of 4 | | | |
|---|---|---|---|
| **Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria**<br><br>**Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?** | | | |
| Criteria | Screening Question | **Yes** | **No** |
| 5 | **Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D. | | **X** |
| Provide basis:<br><br>*The infiltration rate of the existing soils for location P-1 and P-2, based on the on-site infiltration study was calculated to be less than 0.5 inches per hour (P-1=0.00 and P-2=0.01 inches per hour) after applying a minimum factor of safety (F) of F=2.*<br><br>*These widespread very low permeability soils and geologic conditions do not allow for infiltration in any appreciable rate or volume.*<br><br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates. | | | |
| 6 | **Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2. | | **X** |
| Provide basis:<br>*C2.1 A geologic investigation was performed at the subject site.*<br>*C2.2 Settlement and volume change due to water infiltration is possible due to the expansive soils underlying the site.*<br>*C2.3 Infiltration has the potential to cause slope failures. BMPs are to be sited a minimum of 50 feet away from any slope.*<br>*C2.4 BMPs are to be sited a minimum of 10 feet away from all underground utilities.*<br>*C2.5 Stormwater infiltration can result in damaging ground water mounding during wet periods. Due to the low infiltration rates and shallow depths to groundwater, this site is at a high risk.*<br>*C2.6 Infiltration has the potential to increase lateral pressure and reduce soil strength which can impact foundations and retaining walls. BMPs are to be sited a minimum of 10 feet away from any foundations or retaining walls.*<br>*C2.7 Other Factors: Based on the low infiltration rates, high risk for groundwater mounding, and clayey soils underlying the site, infiltration is not feasibile.* | | | |

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:<br>Water contamination was not evaluated by NOVA Services.<br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:<br>The potential for water balance was not evaluated by NOVA Services.<br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.<br><br>If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No Infiltration</td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings

# C.5 Feasibility Screening Exhibits

Table C.5-1 lists the feasibility screening exhibits that were generated using readily available GIS data sets to assist the project applicant to screen the project site for feasibility.

**Table C.5-1: Feasibility Screening Exhibits**

| Figures | Layer | Intent/Rationale | Data Sources |
|---|---|---|---|
| C.1 Soils | Hydrologic Soil Group – A, B, C, D | Hydrologic Soil Group will aid in determining areas of potential infiltration | SanGIS<br>http://www.sangis.org/ |
| | Hydric Soils | Hydric soils will indicate layers of intermittent saturation that may function like a D soil and should be avoided for infiltration | USDA Web Soil Survey. Hydric soils, (ratings of 100) were classified as hydric.<br>http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm |
| C.2: Slopes and Geologic Hazards | Slopes >25% | BMPs are hard to construct on slopes >25% and can potentially cause slope instability | SanGIS<br>http://www.sangis.org/ |
| | Liquefaction Potential | BMPs (particularly infiltration BMPs) must not be sited in areas with high potential for liquefaction or landslides to minimize earthquake/landslide risks | SanGIS<br>http://www.sangis.org/ |
| | Landslide Potential | | SanGIS Geologic Hazards layer. Subset of polygons with hazard codes related to landslides was selected. This data is limited to the City of San Diego Boundary.<br>http://www.sangis.org/ |
| C.3: Groundwater Table Elevations | Groundwater Depths | Infiltration BMPs will need to be sited in areas with adequate distance (>10 ft) from the groundwater table | GeoTracker. Data downloaded for San Diego county from 2014 and 2013. In cases where there were multiple measurements made at the same well, the average was taken over that year.<br>http://geotracker.waterboards.ca.gov/data_download_by_county.asp |
| C.4: Contaminated Sites | Contaminated soils and/or groundwater sites | Infiltration must limited in areas of contaminated soil/groundwater | GeoTracker. Data downloaded for San Diego county and limited to active cleanup sites<br>http://geotracker.waterboards.ca.gov/ |

