REPORT
GEOTECHNICAL INVESTIGATION

Proposed 17 on Voltaire Townhouses
Voltaire Street and San Clemente Street, San Diego, CA

PREPARED FOR

CityMark Communities, LLC
3818 Park Boulevard
San Diego, CA 92103

PREPARED BY

NOVA Services, Inc.
4373 Viewridge Avenue, Suite B
San Diego, CA 92123

August 2, 2019
NOVA Project 2019147

NOVA

GEOTECHNICAL ■ MATERIALS ■ SPECIAL INSPECTI
SBE ■ SLBE ■ SCOOP

CityMark Communities, LLC
3818 Park Boulevard
San Diego, CA 92103

August 2, 2019
NOVA Project 2019147

Attention: Rich Gustafson

Subject: Report
Geotechnical Investigation
Proposed 17 on Voltaire Townhouses
Voltaire Street and San Clemente Street, San Diego, California

Dear Mr. Gustafson:

NOVA Services, Inc. (NOVA) is pleased to present herewith the above-referenced report. The report was completed by NOVA for CityMark Communities, LLC (CityMark) in accordance with NOVA's proposal dated July 2, 2019, as authorized on that date.

NOVA appreciates the opportunity to be of continued support to CityMark and its commitment to the San Diego area. If you have any questions regarding the content of this report or if we may be of assistance in any way, please do not hesitate to call.

Sincerely,

NOVA Services, Inc.

Wail Mokhtar
Senior Project Manager

Melissa Stayner, P.G., C.E.G.
Senior Geologist

CERTIFIED ENGINEERING GEOLOGIST
MELISSA STAYNER
2707
STATE OF CALIFORNIA

John F. O'Brien, P.E., G.E.
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
JOHN F. O'BRIEN
651
EXPIRES 3-31-2021
GEOTECHNICAL
STATE OF CALIFORNIA

Hillary A. Price
Project Geologist

REPORT
GEOTECHNICAL INVESTIGATION

Proposed 17 on Voltaire Townhouses
Voltaire Street and San Clemente Street, San Diego, California

TABLE OF CONTENTS

List of Plates

List of Appendices

List of Tables

List of Figures

List of Figures (continued)

1.0 INTRODUCTION

1.1 Terms of Reference

This report presents the findings of a preliminary geotechnical investigation by NOVA Services, Inc. (NOVA) for a mixed townhouse and commercial development now known as *17 on Voltaire*. The development will be sited on a parcel located at Voltaire and San Clemente Streets in San Diego.

The work reported herein was completed by NOVA for CityMark Communities, LLC in accordance with NOVA's proposal dated July 2, 2019, as authorized on that date. Figure 1-1 provides a graphic that depicts the site vicinity.

Figure 1-1. Vicinity Map

1.2 Geotechnical Work by Others

This site and the planned development thereon have been the object of a prior geotechnical study by Allied Earth Technology (reference, *Soil Investigation, Proposed Mixed-Use Apartment/Retail Complex Site, Southwest Corner of Voltaire Street And San Clemente St., San Diego, California*, Allied Earth Technology, Project 07-116B7, July 25, 2007, hereinafter 'AET 2007').

The work reported herein utilizes the indications of the test trenches completed by AET for the subsurface exploration. The recommendations provided herein supersede those provided in AET 2007.

1.3 Objectives, Scope, and Limitations of This Work

1.3.1 Objectives

The objectives of the work reported herein are twofold, as described below.

1. Objective 1, Geotechnical. Characterize the occurrence of subsurface soil and formational rock to supplement the findings of AET 2007, thereafter providing recommendations for geotechnical-related development, including foundations and earthwork.

2. Objective 2, Infiltration. Conduct percolation testing sufficient to identify requirements for development of permanent stormwater infiltration Best Management Practices ('BMPs').

1.3.2 Scope

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

1. Task 1, Background Review. Reviewed available background data regarding the site area, including geotechnical reports, topographic maps, geologic data, fault maps and reports, and preliminary development plans for the project. No structural information was available.

2. Task 2, Subsurface Exploration. The exploration included the following subtasks.

    - *Subtask 2-1, Reconnaissance*. Prior to undertaking any invasive work, NOVA conducted a site reconnaissance, including layout of subsurface explorations used to determine subsurface conditions. Underground Service Alert (USA) and a private utility locator were notified for underground utility mark-out services.

    - *Subtask 2-2, Coordination.* NOVA coordinated with CityMark regarding access and scheduling for the drilling.

    - *Subtask 2-3, Engineering Borings.* NOVA retained a specialty subcontractor to drill, log, and sample two (2) hollow-stem auger borings. A NOVA geologist directed the drilling and sampling using ASTM methods.

    - *Subtask 2-4, Percolation Testing.* A single hollow stem auger boring was located in a prospective Drainage Management Area ('DMA'). The boring was extended to about 5.5 feet below ground surface. Thereafter, the boring was converted to a well and percolation testing conducted in accordance with the City of San Diego Storm Water Standards, Part 1 BMP Design Manual, October 2018 edition.

    - *Subtask 2-5, Closure*. The completed borings and percolation test well were backfilled with drill cuttings and the area of work cleaned following drilling/testing.

3. Task 3, Laboratory Testing. Laboratory testing was conducted on representative samples of soils recovered from the engineering borings.

4. Task 4, Engineering Evaluation. The findings of Tasks 1-3 were utilized to support geotechnical evaluations relevant to the planned new construction.

5. Task 5, Reporting. Submittal of this report concludes the scope of work described in NOVA's proposal. The report provides the findings of the subsurface investigation and recommendations for foundation design, earthwork and development of stormwater infiltration BMPs.

1.3.3 Limitations

The recommendations included in this report are not final. These recommendations are developed by NOVA using judgment and opinion and based upon the limited information available from the borings. 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 or toxic materials in the soil, groundwater, or surface water within or beyond the site.

Appendix A to this report provides important additional guidance regarding the use and limitations of this report. This information should be reviewed by all users of the report.

1.4 Understood Use of This Report

NOVA expects that the findings and recommendations provided herein will be utilized in decision-making by CityMark and its design Team regarding geotechnical-related 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 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.

1.5 Report Organization

The remainder of this report is organized as abstracted below.

- Section 2 reviews available project information.
- Section 3 describes the field investigation and laboratory testing.
- Section 4 describes the surface and subsurface conditions.
- Section 5 reviews geologic, soil and siting-related hazards common to this area of California, considering each for its potential to affect construction and long-term use of the development.
- Section 6 provides recommendations for earthwork and foundation design.
- Section 7 provides recommendations for development of stormwater infiltration BMPs.
- Section 8 provides recommendations for use of permeable pavers.
- Section 9 provides recommendations for development of pavements
- Section 10 lists the principal references utilized in the development of the report.

Figures and tables are embedded in the text of the report at the point which they are referenced. Plate 1, provided immediately following the text of this report, shows the location of field work in larger scale.

The report is supported by four appendices. Appendix A provides guidance regarding the use and limitations of this report. Appendix B presents logs of NOVA's borings & AET trench logs. Appendix C provides the records of the laboratory testing. Appendix D provides an Infiltration Feasibility Condition Letter and Worksheet C.4-1: Form I-8A.

2.0 BACKGROUND

2.1 Site Description

2.1.1 Location

The residential townhouse and commercial development are proposed to be constructed on four parcels located southwest of the intersection of Voltaire Street and San Clemente Street in San Diego (hereinafter, also referenced as 'the site'). The site is bounded to the north by Voltaire Street, to the east by San Clemente Street, to the south by an alleyway, and to the west by commercial and residential development.

Figure 2-1 depicts the site location and limits.

Figure 2-1. Site Location and Limits

2.1.2 Current and Past Site Use

The site is comprised of a collection of four parcels with the following APNs: 449-251-05, -06, -07 and -08-00. The eastern parcels are currently occupied by a pet care business and a surfboard repair business. The western parcels are vacant, used by the neighborhood as community gardens.

Aerial photos from 1964 and 1972 indicate that there were residential structures across this property. By 1980, the structures on the western half of the property are not visible, and the existing buildings are shown in their current configuration on the eastern portion of the site. The gardens on the western portion of the site were planted around 2012.