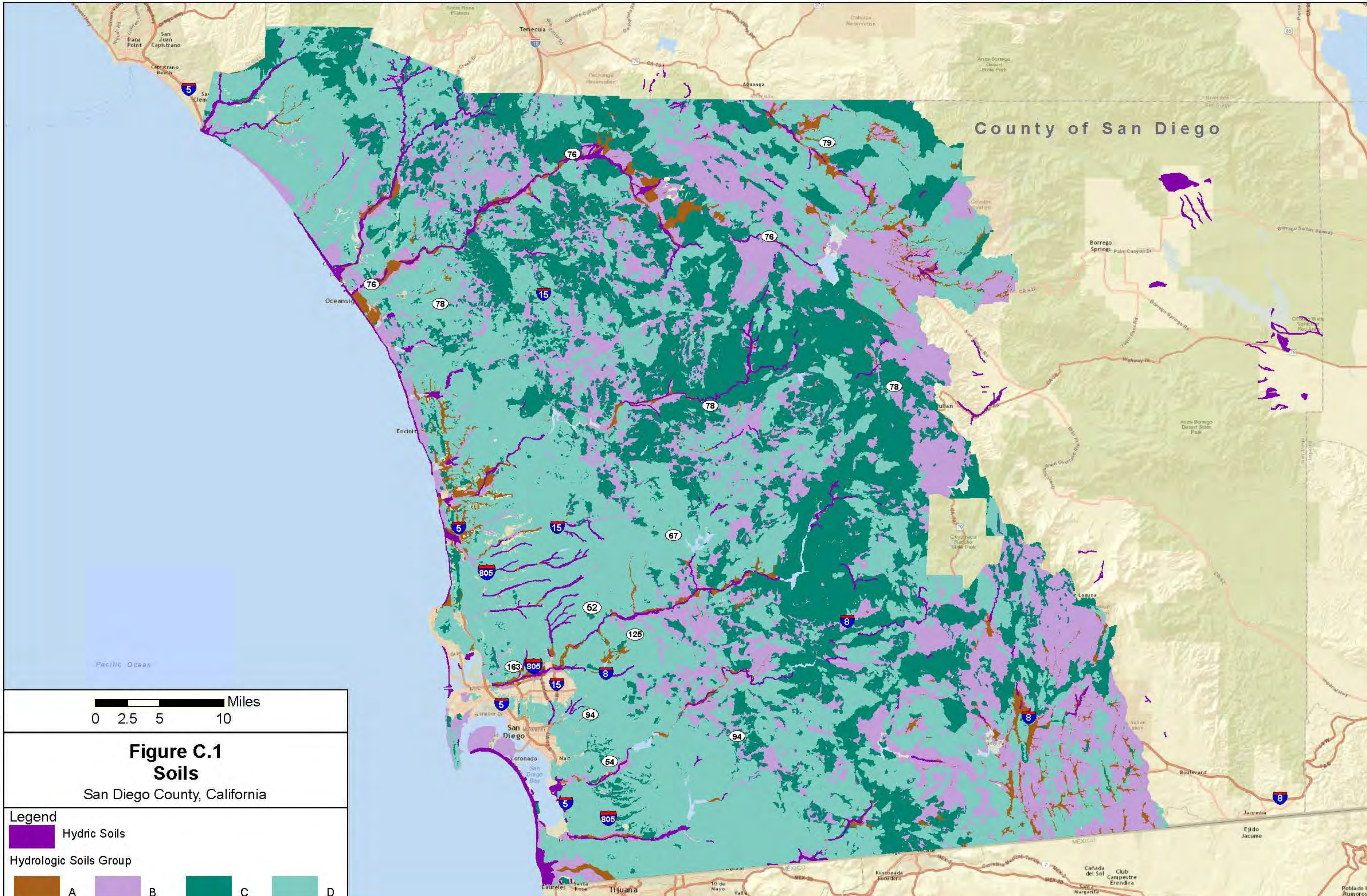


**Figure C.1**
**Soils**
San Diego County, California

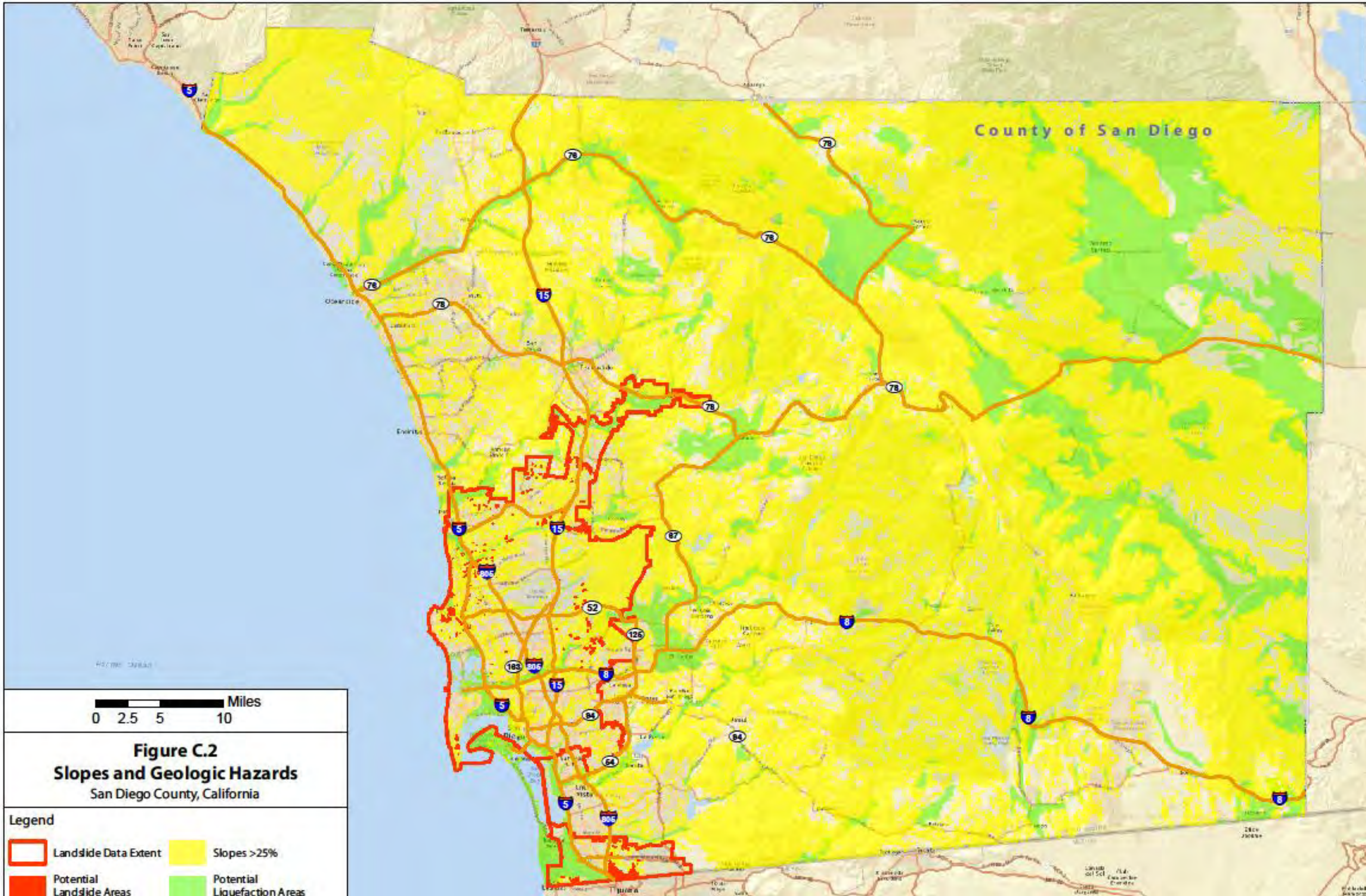


**Figure C.2**
**Slopes and Geologic Hazards**
San Diego County, California

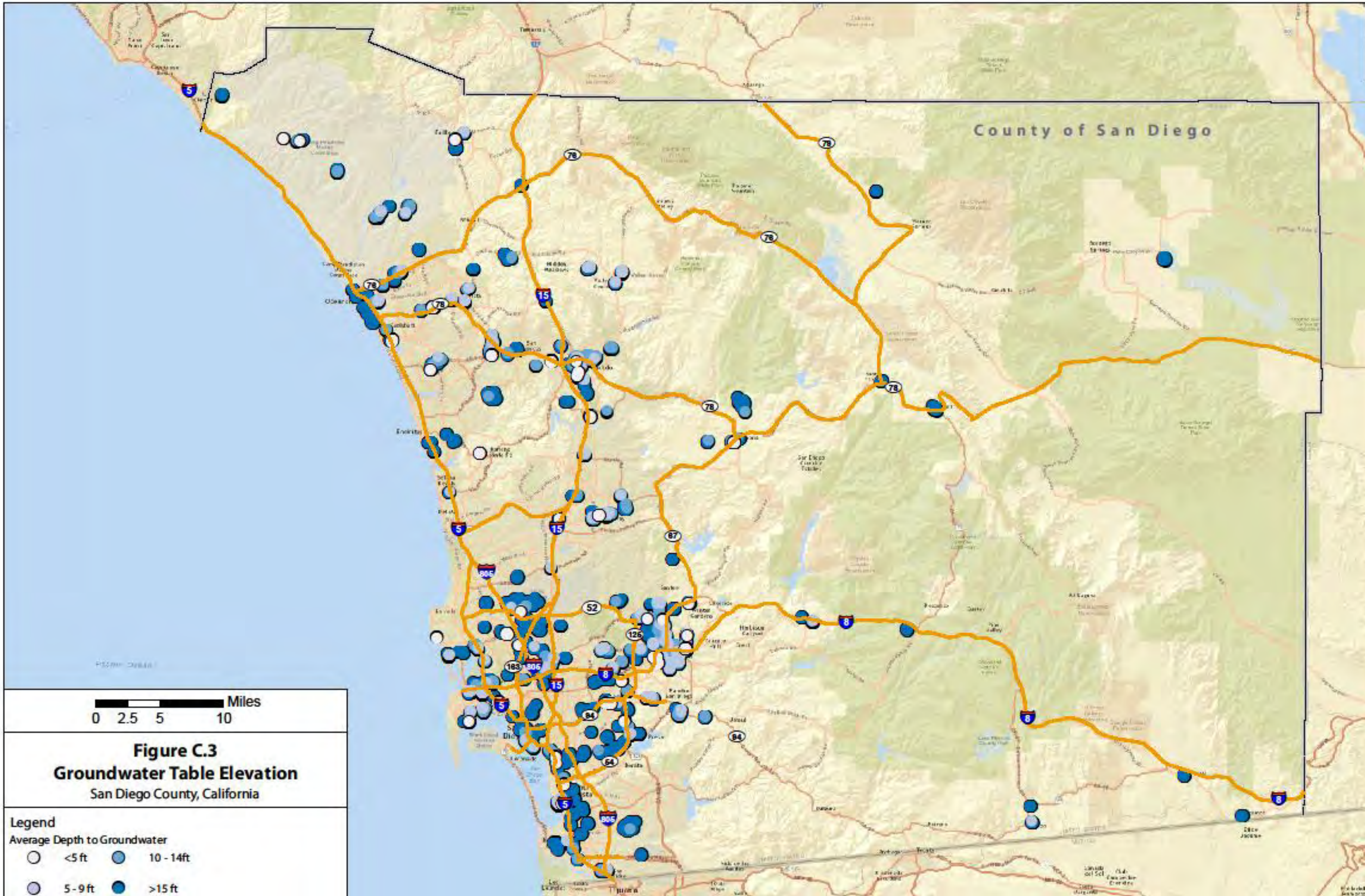


**Figure C.3**
**Groundwater Table Elevation**
San Diego County, California

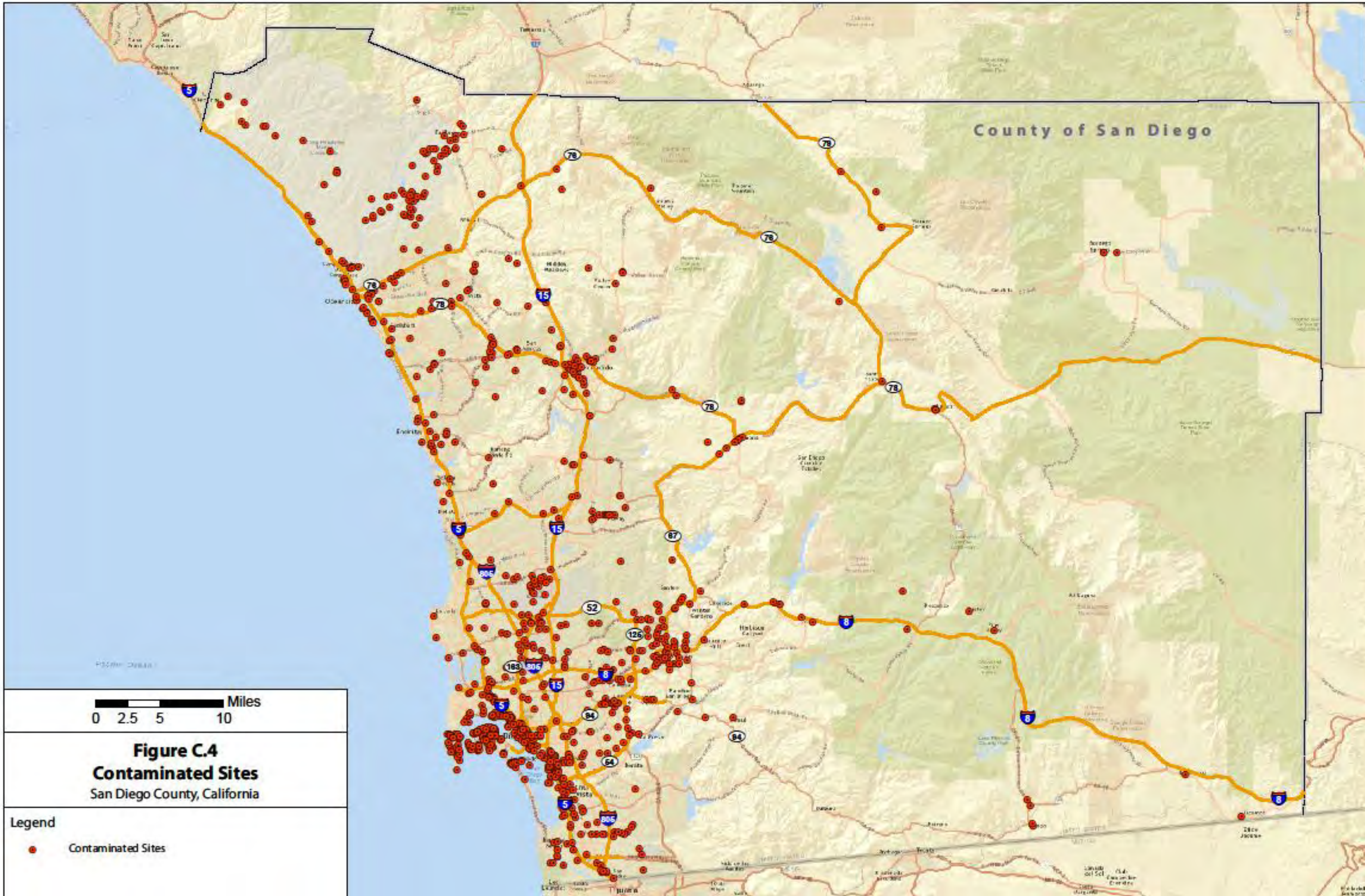


**Figure C.4**
**Contaminated Sites**
San Diego County, California

# APPENDIX D

## Records of Laboratory Testing

*Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. Brief descriptions of the tests performed are presented below:*

- **CLASSIFICATION:** *Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soils Classification System and are presented on the exploration logs in Appendix B.*

- **MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT (ASTM D1557 METHOD A,B,C):** *The maximum dry density and optimum moisture content of typical soils were determined in the laboratory in accordance with ASTM Standard Test D1557, Method A, Method B, Method C.*

- **DENSITY OF SOIL IN PLACE (ASTM D2937):** *In-place moisture contents and dry densities were determined for representative soil samples. This information was an aid to classification and permitted recognition of variations in material consistency with depth. The dry unit weight is determined in pounds per cubic foot, and the in-place moisture content is determined as a percentage of the soil's dry weight. The results are summarized in the exploration logs presented in Appendix B.*

- **ATTERBERG LIMITS (ASTM D 4318):** *Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System.*

- **EXPANSION INDEX (ASTM D 4829):** *The expansion index of selected materials was evaluated in general accordance with ASTM D 4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thich by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours.*

- **R-VALUE (ASTM D 2844):** *The resistance Value, or R-Value, for near-surface site soils were evaluated in general accordance with California Test (CT) 301 and ASTM D 2844. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results.*

- **CORROSIVITY (CAL. TEST METHOD 417, 422, 643):** *Soil PH, and minimum resistivity tests were performed on a representative soil sample in general accordance with test method CT 643. The sulfate and chloride content of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively.*

- **GRADATION ANALYSIS (ASTM C 136 and/or ASTM D422):** *Tests were performed on selected representative soil samples in general accordance with ASTM D422. The grain size distributions of selected samples were determined in accordance with ASTM C 136 and/or ASTM D422. The results of the tests are summarized on Appendix D.3 through Appendix D.19.*

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**LAB TEST SUMMARY**

**HOLLANDIA DIARY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.1 |
|---|---|---|---|

## Expansion Index (ASTM D4829)

| Sample Location | Sample Depth (ft.) | Expansion Index | Expansion Potential |
|---|---|---|---|
| B-2 | 1.0'-4.0' | 71 | Medium |

## DENSITY OF SOIL IN PLACE (ASTM D2937)

| Sample Location | Sample Depth (ft) | Soil Description | Moisture (%) | Dry Density (pcf) |
|---|---|---|---|---|
| B-1 | 5.0' | Dark Gray Brown-Dark Gray Silty Clay | 26.2 | 97.5 |
| B-3 | 3.0' | Gray Brown Mottled Red Brown Clayey Sand | 12.1 | 114.1 |
| B-3 | 10.0' | Clayey Silt-Silty Clay Light Brown Mottled Red | 19.1 | 110.6 |
| B-4 | 2.5' | Dark Gray-Black Mottled Brown Silty Clay | 24.6 | 97.3 |

## Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

| Sample Location | Sample Depth (ft.) | Soil Description | Maximum Dry Density (pcf) | Optimum Moisture Content (%) |
|---|---|---|---|---|
| B-3 | 0.0'-3.0' | Gray Brown Mottled Red Brown Clayey Sand | 127.2 | 7.8 |

## Atterberg Limits (ASTM D4318)

| Sample Location | Sample Depth (ft.) | Liquid Limit, LL | Plastic Limit, PL | Plasticity Index, PI | USCS |
|---|---|---|---|---|---|
| B-1 | 1.0'-5.0' | 40 | 19 | 21 | CL |
| B-1 | 12.0'-15.0' | 42 | 21 | 21 | CL |
| B-2 | 1.0'-4.0' | 31 | 16 | 15 | CL |
| B-3 | 0.0'-3.0' | 32 | 18 | 14 | CL |

## Resistance Value (Cal. Test Method 301 & ASTM D2844)

| Sample Location | Sample Depth (ft.) | Soil Description | R-Value |
|---|---|---|---|
| B-4 | 1.0'-3.0' | Dark Gray to Black Mottled Brown Silty Clay | 12 |

## Corrosivity (Cal. Test Method 417,422,643)

| Sample Location | Sample Depth (ft.) | pH | Resistivity (Ohm-cm) | Sulfate Content (ppm) | Sulfate Content (%) | Chloride Content (ppm) | Chloride Content (%) |
|---|---|---|---|---|---|---|---|
| B-1 | 1.0'-5.0' | 8.6 | 1000 | 93 | 0.009 | 75 | 0.007 |

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**LAB TEST RESULTS**

**HOLLANDIA DIARY**
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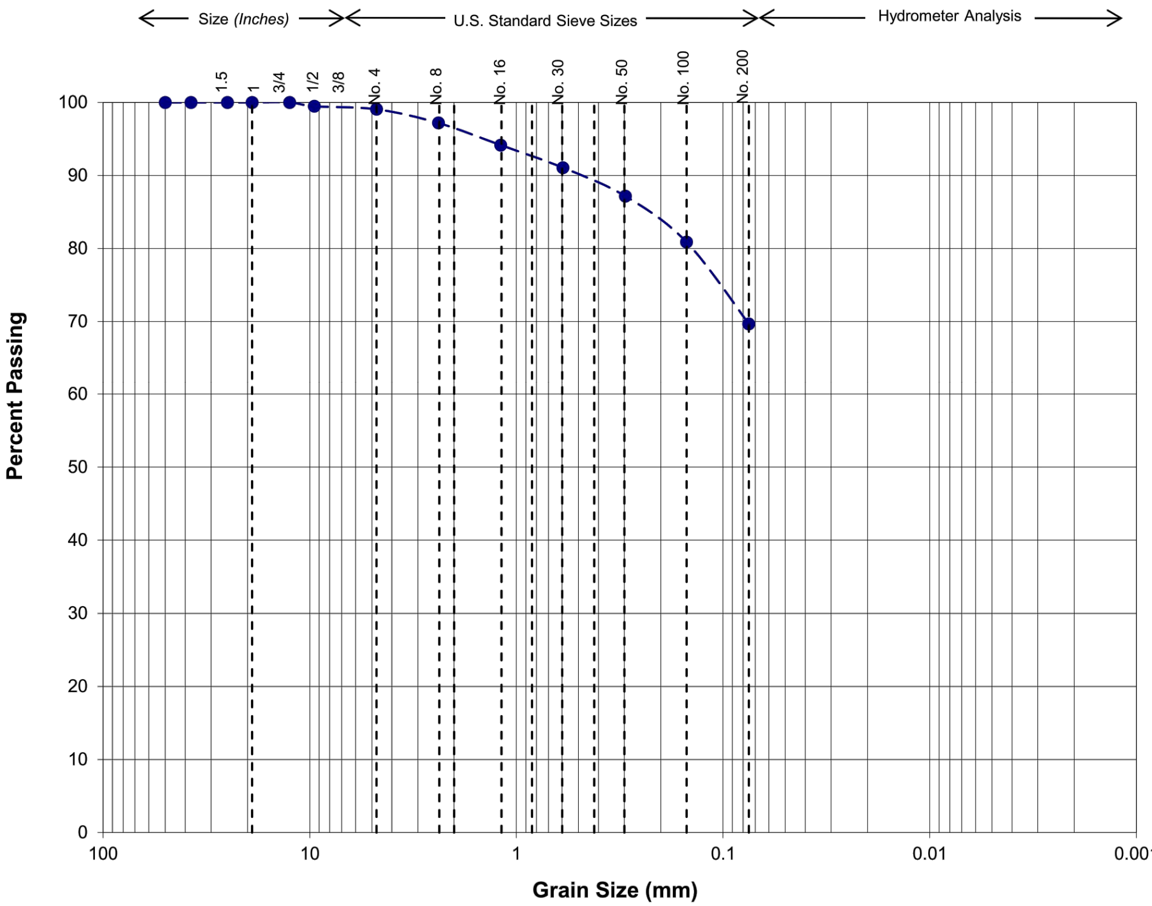
| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.2 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-1

Depth (ft): 12.0'-15.0'

USCS Soil Type: CL

Passing No. 200 (%): 70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# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**

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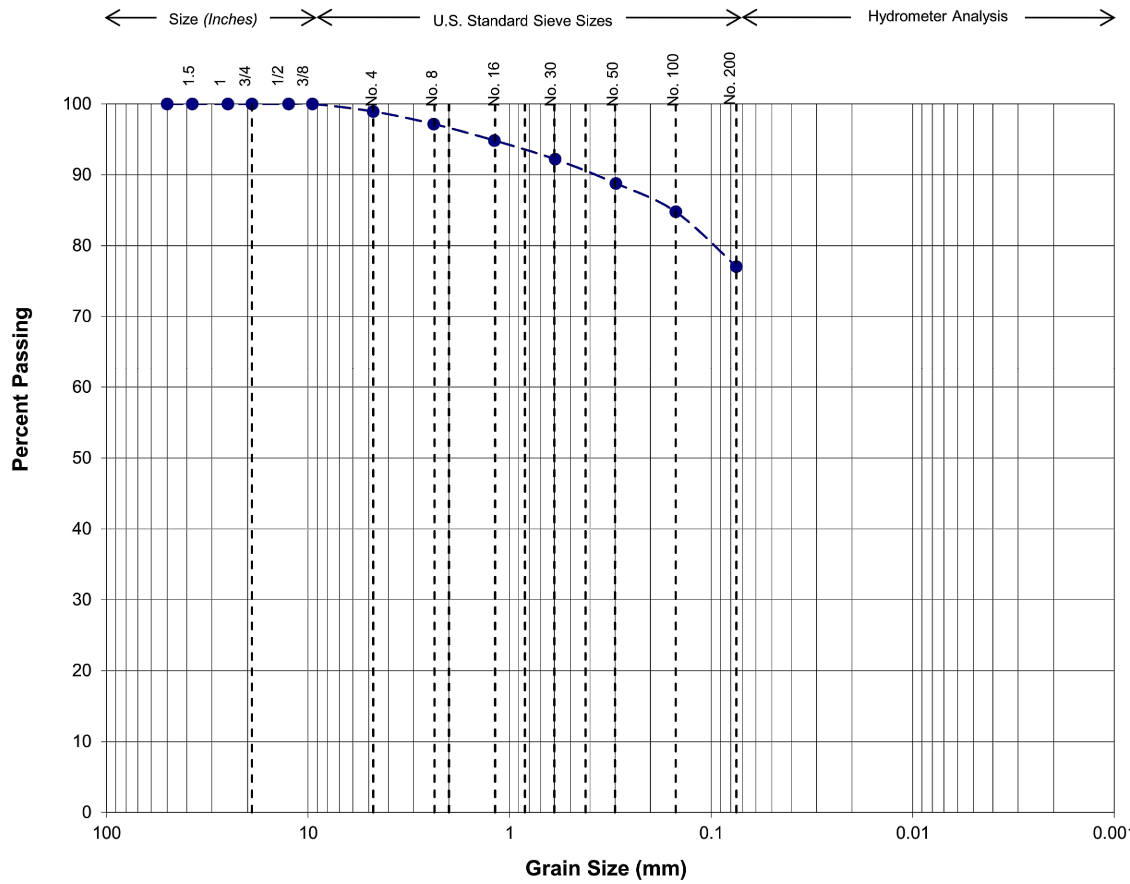
| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.3 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

| | |
|---|---|
| Sample Location: | B-1 |
| Depth (ft): | 2.5' |
| USCS Soil Type: | CL |
| Passing No. 200 (%): | 77 |