2.2 Planned Development

2.2.1 General

NOVA's understanding of current planning for the development is based upon review of:

1. Architectural documentation developed by The McKinley Associates (reference, *17 on Voltaire, CityMark, Architectural Submittal Package,* The McKinley Associates, Inc., 14 June 2019, hereinafter 'TMA 2019').

2. Civil Plans developed by Pasco Laret Suiter& Associates (reference, *17 on Voltaire, Site Development Permit/Map waver,* Pasco Laret Suiter& Associates, 7 June 2019, hereinafter 'PLSA 2019').

TMA 2019 indicates planning for a proposed residential townhouse and commercial development that will include the construction of two 3-story townhouse buildings and commercial space. The buildings will accommodate a total of 17 townhouses, ranging from 1,375 sf to 1,662 sf. Commercial space will be about 2,879 sf. The development will provide parking for 44 vehicles in a partially below-grade basement garage.

Figure 2-2 shows an elevation view of the development, depicting the manner by which the buildings will be adapted to the existing groundform.

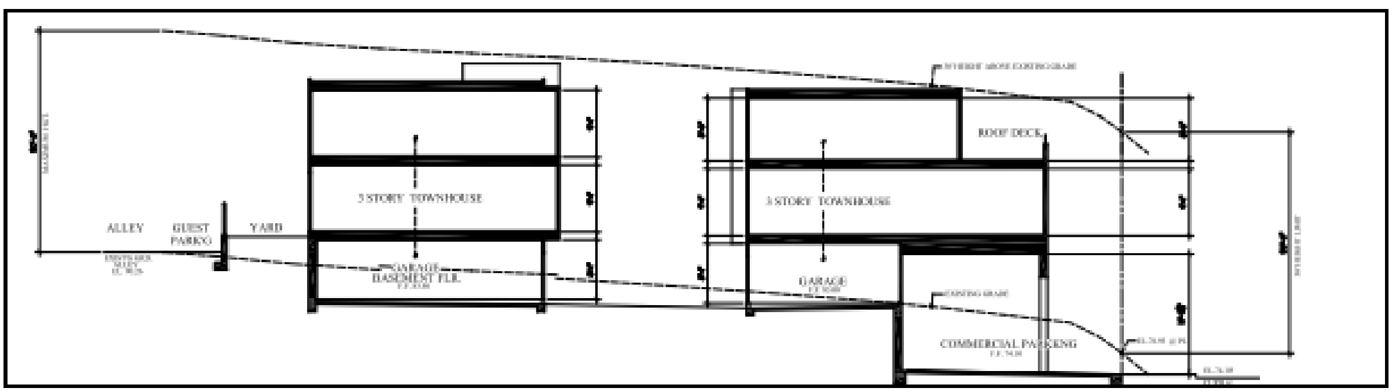


Figure 2-2. Representative Building Section
(source: TMA 2019)

2.2.2 Structural

Structural information regarding the planned additions is not yet available. However, it is expected that foundation loads will be relatively light, characteristic of this genre of residential construction.

2.2.3 Potential for Earthwork

Development of the site will include demolition of the existing structures, trees, and pavement as well as removal or relocation of existing utilities. Detailed planning regarding civil development of the site and related earthwork was not available for review by NOVA. However, based on cursory review it appears that earthwork will be limited to performing the required excavations to achieve pad grades, but is expected to result in a net export.

The majority of earthwork for this project will include cutting pads to grade, and constructing and backfilling retaining walls.

2.2.4 Stormwater

The *Preliminary Site Drainage Plan* prepared by Pasco Laret Suiter & Associates (PLSA 2019) indicates the use of biofiltration planters on the eastern and western sides of the proposed buildings. Permeable pavers are also indicated between Buildings A and B, as well as along the southern property boundary adjacent to the alley.

3.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

3.1 Overview

The subsurface exploration was completed on July 11th and 12th, 2019. The work included drilling and sampling of two engineering borings (referenced as 'B-1' and 'B-2') and conducting one percolation test ('P-1'). This work supplements the initial exploration of the site by excavation of five test trenches ('T-1' through 'T-5'), as reported in AET 2007.

The engineering borings were completed by a specialty subcontractor working under the surveillance of a NOVA geologist. Figure 3-1 presents a plan view of the development, indicating the location of the subsurface exploration by NOVA and that reported in AET 2007. Plate 1, provided immediately following the text of this report, shows the location of this work in larger scale.

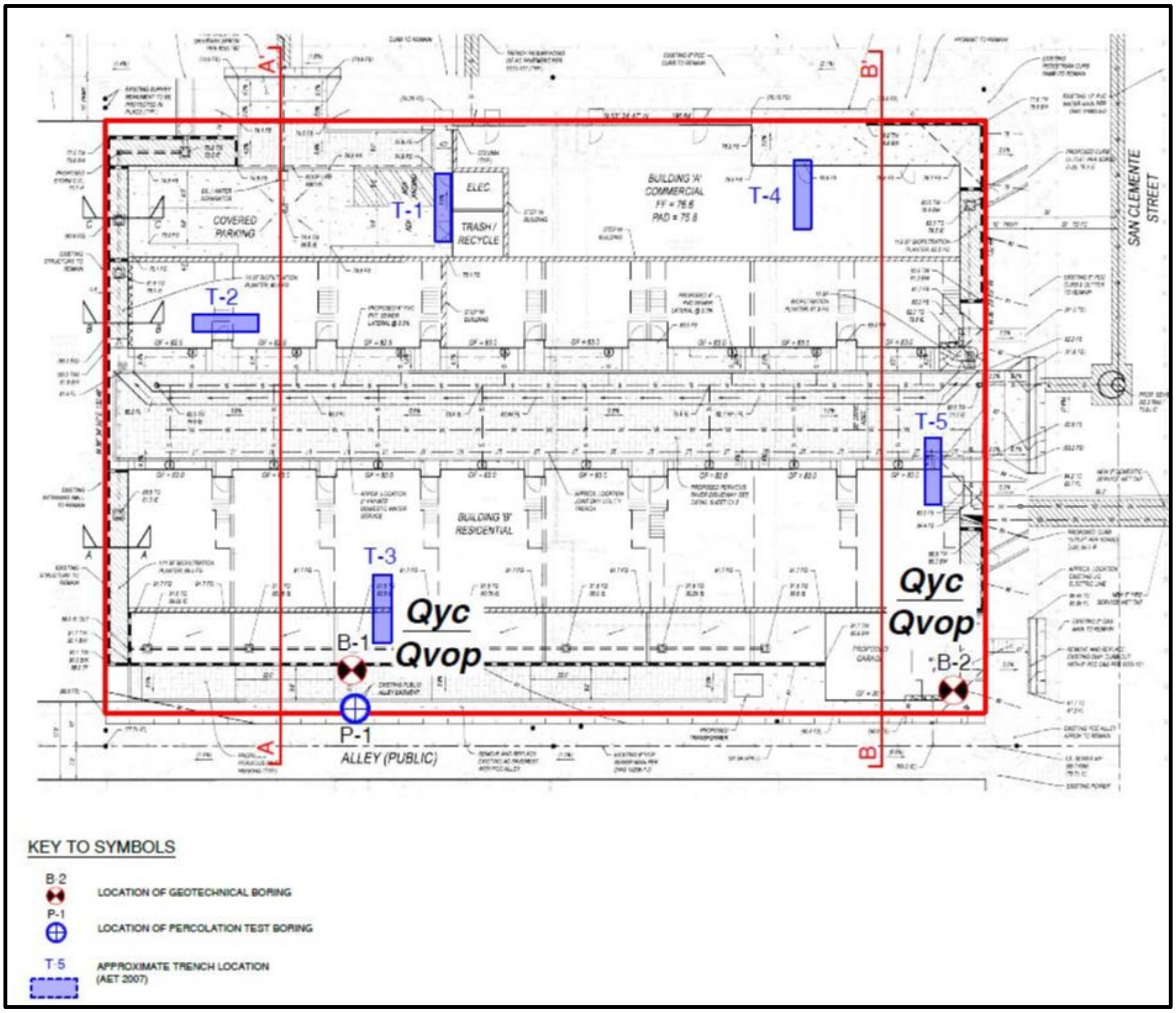


Figure 3-1. Location of Engineering Borings, Test Trenches, and Percolation Test

The remainder of this section provides detail regarding the engineering borings (Section 3.2), test pits by others (Section 3.3), percolation testing (Section 3.4) and related laboratory testing (Section 3.5).