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## GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**
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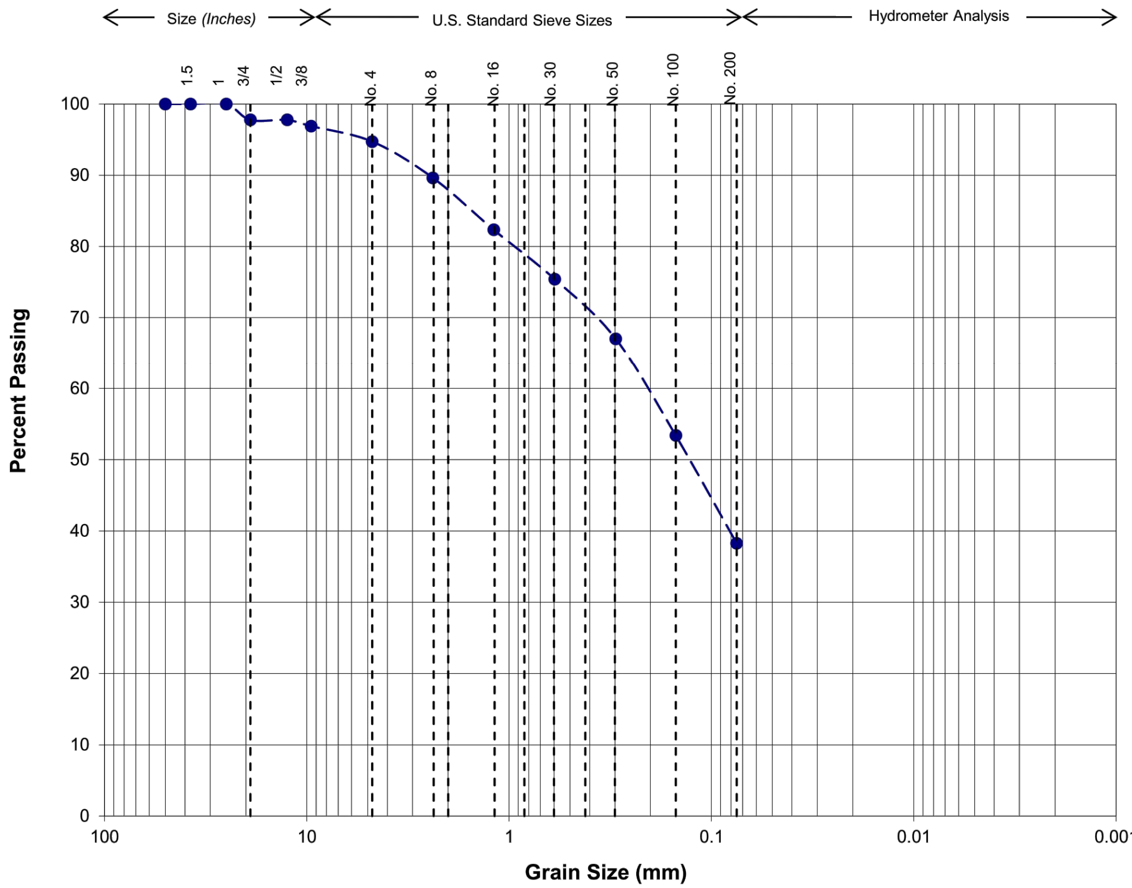
| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.4 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-1

Depth (ft): 20.0'

USCS Soil Type: SM

Passing No. 200 (%): 38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# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**

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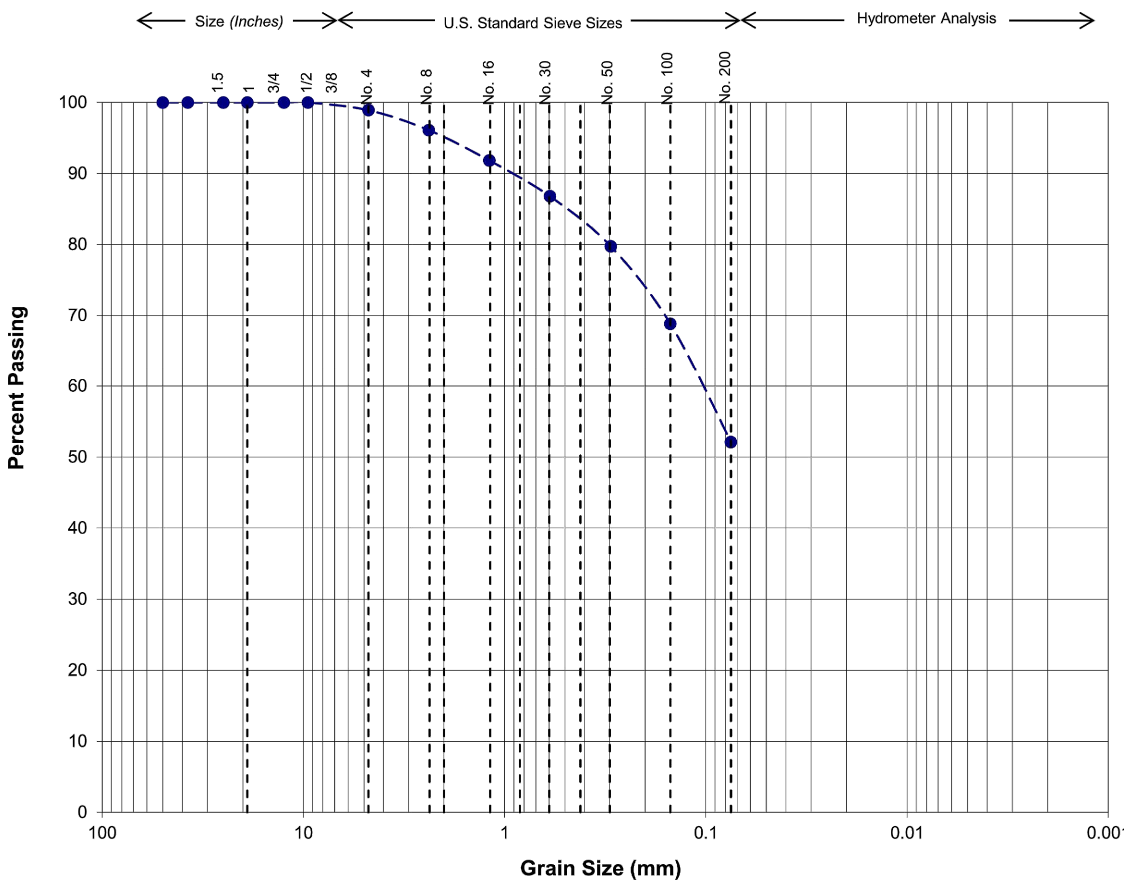
| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.5 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-1

Depth (ft): 25.0'

USCS Soil Type: ML

Passing No. 200 (%): 52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## GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**

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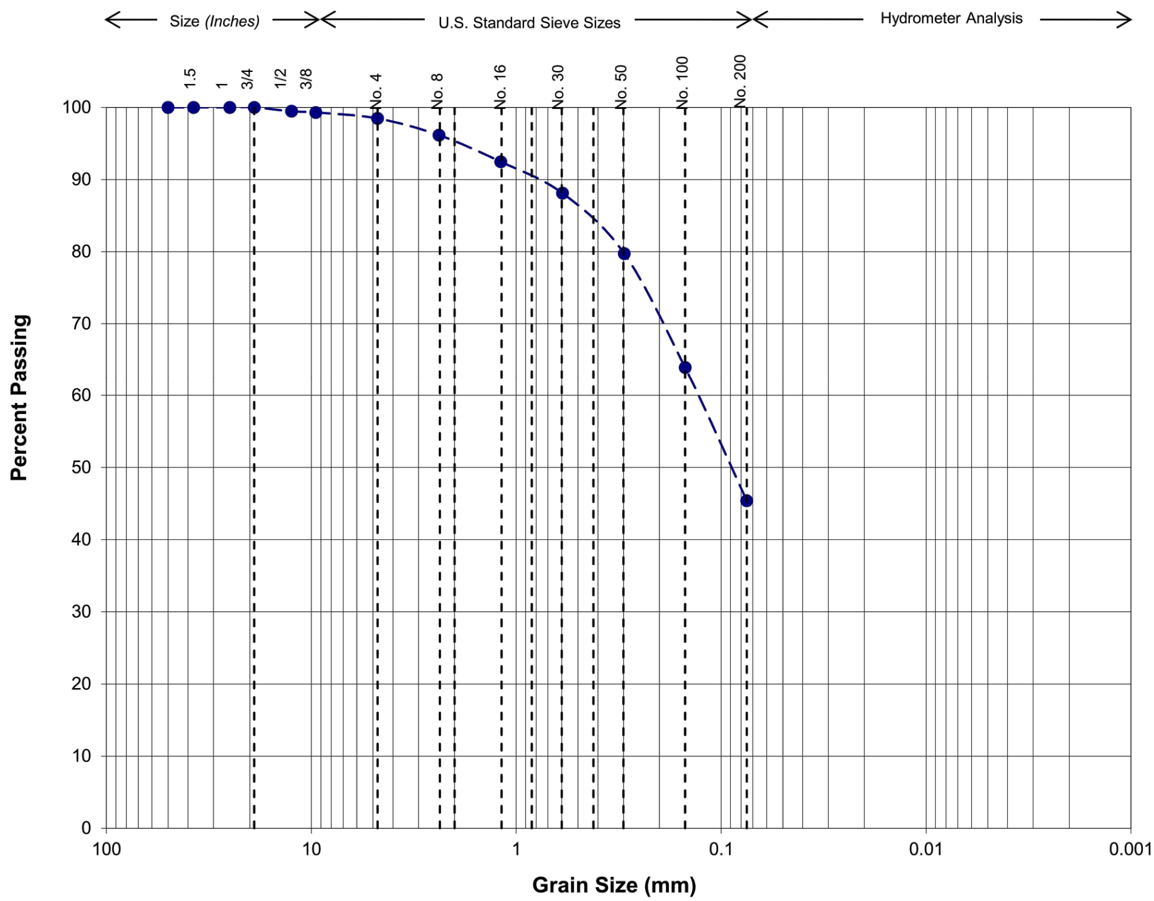
| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.6 |
|---|---|---|---|




<table>
<tr><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Sample Location: B-1

Depth (ft): 30.0'

USCS Soil Type: SM-ML

Passing No. 200 (%): 45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# GRADATION ANALYSIS TEST RESULTS