3.2 Engineering Borings by NOVA

3.2.1 General

Two (2) hollow-stem auger borings were drilled to depths of 17 feet and 19.5 feet below ground surface (bgs) on July 11th and 12th, 2019. The borings were drilled under the surveillance of a NOVA geologist. Samples recovered from the borings were delivered to NOVA's materials laboratory for analysis.

The engineering borings were advanced by a truck-mounted drilling rig utilizing hollow-stem auger drilling equipment. Boring locations were determined in the field by the NOVA geologist. Elevations of the ground surface at the boring locations were estimated. Table 3-1 provides an abstract of the engineering borings.

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 Formation (feet)
B-1	+89	17	+72	3.5
B-2	+89	19.5	+69.5	2.5

Notes to Table 3-1:
1. Elevations are approximate and should be reviewed
2. 'Formation' is the Very Old Paralics (Qvop, formerly the 'Bay Point Formation')

Figure 3-2 (following page) depicts drilling operations on July 11.

3.2.2 Logging and Sampling

The geologist directed sampling and maintained a log of the subsurface materials that were encountered. Both disturbed and relatively undisturbed samples were recovered from the borings as 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 each 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 strength and compressibility.

3. Bulk samples were recovered from the near subsurface.

3.2.3 Closure

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

Figure 3-2. Drilling Operations, July 11, 2019

3.3 Review of Test Trenches by Others

AET 2007 reported the findings of a series of five backhoe-excavated test trenches. The approximate locations of these trenches are depicted on Figure 3-1. Table 3-2 provides an abstract of the test trenches.

Table 3-2. Abstract of the Test Trenches Reported in AET 2007

Trench Reference	Total Depth Below Ground Surface (feet)	Depth to Formation (feet)
T-1	12	4
T-2	10	2
T-3	7	4.5
T-4	5	3
T-5	5	2

Notes to Table 3-2:
1. 'Formation' is the Very Old Paralics (Qvop, formerly the 'Bay Point Formation')
2. AET 2007 does not estimate ground elevations at the test trenches.
3. No groundwater reported in any of the test trenches.
4. Refusal of the Case 580D excavator with 24" bucket on dense, cemented sandstone in T-3, T-4, T-5.

As may be seen by comparison of Table 3-2 with Table 3-1, AET 2007 reports subsurface conditions similar to that encountered by the NOVA borings. A veneer of colluvium typically three feet to four feet in thickness overlies dense formational sandstones.

3.4 Percolation Testing

3.4.1 General

NOVA directed the excavation and construction of one (1) percolation test well following the recommendations for percolation testing presented in the City of San Diego Storm Water Standards, Part 1 BMP Design Manual, October 2018 edition. The percolation test location is shown on Figure 3-1.

3.4.2 Drilling

The boring for the well was drilled with an 8-inch hollow stem auger to a depth of 5.5 feet bgs. Field measurements were taken to confirm that the boring was excavated to approximately 8-inches in diameter. The boring was logged by a NOVA geologist, who observed and recorded exposed soil cuttings and the boring conditions.

3.4.3 Conversion to Percolation Well

Once the boring was drilled to the desired depth, the boring was converted to a percolation test well by placing an approximately 2-inch layer of ¾-inch gravel on the bottom, 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 well was pre-soaked by filling the hole with water to at least 5 times the hole's radius. In the test well, the pre-soak water did not percolate at least 6 inches into the soil unit within 25 minutes; therefore, the hole was 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, or until the water percolation stabilized after each reading (minimum of 12 readings). At the beginning of each half-hour test period, the water level was filled to approximately the same starting water level of the previous tests in order to maintain a near-constant head during the entire testing period.

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

Table 3-3. Abstract of the Percolation/Infiltration Testing

Boring	Approximate Ground Elev. (feet, msl)	Depth of Test (feet)	Approximate Test Elev. (feet, msl)	Percolation Rate (inches/hour)	Infiltration Rate (inches/hour)	Design Infiltration Rate (in/hour, F=2*)
P-1	+89	5.5	83.5	1.92	0.08	0.04

Notes: (1) elevation is approximate
(2) the referenced geologic unit is Very Old Paralic Deposits (Qvop).

3.4.5 Closure

At the conclusion of the percolation testing, the PVC pipe was removed and the resulting hole was backfilled with soil cuttings and patched 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 testing in general accordance with ASTM standards.

Records of the geotechnical laboratory testing by NOVA are provided in Appendix C.

3.5.2 Compaction

AET 2007 reports testing two bulk samples of the colluvium that mantles the site to determine the moisture-density relationship after ASTM D 1557. This testing is abstracted on Table 3-4 (following page).

Table 3-4. Abstract of Compaction Testing After ASTM D 157 Reported in AET 2007

Test Trench	Depth (feet)	Soil Description	Maximum Dry Density, γ_D (lb/ft^3)	Optimum Moisture Content, *w* (Pct Dry Weight)
T-3	2.5	Brown/gray sandy clay (SC)	122	11.5
T-4	1.5	Brown silty sand (SM)	124	9.5

3.5.3 Expansion Potential

AET 2007 reports testing after ASTM D 4829 to determine expansion index (EI) of the clayey fraction of the colluvium that mantles the site. This testing indicates EI = 71, indicating a soil with 'Medium' expansion potential.

3.5.4 Plasticity

The visual classifications were supplemented by index testing to determine plasticity. Atterberg limits testing after ASTM D 4318 of the clayey fraction of the colluvium (Boring 1, 1-5 feet to 3 feet depth) indicated a liquid limit (LL) of LL = 33 and a plasticity index (PI) of PI = 20. As is summarized below, this sample was shown to have 45% by weight silt and clay-sized soils.

3.5.5 Soil Gradation

Mechanical gradation of two soil samples is summarized below.

Table 3-5. Abstract of the Gradation Testing

Boring	Depth (feet)	Soil Description	Percent by weight Finer Than the U.S. No. 200 Sieve	Classification After ASTM D 2487
B-1	1.5 - 3	Colluvium: Olive/gray sandy clay to clayey sand	45	SC-CL
B-2	5 – 7	Brown silty sandstone	26	SM

3.5.6 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, but the soils are potentially severely corrosive to buried metals based on resistivity measurements. Section 6 discusses the indications of the chemical testing in more detail.

4.0 SUBSURFACE CONDITIONS

4.1 Geologic Setting

4.1.1 Regional

The site 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.

The western portion of the province in San Diego County that includes the site area is underlain by Quaternary-age surficial deposits which are in turn underlain by sedimentary rocks of Late Cretaceous, Eocene, and Pliocene age. The Tertiary and Quaternary sedimentary rocks were deposited on upper Cretaceous sedimentary rocks in a basin known as the San Diego embayment. The most abundant rocks in the embayment are gently folded and faulted Eocene marine, lagoonal and nonmarine rocks.

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

Geologic units encountered during the subsurface investigation include colluvium (Qyc) and Very Old Paralic deposits (Qvop). The colluvial soils were deposited by gravity, and occur along the lower reaches of most hillsides in the area. These deposits are characteristically loose sandy clay, clayey sand, and silty sand. Cobbles and occasional boulders can also be encountered.

The Very Old Paralic deposits (Qvop) are mapped to occur widely in this portion of San Diego (see Figure 4-1, following page). These late to middle Pleistocene-aged deposits consist mainly of strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate. Variations in soil type represent episodes of deposition in offshore bar, estuarine and nearshore terrestrial and marine abrasion platform environments during that time. Differently numbered paralic deposits (evident by review of Figure 4-1) designate different ages and elevations of abrasion platforms.

The paralic deposits are competent as a foundation material, of relatively higher strength and low compressibility. Many of the monumental civil structures in San Diego are founded on this unit.

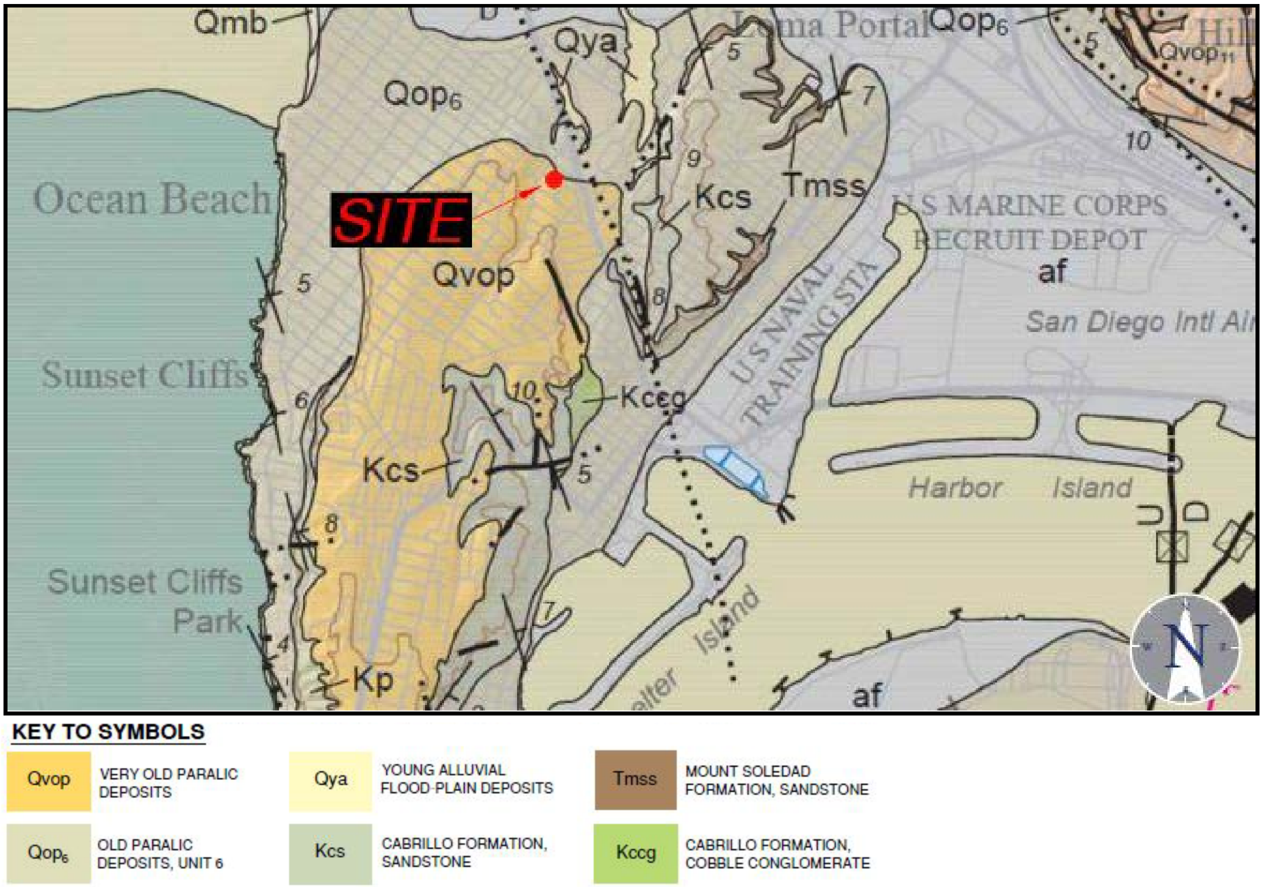


Figure 4-1. Geologic Mapping of the Site Vicinity

4.2 Site-Specific Conditions

4.2.1 Surface

The four parcels that comprise the site include both undeveloped and developed land. The eastern parcels are currently occupied by a pet care business and a surfboard repair business. The western parcels are undeveloped, occupied by neighborhood community gardens.

Elevations across the site onsite range from about +92 feet mean sea level (msl) along the southerly property line, to about +82 msl along the northerly property line paralleling Voltaire Street. There is a low slope approximately 3 to 4 feet in height fronting Voltaire Street.

Figure 4-2 (following page) provides a photograph depicting surface conditions.

Figure 4-2. Surface Conditions Looking South from Voltaire Street

4.2.2 Subsurface

For the purposes of this report, the subsurface may be generalized to occur as the sequence of soil and rock described below.

1. Unit 1, Colluvium. The site is covered by a mantle of colluvial deposits (Qyc) approximately 3 to 4.5 feet in thickness. The colluvium is a somewhat heterogeneous mix of clayey sands and sandy clays of medium dense/stiff consistency. Zones with a higher clay fraction exhibit Medium expansion potential.

2. Unit 2, Paralics. Beneath the colluvium, the site is underlain by Quaternary-aged Very Old Paralic deposits (Qvop). The unit is a well-cemented sandstone of very dense consistency, characterized by Standard Penetration Test ('SPT,' after ASTM D 1586) blow counts ('N', blows/foot) of $N \geq 50$.

   The paralics extend to well below the depths explored in the borings. Figure 4-3 (following page) provides a photograph of a representative sample of this sandstone.

4.2.3 Groundwater

Groundwater was not encountered in either of the borings by NOVA or in the test trenches reported in AET 2007. Groundwater likely first occurs at depths greater than 30 feet below ground surface.

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 site development is complete and landscape irrigation commences.

4.2.4 Surface Water

NOVA did not observe any evidence of seeps, springs, surface staining or eroded areas that would suggest the recent problems with surface water on the site.

Figure 4-3. Unit 2 Very Old Paralic Sandstone

4.3 Subsurface Conditions Following Development

4.3.1 General

Figure 4-4 and Figure 4-5 (following page) provide cross-sections across the pad, and present the position of Unit 1 colluvium and Unit 2 paralics relative to the proposed grades for the site's development.

Larger scale views of Figure 4-4 and Figure 4-5 are provided on Plate 2 following the text of this report, while the cross-section locations are presented on Plate 1.

4.3.2 Excavation Characteristics

The Unit 1 colluvium will be readily excavated by earthwork equipment usual for developments of this nature. AET 2007 reported that the Unit 2 paralics refused the 24" bucket of a Case 580D excavator on dense sandstone of Unit 2 in test trenches T-3, T-4, T-5 at depths of about 5 to 7 feet (about 3 to four feet penetration into Unit 2). Two test trenches (T-1, T-2) were excavated to 12 feet depth without refusal. This finding suggests special excavation techniques may be necessary at certain locations.

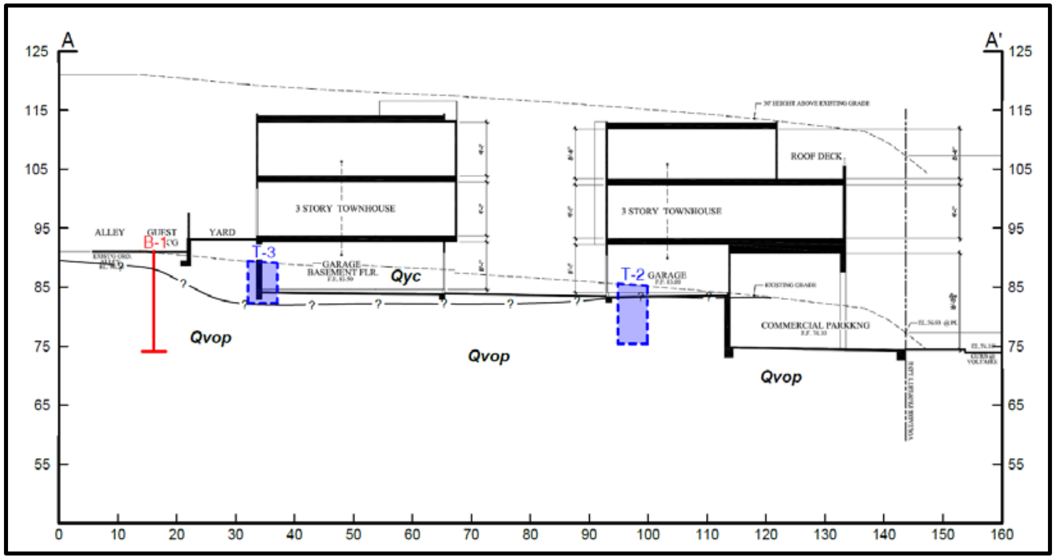


Figure 4-4. North-South Cross Section A-A'