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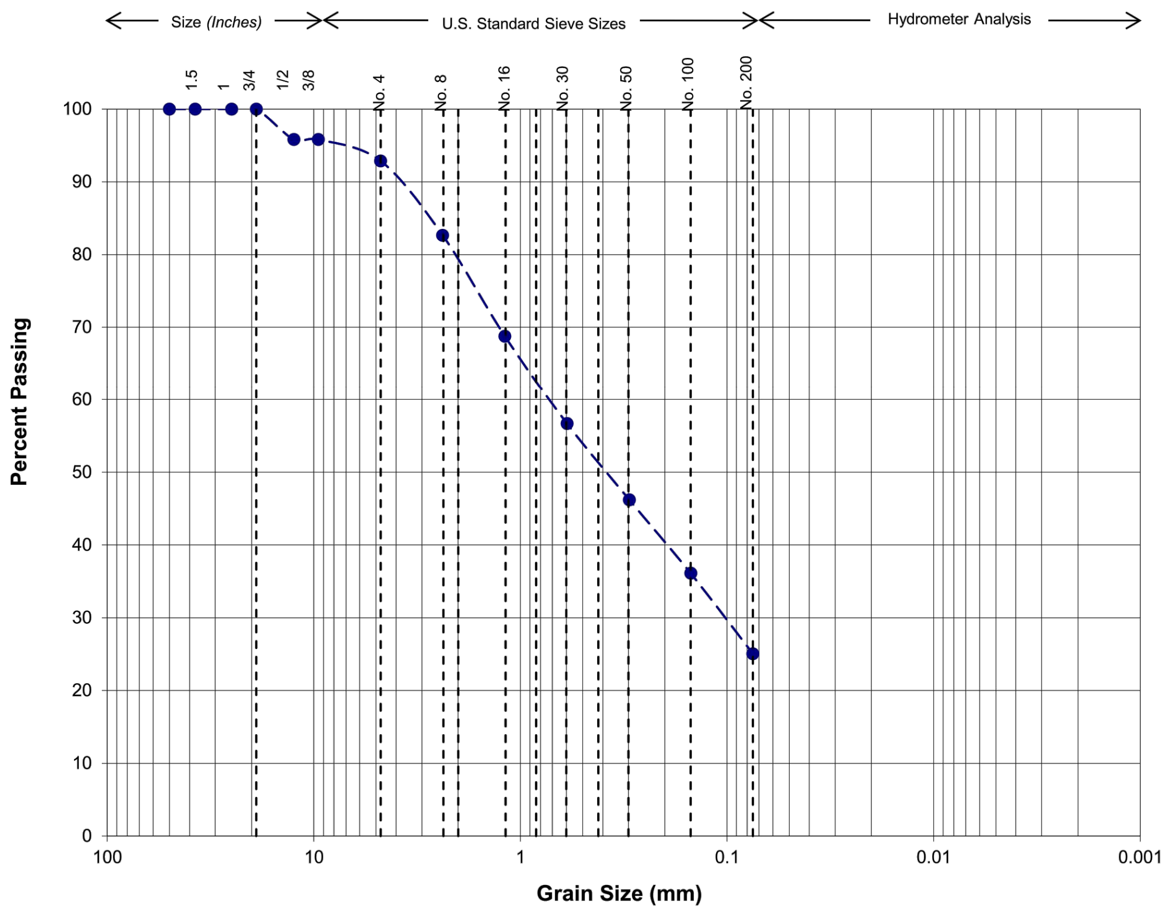
| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.7 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-1

Depth (ft): 35.0'

USCS Soil Type: SM

Passing No. 200 (%): 25

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# GRADATION ANALYSIS TEST RESULTS

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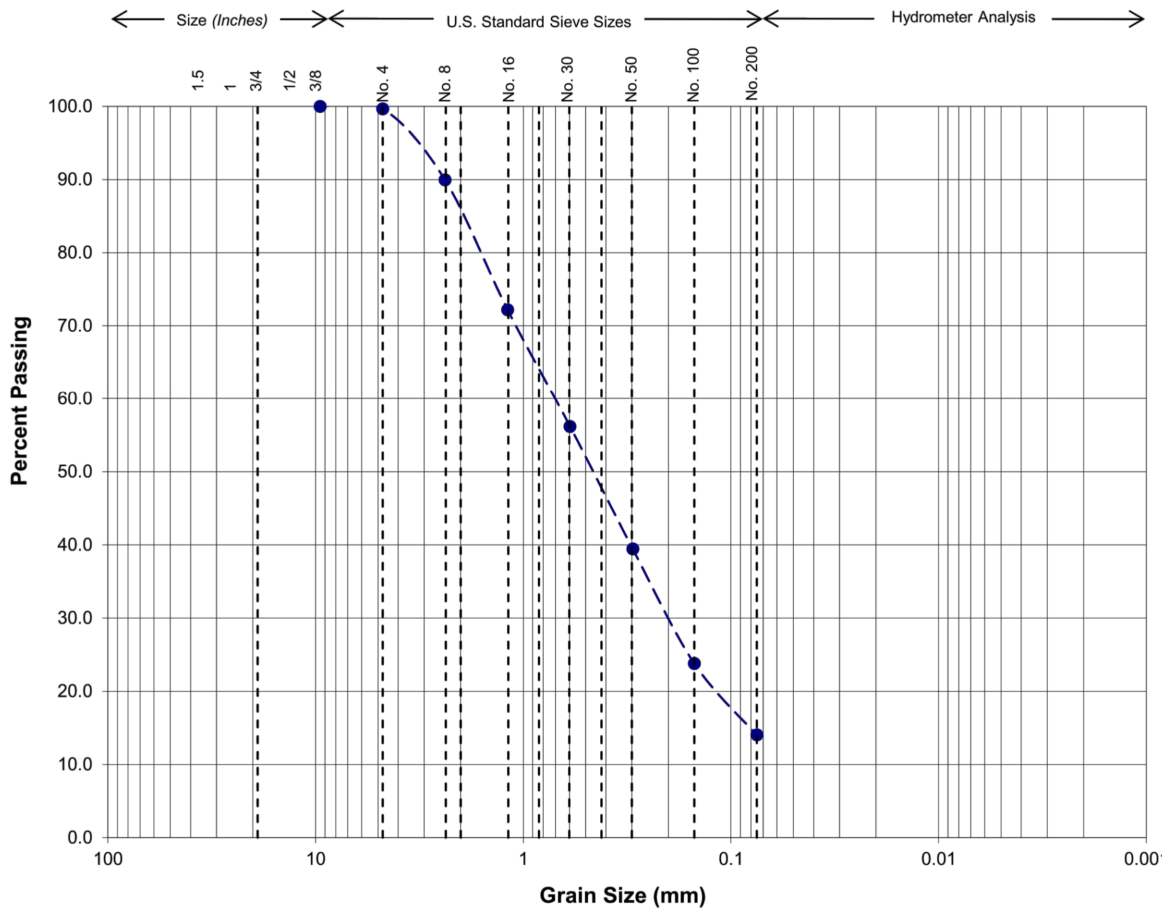
| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.8 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-1

Depth (ft): 40.0'

USCS Soil Type: SM

Passing No. 200 (%): 14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## GRADATION ANALYSIS TEST RESULTS

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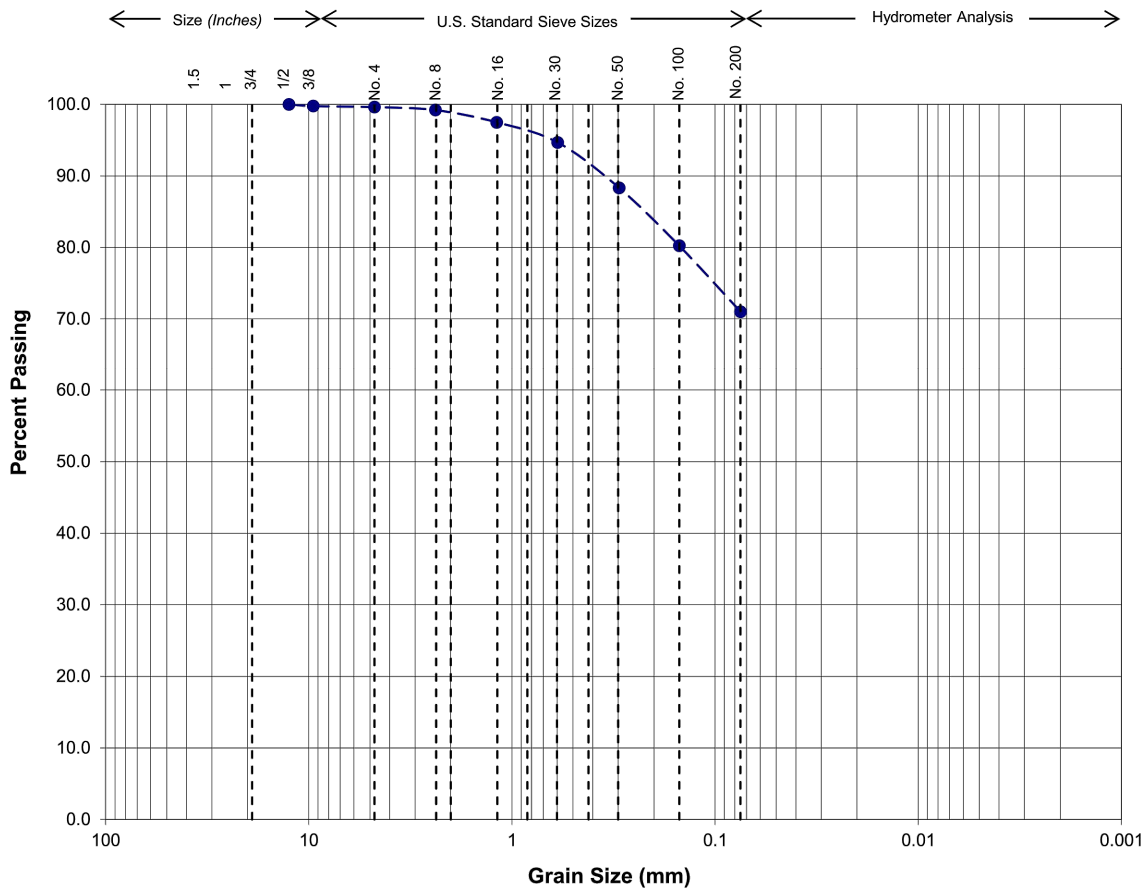
| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.9 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-2

Depth (ft): 10.0'

USCS Soil Type: CL-ML

Passing No. 200 (%): 71

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# GRADATION ANALYSIS TEST RESULTS

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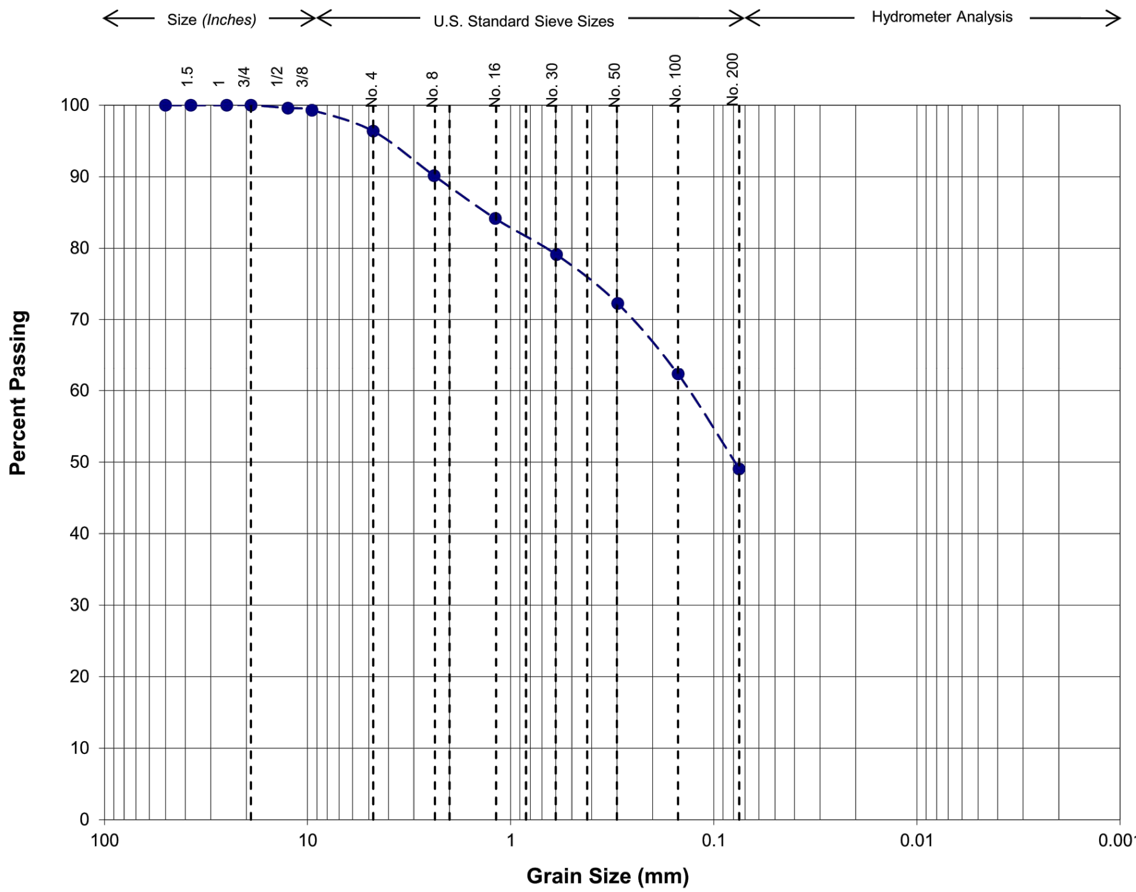
| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.10 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-2

Depth (ft): 15.0'

USCS Soil Type: SM-ML

Passing No. 200 (%): 49

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4373 VIEWRIDGE AVENUE, SUITE B

SAN DIEGO, CALIFORNIA

PHONE: 858-292-7575 FAX: 858-292-7570

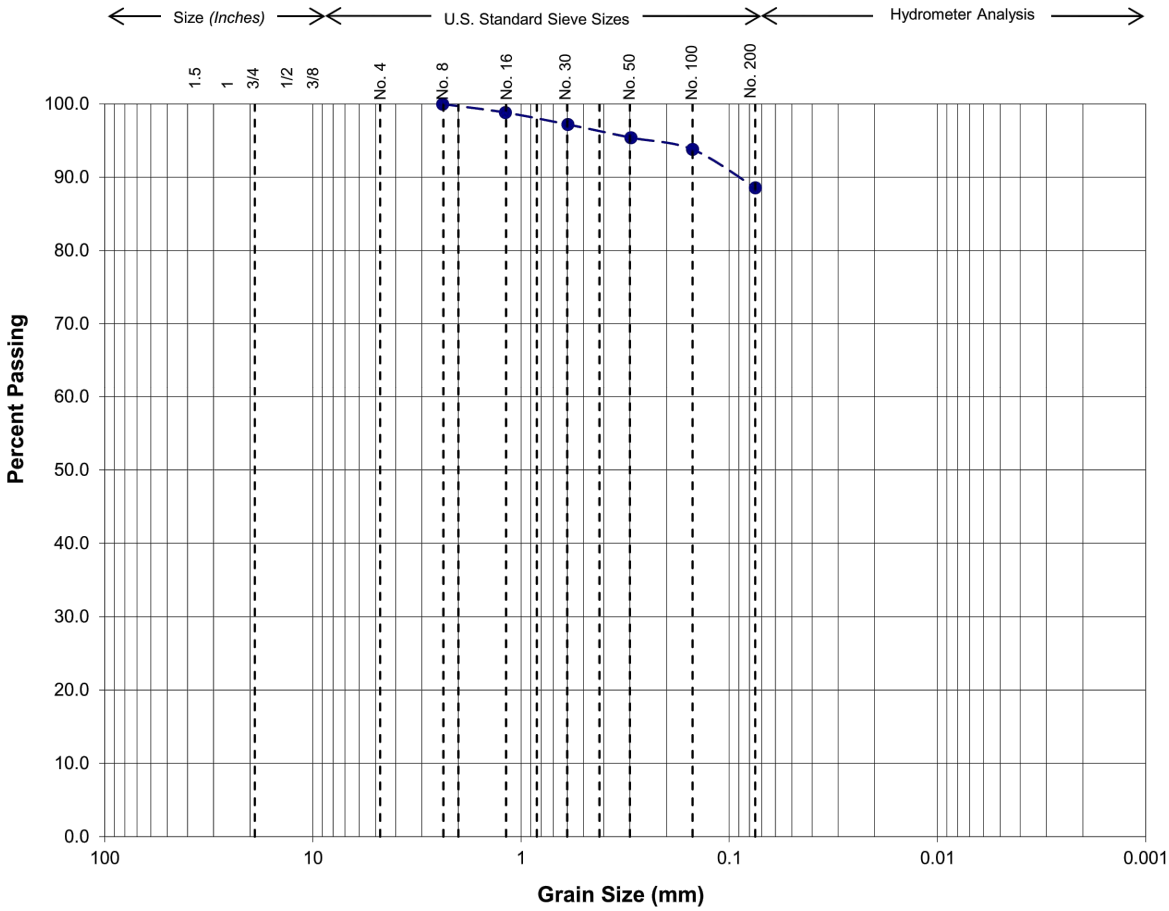
# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**

622 E. MISSION ROAD

SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.11 |
|---|---|---|---|




<table>
<tr><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

| | |
|---|---|
| Sample Location: | B-2 |
| Depth (ft): | 20.0' |
| USCS Soil Type: | ML |
| Passing No. 200 (%): | 89 |


NOVA

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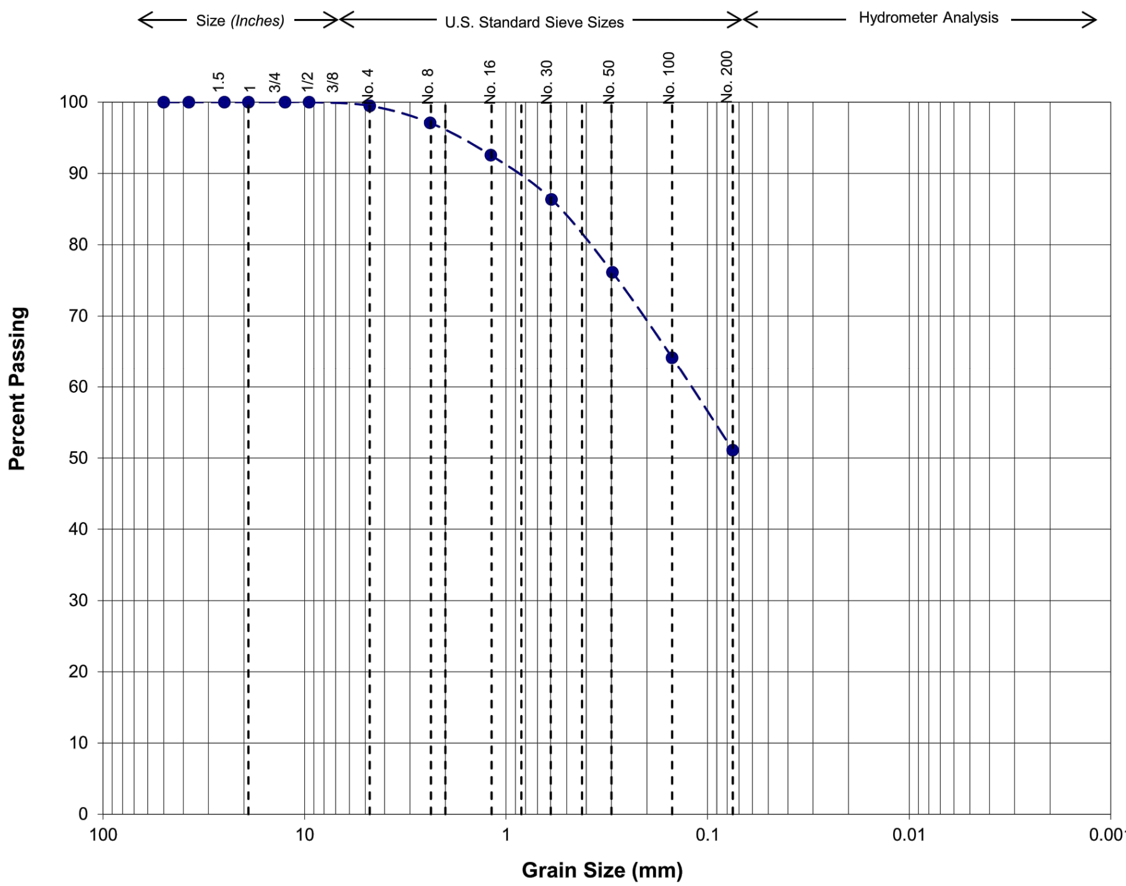
# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.12 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

| | |
|---|---|
| Sample Location: | B-2 |
| Depth (ft): | 30.0' |
| USCS Soil Type: | ML |
| Passing No. 200 (%): | 51 |