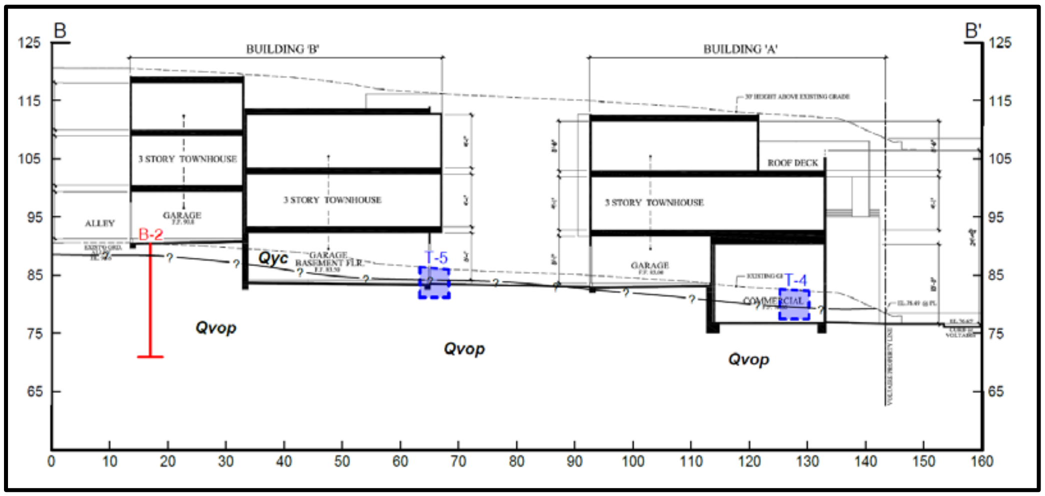


Figure 4-5. North-South Cross Section B-B'

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

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

5.2 Geologic Hazards

5.2.1 Strong Ground Motion

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion is considered significant during the design life of the proposed structure. Major known active faults in the region consist generally of *en echelon*, northwest striking, right-lateral, strike-slip faults. These include the San Andreas, Elsinore, and San Jacinto faults located east of the site; and, the Rose Canyon, San Clemente, San Diego Trough, and Agua Blanca-Coronado Bank faults located to the west of the site. San Diego's tectonic setting includes north and northwest striking fault zones, the most prominent and active of which is the Rose Canyon fault zone, located approximately 2.5 miles east of the site.

Fault segments within the Rose Canyon fault zone can generate an earthquake with a moment magnitude (MW) of up to MW = 7.2. A web-based analytical tool was used to estimate a corresponding risk-based Peak Ground Acceleration (PGA_M) of $PGA_M \sim 0.7$ g.

5.2.2 Fault Rupture and Seismic Hazard

The site is not located in a designated Alquist-Priolo earthquake fault zone, a state-zoned area that surrounds the surface trace of an active fault, considered to be areas most likely for fault rupture. The nearest earthquake fault zone is the Silver Strand section of the Rose Canyon Fault, about 2.5 miles east of the site.

Review of the City of San Diego's 2008 *Seismic Safety Study* indicates the site is located within an area defined as '…. *gently sloping to steep terrain, favorable geologic structure, low risk.* The portion of the earthquake hazard mapping within the Seismic Safety Study that includes the site is reproduced as Figure 5-1 (following page).

As may be seen by review of Figure 5-1, the site is located about 350 feet to the west of the potentially active Point Loma Fault.

In consideration of the foregoing, NOVA considers the risk of fault rupture at this site to be low.

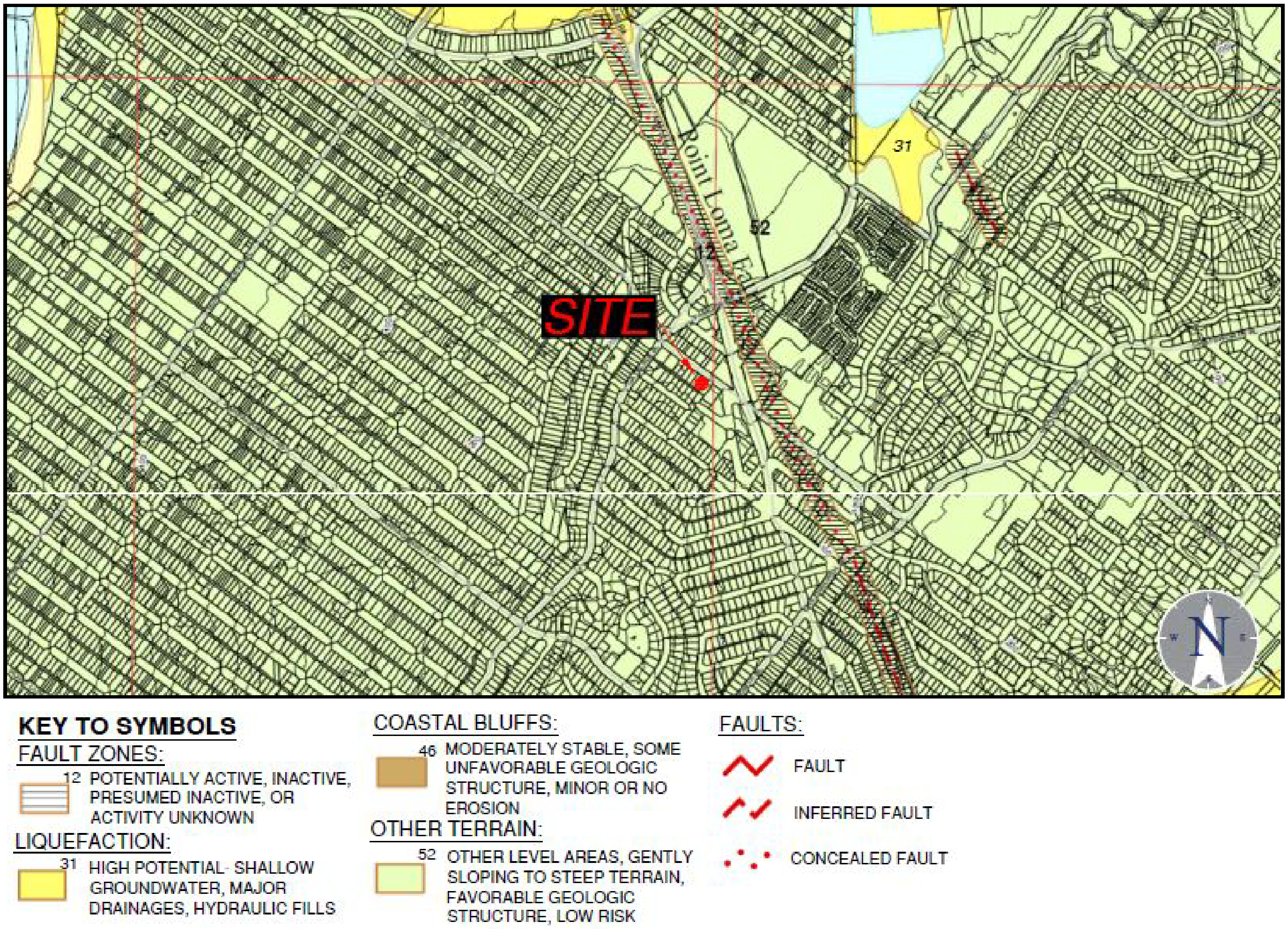


Figure 5-1. Seismic Safety Mapping of the Site Area
(source: *Seismic Safety Study*, City of San Diego, 4/3/2008)

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. These mass displacements can also include similarly larger-scale, but more narrowly confined modes of mass wasting such as rock topples, mud flows and debris flows.

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.). Rainfall is the most common trigger for landslide events. In the San Diego area, landsliding has also been precipitated by larger-scale earthwork, by destabilizing slopes by the cutting and/or filling on existing adverse geologic structure.

In assessment of this hazard, NOVA conducted a geologic reconnaissance and reviewed aerial photography for indications of landslide instability at the site. This review indicated no evidence of active or dormant landsliding.

Clues to the landslide hazard for an area can also be obtained by review of mapping that depicts both historic landslides and landslide-prone geology/topography. Figure 5-2 reproduces such mapping for the site area. The mapping indicates that the site is in an area judged ‘generally susceptible’ to landsliding, but maps no existing or questionable landslides.