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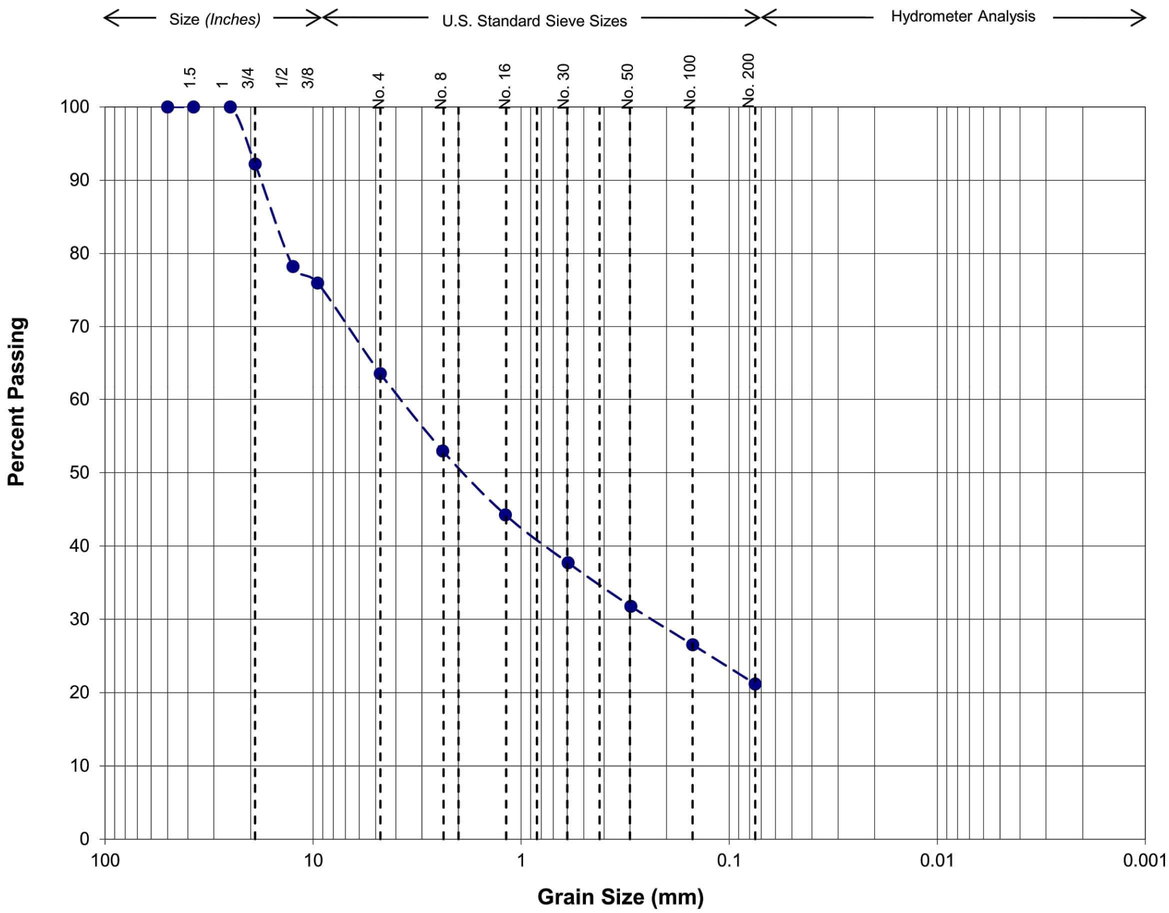
# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.13 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-2

Depth (ft): 38.0'

USCS Soil Type: SM

Passing No. 200 (%): 21

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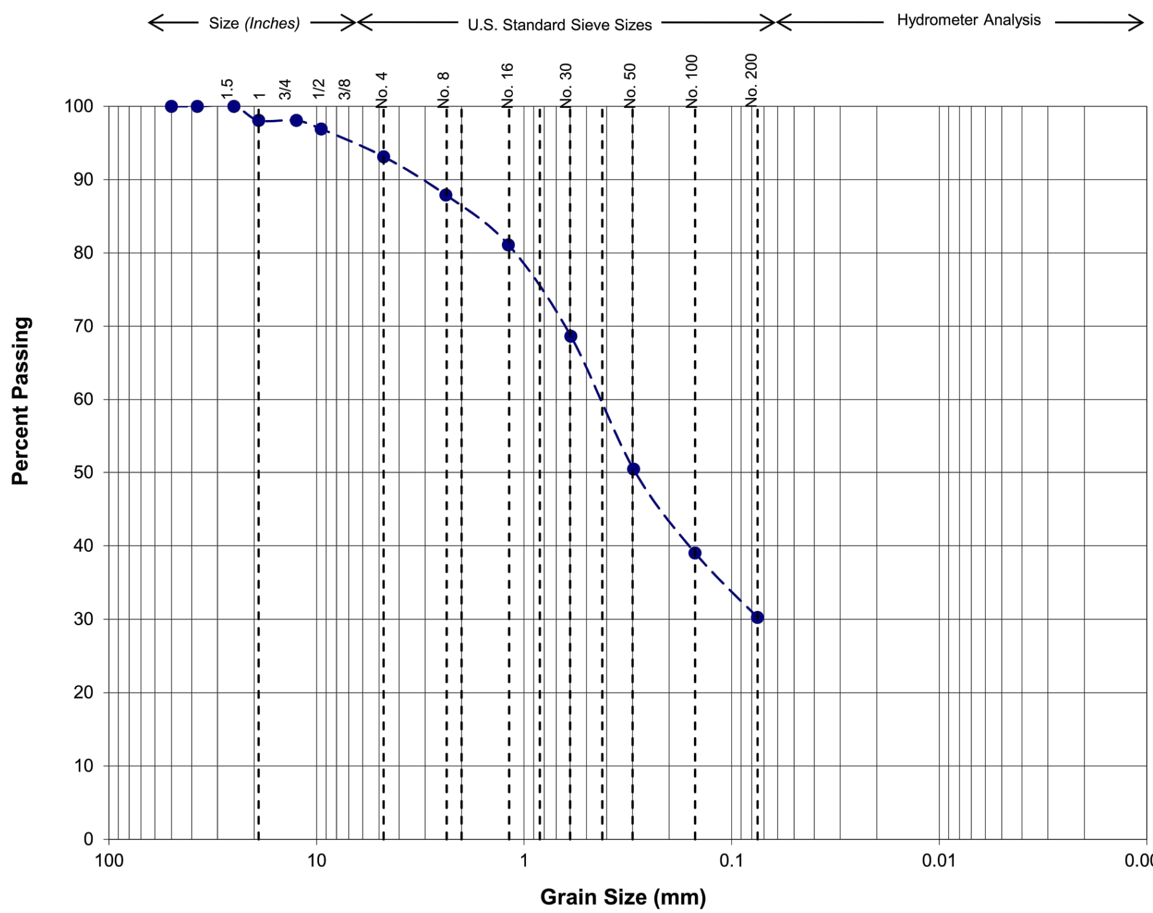
# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**

622 E. MISSION ROAD

SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.14 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-3

Depth (ft): 1.0'

USCS Soil Type: SC

Passing No. 200 (%): 30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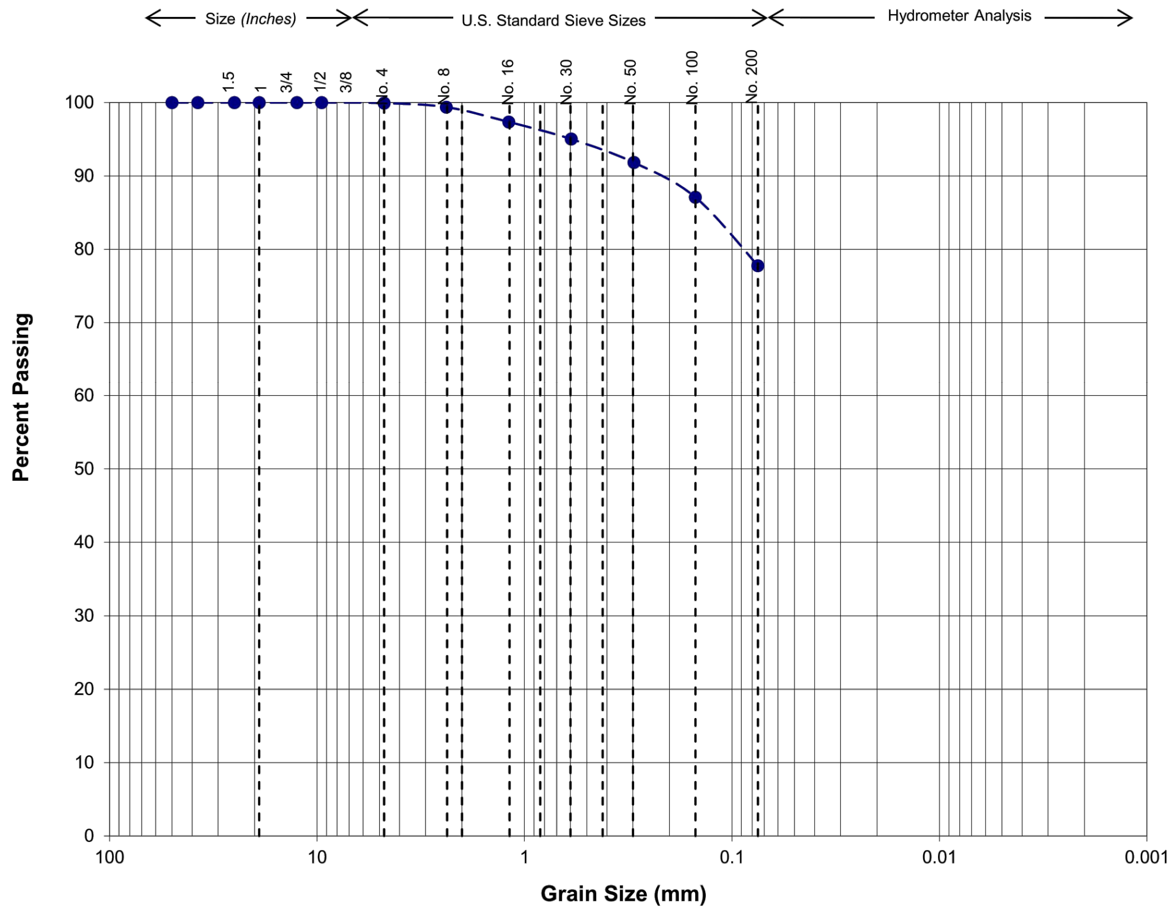
# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**

622 E. MISSION ROAD

SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.15 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-3

Depth (ft): 7.5'