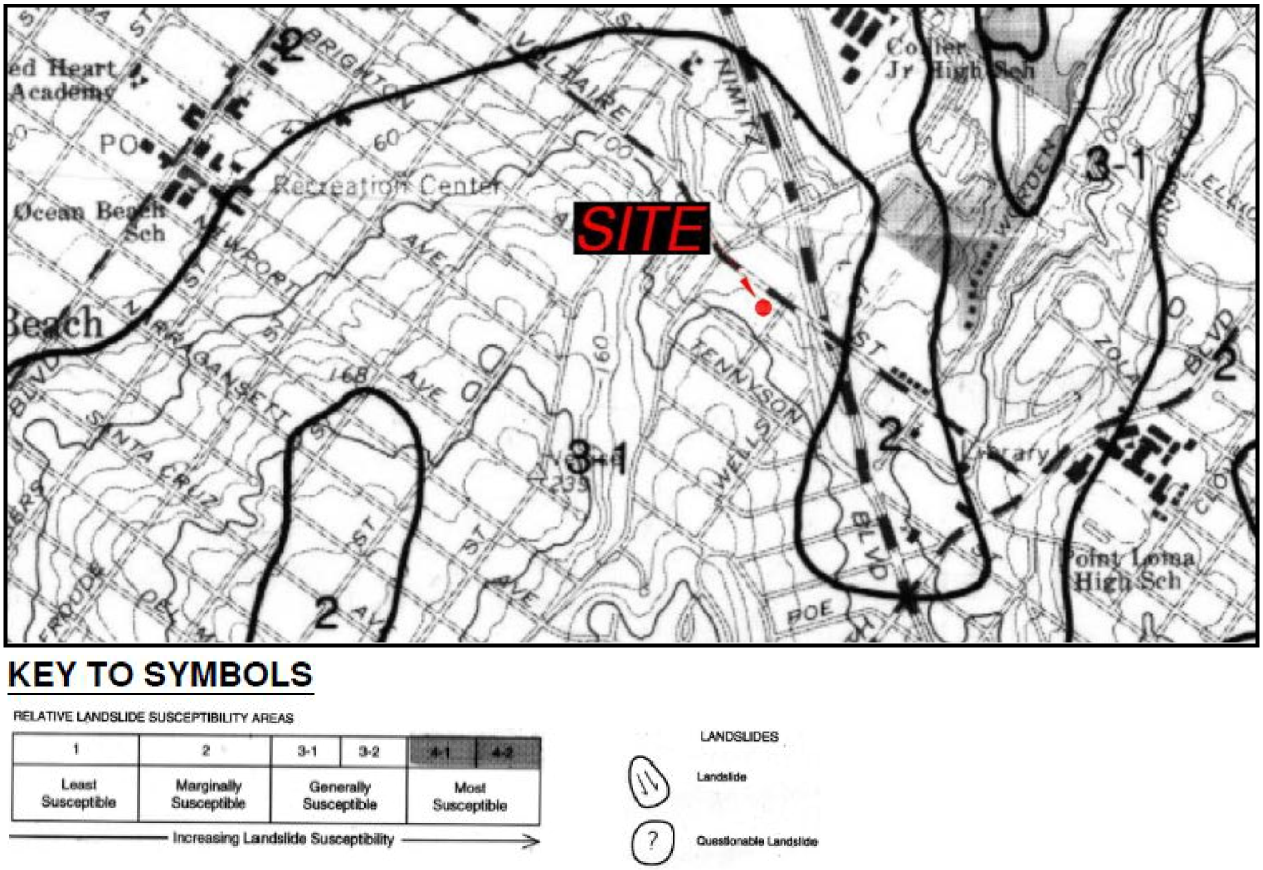


Figure 5-2. Mapping of Landslide Susceptibility in the Site Area

The above mapping is consistent with that published in the 2008 *Seismic Safety Study* by the City of San Diego and reproduced herein as Figure 5-1. The City of San Diego identifies the area of the development as including “*…gently sloping to steep terrain, favorable geologic structure, low risk.*”

In consideration of the indications of the geologic investigations, review of published mapping, and review of aerial photography, NOVA considers the landslide hazard at the site to be low 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 and there are no existing embankment slopes on the site, such that there is no concern regarding embankment stability at the residence.

5.3.2 Seismic

Liquefaction

'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. The very dense, cemented 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. Neither the Unit 1 colluvium nor the dense Unit 2 paralics will be affected by seismically induced settlement.

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 insects.

The soils have been characterized by testing to determine Expansion Index ('EI' after ASTM D 4829). EI has been adopted by the California Building Code ('CBC', Section 1803.5.3) for characterization of expansive soils. Table 5-1 summarizes 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	

The Unit 1 colluvium includes a limited thickness (less than about 2 feet) of clayey soils near its contact with the Unit 2 paralics. AET 2007 reports that this Unit 1 soil tested with 'Medium' expansion potential and meeting the criterion of CBC 2016 for expansive soil. It should be noted that medium expansive materials are not suitable for use as fill or for retaining wall backfill.

The Unit 2 paralics are characteristically sandy, with very low to low expansion potential. This Unit is suitable for use as fill and backfill.

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 soil grains of hydro-collapsible soils were initially deposited in a loose state (i.e., high initial 'void ratio') and thereafter lightly bonded by water sensitive binding agents (e.g., clay particles, low-grade cementation, etc.). While relatively strong in a dry state, the introduction of water into these soils causes the binding agents to fail. Destruction of the bonds/binding causes relatively rapid densification and volume loss (collapse) of the soil. This change is manifested at the ground surface as subsidence or settlement. Ground settlements from the wetting can be damaging to structures and civil works. Human activities that can facilitate soil collapse include irrigation, water impoundment, changes to the natural drainage, disposal of wastewater, etc.

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

5.3.5 Corrosivity

The near-surface soils were tested to show low levels of sulfates and chlorides. The potential for sulfate attack to embedded concrete is negligible. The potential for corrosion of embedded metals is relatively low; however, the soils are potentially severely corrosive to buried metals based on resistivity measurements. The indications of this testing are discussed in more detail in Section 6.

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 not located within a FEMA-designated flood zone. FIRM Panel No 06073C1880G, effective on 05/16/2012, maps the site area as an '…*area of minimal flood hazard*.' Figure 5-3 (following page) reproduces flood mapping of the site area by FEMA.

5.4.3 Tsunami

Tsunami is a term that 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. Figure 5-4 shows the site in relation to mapped estimates of tsunami inundation (red-shaded areas) in the site vicinity.

Figure 5-3. Flood Hazard Mapping of the Site Area
(source: adapted from FEMA 2012)

Figure 5-4. Tsunami Inundation Mapping of the Site Vicinity
(source: adapted from California Geological Survey2009)

5.4.4 Seiche

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. Most commonly caused by strong winds and rapid atmospheric pressure changes, seiches can be effected by seismic events and tsunamis.

The altitude and distance of the site from San Diego bay preclude this threat.

6.0 EARTHWORK AND FOUNDATIONS

6.1 General

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

Section 6.2 provides seismic design parameters. Section 6.4 addresses maintenance of the site groundform in development of new construction

6.1.2 Effect on Adjacent Properties

The proposed development is suitable for its site and 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.

6.1.3 Review and Surveillance

The subsections following provide geotechnical recommendations for the planned development as it is now understood. NOVA should 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, the Geotechnical Engineer of Record (GEOR) for this work.

6.2 Seismic Design Parameters

6.2.1 Site Class

Though the depth of soil information available for this site is limited, the deeper geology of the site area is well understood. The site and all of this area of San Diego is underlain by a variety of dense sedimentary rock to great depth, such that the site is classified as Site Class C per ASCE 7-16 (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 ASCE 7-16.

Table 6-1. Seismic Design Parameters, ASCE 7-16

Parameter	Value
Site Soil Class	C
Site Latitude (decimal degrees)	32.742760
Site Longitude (decimal degrees)	-117.234065
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.5
Mapped Short Period Spectral Acceleration, S_S	1.313 g
Mapped One-Second Period Spectral Acceleration, S_1	0.453 g
Short Period Spectral Acceleration Adjusted For Site Class, S_{MS}	1.576 g
One-Second Period Spectral Acceleration Adjusted For Site Class, S_{M1}	0.679 g
Design Short Period Spectral Acceleration, S_{DS}	1.051 g
Design One-Second Period Spectral Acceleration, S_{D1}	0.453 g

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. Water-soluble sulfates are used as an index of the potential for sulfate attack to concrete. These chemical tests were performed on a representative sample of the near-surface soils. The results of the testing to assess corrosion potential are tabulated in Table 6-2. Records of the testing are provided in Appendix C.

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

Parameter	Units	Value
pH	standard unit	6.9
Resistivity	Ω-cm	540
Water-Soluble Chloride	ppm	280
Water Soluble Sulfate	ppm	150

6.3.2 Metals

Caltrans considers a soil to be corrosive to embedded metals 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 site soils would not be considered 'corrosive' to embedded metals.

Appendix C provides records of the chemical testing that include estimates of the life expectancy of buried metal culverts of varying gauge.

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 severely 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 tested indicated water-soluble sulfate (SO_4) content of 150 parts per million (‘ppm,’ 0.015% by weight). Testing reported in AET 2007 indicates SO_4 content of 136 ppm. 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 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 or attack construction materials are traditionally completed by the Geotechnical Engineer, comparing testing results with a variety of indices regarding corrosion potential. 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

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 Select Fill

Materials

All fill should be Select Fill, a mineral soil free of organics and toxic or regulated constituents, with the characteristics listed below:

- at least 40 percent by weight finer than ¼-inch in size;
- cohesionless, classified as GW, GM, SW, SM or SC after ASTM D 2487;
- maximum particle size of 4 inches; and,
- expansion index (EI) of less than 50 (i.e., EI < 50, after ASTM D 4829).

Only the sandy portions of the Unit 1 soil will conform to the above criteria. The moderately expansive clayey portions of the Unit 1 will not conform to these criteria and should not be used as fill or backfill. Mixing of the onsite soils to create a suitable soil maybe required. The mixed soils should be tested by NOVA to verify suitability prior to use. The Unit 2 paralics can be processed to meet the criteria for Select Fill.

Placement

Compact Select Fill 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.

Fill should be placed in loose lifts no thicker than the ability of the compaction equipment to thoroughly densify the lift. For most smaller, hand-operated equipment (tampers, walked behind compactors, etc.) will be limited to on the order of 4 inches or less. Vibratory equipment should be used to densify the cohesionless Select Fill that will be used for this work.

6.4.3 Site Preparation

At the outset of site work the Contractor should establish construction Best Management Practices ('BMPs') to prevent erosion of graded/excavated areas until such time as permanent drainage and erosion control measures have been installed.

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

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.

6.4.4 Foundation Preparation

<u>Ground Supported Slab</u>

The ground supported slab at the first level of the structures may be supported on either of the conditions listed below.

- *Condition 1, Select Fill.* Constructed following removal of the Unit 1 colluvium backfilling up to finish pad grade with Select Fill that conforms with Section 6.4.2.

- *Condition 2, Unit 2 Paralics.* Constructed following removal of the Unit 1 colluvium.

<u>Grading for Buildings Supported on Shallow Foundations</u>

Where the Unit 1 colluvium is not removed from the foundation level beneath structures, the Unit 1 colluvium should be removed to contact with the level of the Unit 2 sandstones if shallow foundations are to be employed for support of the structures. This removal should extend at least five feet outside the building limits or to the property line, whichever is less. Thereafter, excavation should be backfilled with soil that conforms to the "Select Fill" criteria of Section 6.4.2. As an alternative, a controlled low strength material (CLSM, sometimes referenced as 'flowable fill') can be used.

<u>Grading for Buildings with a Cut and Fill Transitions</u>

Where building pads are underlain by a combination of fill and Unit 2 Sandstone ('cut and fill transition'), all areas of the ground supported slabs and foundations should be underlain by no less than two feet of Select Fill.

Cuts in the Unit 2 should be extended to a depth of 2 feet below the design building pad and all foundation elevations and be replaced with soil that meets the criteria for Select Fill (Section 6.4.3). Areas requiring such cuts should be completed using the steps described below.

1. *Step 1, Over-Excavate*. Over-excavate the Unit 2 Sandstone to a depth of 2 feet below the pad and footing elevation to at least 3 feet laterally outside the building limits.

2. *Step 2, Select Fill.* Fill to the base of the ground level slab with Select Fill placed and densified per Section 6.4.3, extending this fill to at least 3 feet outside the building limits.

An alternative to undercutting the cut portion of the pad is to deepen all foundations into the Unit 2 paralics.

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 30 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 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

6.4.5 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 Select Fill.

Compaction testing should be performed for every 20 cubic yards of backfill placed or each lift within 30 lineal feet of trench, whichever is less.

Backfill of utility trenches should not be placed with water standing in the trench. If granular material is used for the backfill, the material should have a gradation that will filter protect the backfill material from the adjacent soils. If this gradation is not available, a geosynthetic non-woven filter fabric should be used to reduce the potential for the migration of fines into the backfill material.

6.4.6 Flatwork

Prior to casting exterior flatwork, the upper one foot of subgrade soils should be removed and replaced with "Select" fill, moisture conditioned and recompacted, as recommended in Section 6.4.5. Concrete slabs for pedestrian traffic or landscaping should be at least four (4) inches thick.

6.5 Shallow Foundations

6.5.1 General

Structures can be supported on shallow foundations embedded in either compacted fill or the Unit 2 sandstone provided the earthwork is completed as described in Section 6.4. The following subsections provide recommendations for shallow foundations. It is recommended that all foundation elements, including any grade beams, be reinforced top and bottom. The actual reinforcement should be designed by the Structural Engineer.

6.5.1 Shallow Foundations Supported on Compacted Fill

<u>Minimum Dimensions and Reinforcing</u>

Continuous footings should be at least 24 inches wide and have a minimum embedment of 24 inches below lowest adjacent grade. Isolated square or rectangular footings should be a minimum of 30 inches wide, embedded at least 24 inches below surrounding grade.

<u>Allowable Contact Stress</u>

Continuous and isolated footings constructed as described in the preceding sections and supported on compacted fill may be designed using an allowable (net) contact stress of 2,500 pounds per square foot (psf). An allowable increase of 500 psf for each additional 12 inches in depth may be utilized, if desired.

In no case should the maximum allowable contact stress should be greater than 4,000 psf. The maximum bearing value applies to combined dead and sustained live loads (DL + LL). The allowable bearing pressure may be increased by one-third when considering transient live loads, including seismic and wind forces.

<u>Lateral Resistance</u>

Resistance to lateral loads will be provided by a combination of (i) friction between the soils and foundation interface; and, (ii) passive pressure acting against the vertical portion of the footings. Passive pressure may be calculated at 250 psf per foot of depth. A frictional coefficient of 0.35 may be used. No reduction is necessary when combining frictional and passive resistance.

<u>Settlement</u>

Structure supported on shallow foundations as recommended above will settle on the order of 0.5 inch or less, with about 50% of this settlement occurring 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., Δ/L less than 1:480).

6.5.2 Shallow Foundations Supported on Unit 2 sandstones

Isolated and Continuous Foundations

The Unit 2 sandstones will provide high-capacity foundation support for shallow foundations.

Isolated Foundations

Isolated foundations for interior columns may be designed for an allowable contact stress of 5,500 psf for dead and commonly applied live loads (DL+LL). These foundation units should have a minimum width of 30 inches, embedded a minimum of 24 inches into sound Unit 2 sandstones. This bearing value may be increased by one-third for transient loads such as wind and seismic.

Continuous Foundations

Continuous foundations may be designed for an allowable contact stress of 5,000 psf for dead and commonly applied live loads (DL+LL). These footings must be a minimum of 24 inches in width and embedded a minimum of 24 inches into the Unit 2 sandstones.

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 sandstones may be resisted by passive earth pressure against the face of the footing, calculated as a fluid density of 400 psf per foot of depth, neglecting the upper 1 foot of soil below surrounding grade in this calculation. 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 or less. This movement will occur elastically, as dead load (DL) and permanent live loads (LL) are applied.

In usual circumstance, about 50% 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., Δ/L less than 1:480).

6.6 Conventionally Reinforced Concrete Slabs

The ground level of the garage structures may employ conventional on-grade (ground-supported) slab designed using a modulus of subgrade reaction (k) of 120 pounds per cubic inch (i.e., k = 120 pci) for compacted fill and180 pci for Unit 2 Sandstones.

The actual slab thickness and reinforcement should be designed by the Structural Engineer. NOVA recommends the slab be a minimum 6 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").

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. 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 6-1 (following page).