USCS Soil Type: CL

Passing No. 200 (%): 78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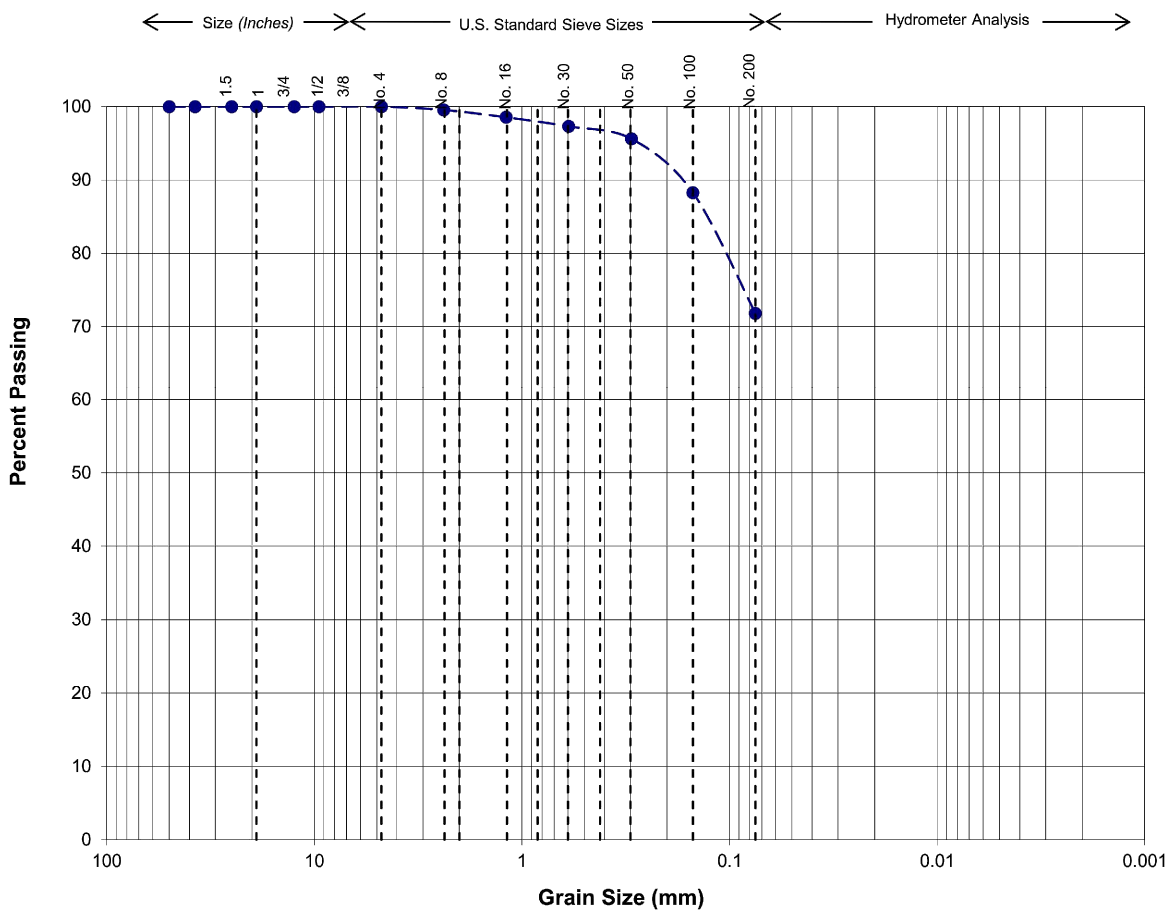
# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**

622 E. MISSION ROAD

SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.16 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-3

Depth (ft): 15.0'

USCS Soil Type: CL

Passing No. 200 (%): 72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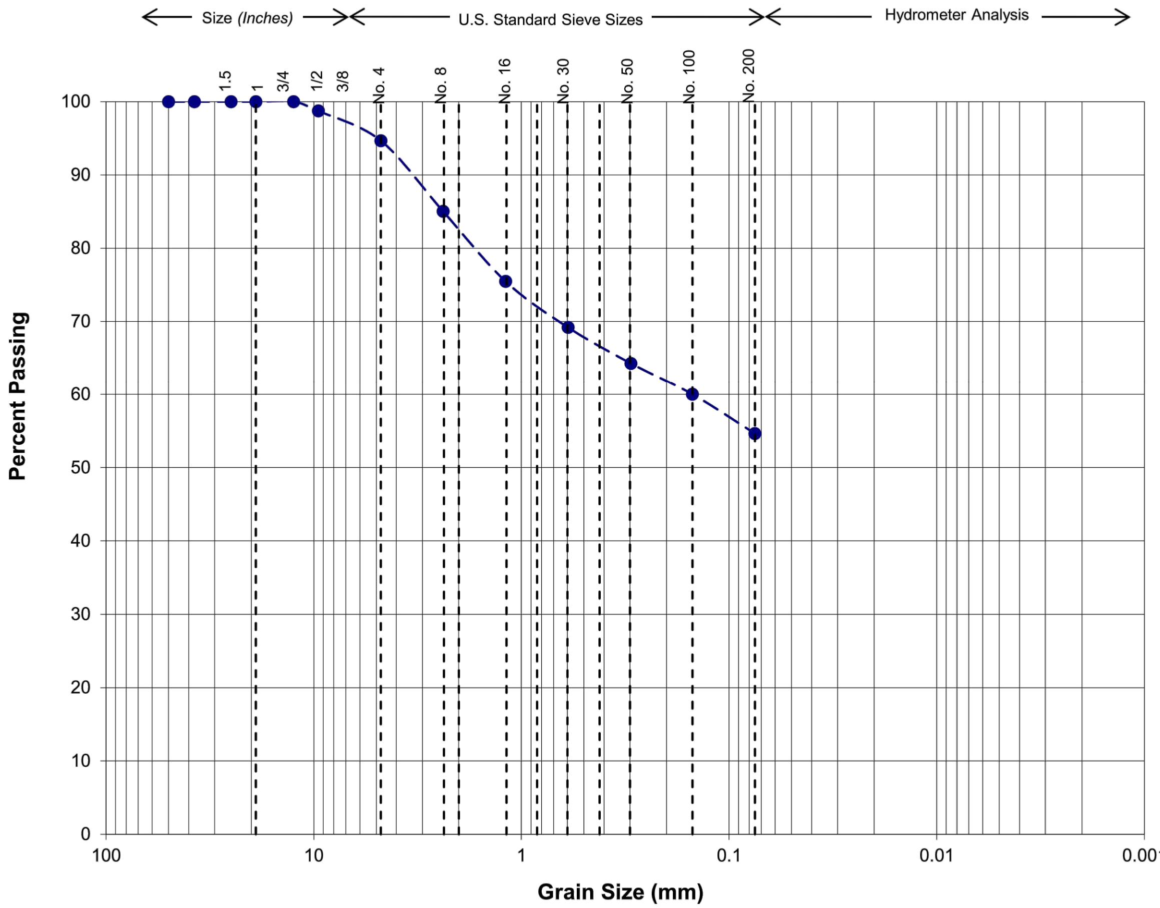
# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**

622 E. MISSION ROAD

SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.17 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-4

Depth (ft): 1.0'

USCS Soil Type: CL

Passing No. 200 (%): 55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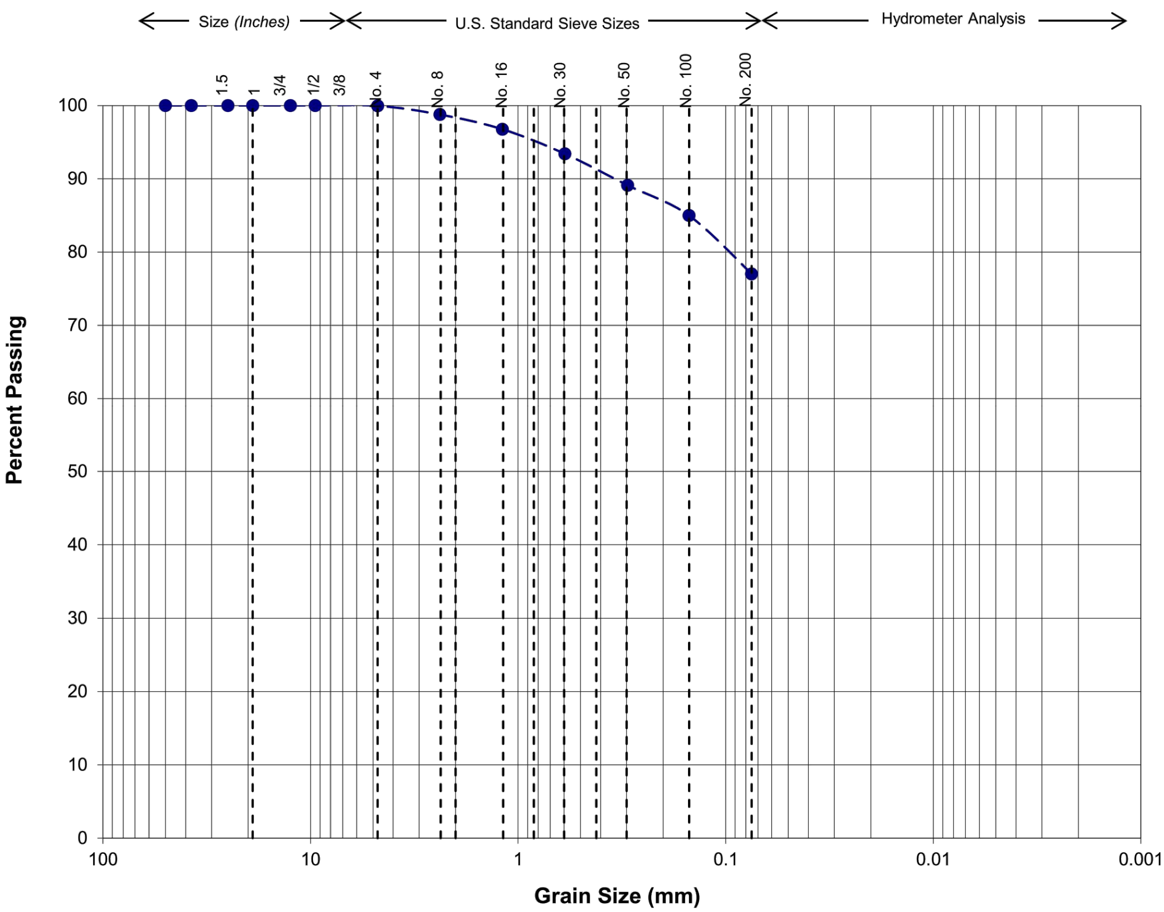
# GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**

622 E. MISSION ROAD

SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.18 |
|---|---|---|---|




| Gravel | | Sand | | | Silt or Clay |
|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | |

Sample Location: B-4

Depth (ft): 5.0'

USCS Soil Type: ML-CL

Passing No. 200 (%): 77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## GRADATION ANALYSIS TEST RESULTS

**HOLLANDIA DIARY**
622 E. MISSION ROAD
SAN MARCOS, CALIFORNIA

| BY: DTW | DATE: APRIL 2019 | PROJECT: 2019039 | APPENDIX: D.19 |
|---|---|---|---|