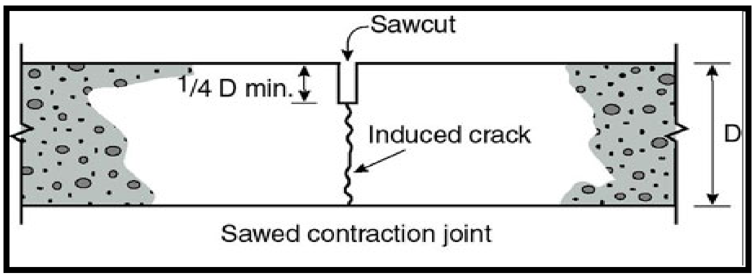


Figure 6-1. Sawed Contraction Joint

6.7 Underslab Capillary Break and Vapor Retarder

6.7.1 Design Responsibility

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 design to address this concern. This responsibility usually falls to the Architect. Decisions regarding the appropriate design 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.

6.7.2 Capillary Break

Design for a capillary break ('sand layer') should be determined in accordance with ACI Publication 302 "*Guide for Concrete Floor and Slab Construction*."

A "capillary break" may consist of a 4-inch thick layer of compacted, well-graded sand should be placed below the floor slab. This porous fill should be clean coarse sand or sound, durable gravel with not more than 5 percent coarser than the 1-inch sieve or more than 10 percent finer than the No. 4 sieve, such as AASHTO Coarse Aggregate No. 57.

6.7.3 Vapor Barrier

General

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.

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*

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 other than are specified by the ASTM E1745 class rating.

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.

6.8 Control of Moisture Around Foundations

6.8.1 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.

6.8.2 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 elimination of areas where water might pond.

6.8.3 Design

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.

NOVA does not recommend planting trees, flowers or shrubs closer than five (5) feet from foundations. Planters and other surface features which could retain water in areas adjacent to the building should be sealed. Sprinkler systems should not be installed within 5 feet of foundations or floor slabs.

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 to approved drainage facilities 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 2 percent for at least 10 feet from perimeter walls. A minimum gradient of 1 percent is recommended in hardscape areas. Drainage should be directed to approved drainage facilities.

6.9 Retaining Walls

6.9.1 General

As is discussed in Section 2, no structural plan is currently available. However, it is expected that retaining walls will be required as design adapts the new structures to the existing groundform. Section 2 (Figure 2-2) indicates retaining walls will be used to develop below-grade parking areas. The following subsections provide guidance for design of retaining walls.

6.9.2 Shallow Foundations

Retaining walls should be developed on ground prepared in accordance with the criteria provided in Section 6.4. Design criteria for continuous shallow foundations is provided in Section 6.5.

6.9.3 Lateral Earth Pressures

Table 6-6 provides recommendations for lateral soil and groundwater wall loading to below-grade walls with level backfill for varying conditions of wall yield.

Table 6-6. Lateral Earth Pressures to Below Grade Walls

Condition	**Equivalent Fluid Pressure (psf/foot) for Approved Backfill** Notes A, B	
	Level Backfill	**2:1 Backfill Sloping Upwards**
Active	35	60
At Rest	55	100
Passive	350	300

Note A: Select Fill or similar imported soil.
Note B: assumes wall includes appropriate drainage and no hydrostatic pressure.

If footings or other surcharge loads are located a short distance outside the wall, these influences should be added to the lateral stress considered in the design of the wall.

6.9.4 Seismic

The seismic load increment should be calculated as a uniform 11H psf (with H the height of the wall in feet).

6.9.5 Resistance to Lateral Loads

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

- Frictional Resistance. A coefficient of friction of 0.35 between the soil and base of the footing.

- Passive Resistance. Passive soil pressure against the face of footings or shear keys will accumulate at an equivalent fluid weight of 250 pounds per cubic foot (pcf). The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in calculations of passive resistance.

6.9.6 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.

6.9.7 Elevator Pits

The buildings may include elevators. Elevators may require pits that extend below the lowest level. Design for the elevator pit walls should consider the circumstances and conditions described below.

1. Wall Yield. NOVA expects that proper function of the elevator pit should not allow yielding of

the elevator pit walls. As such, walls should be designed to resist 'at rest' lateral soil pressures and seismic pressures provided above, also allowing for any structural surcharge.

2. Construction. Design of the elevator pit walls should include consideration for surcharge conditions that will occur during and after construction.

6.10 Temporary Excavations

6.10.1 Regulatory

Temporary slopes may be required for excavations during grading. All temporary excavations should comply with applicable 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 the Unit 1 soils may assume California Occupational Safety and Health Administration (Cal/OSHA) Soil Type C for planning purposes. The design of temporary slopes in the Unit 2 sandstones may assume Cal/OSHA Soil Type B for planning purposes.

6.10.2 Unbraced Excavations

As a matter of practice, temporary excavations 3 feet deep or less can be made vertically. Deeper temporary excavations in Unit 2 should be laid back no steeper than ¾: 1 (horizontal: vertical).

The faces of unbraced temporary slopes should be inspected daily by the Contractor's Competent Person before personnel are allowed to enter the excavation. Any zones of potential instability, sloughing or rattling should be brought to the attention of the Geotechnical Engineer-of-Record (GEOR) and corrective action implemented before personnel began working in the excavation.

Excavated soil should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. The GEOR should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during wet weather, berms are recommended along the tops of slope to prevent storm water run on from affecting the exposed slopes.

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 City of San Diego Storm Water Standards, Part 1 BMP Design Manual, October 2018 edition (hereafter, 'the BMP Manual').

Section 3.4 provides a description of the field work undertaken to complete the 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. As is described in Section 4, the site is underlain by dense sandstones of Very Old Paralics deposits (Qvop). This geologic unit is widely demonstrated in this area to have poor infiltration characteristics. The relatively low measured infiltration rate (see Section 7.2) reflects this characteristic.

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. The section provides NOVA's judgment that the site is not feasible for development of permanent stormwater infiltration BMPs.

7.2 Infiltration Rate

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 rate determined by the percolation testing.

Table 7-1. Infiltration Rate Determined by Percolation Testing

Boring	Approximate Ground Elev. (feet, msl)	Depth of Test (feet)	Approximate Test Elev. (feet, msl)	Percolation Rate (inches/hour)	Infiltration Rate (inches/hour)	Design Infiltration Rate (in/hour, F=2*)
P-1	+89	5.5	83.5	1.92	0.08	0.04

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

As may be seen by review of Table 7-1, 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 rate after applying F = 2 is I = 0.04 inches per hour. Full and partial BMPs are not required on sites with infiltration rates of less than 0.05 inches per hour.

7.3 Review of Geotechnical Feasibility Criteria

7.3.1 Overview

Section C.2.1 of Appendix C of the BMP Manual provides seven factors that should be considered by the project geotechnical professional while assessing the feasibility of infiltration related to geotechnical conditions. These factors are listed below.

- C.2.1.1 Soil and Geologic Conditions
- C.2.1.2 Settlement and Volume Change
- C.2.1.3 Slope Stability
- C.2.1.4 Utility Considerations
- C.2.1.5 Groundwater Mounding
- C.2.1.6 Retaining Walls and Foundations
- C.2.1.7 Other Factors

The above geotechnical feasibility criteria are reviewed in the following subsections.

7.3.2 Soil and Geologic Conditions

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

1. Unit 1, Colluvium. The site is covered by a mantle of 3 to 4.5 feet of clayey and sandy colluvium of medium dense consistency. Testing to determine expansion potential reported in AET 2007 shows the clayey zones of this unit to have Medium expansion potential after ASTM D 4829.

2. Unit 2, Paralics. The colluvium is underlain by dense sandstones of the Quaternary-aged Very Old Paralic deposits (Qvop). The unit is characteristically silty sandstone of very dense consistency. The locally extensive paralic deposits extend beyond the maximum depth explored by this work.

7.3.3 Settlement and Volume Change

The clayey fraction of the Unit 1 colluvium has Medium expansion potential, prone to swelling upon wetting and shrinkage upon drying. Introduction of water to this unit could create damaging foundation movement.

7.3.4 Slope Stability

Embankment stability for this site is not a constraint to BMPs.

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 likely result in damaging groundwater mounding during wet periods, affecting utilities, pavements, flat work, and foundations.

7.3.7 Retaining Walls and Foundations

The *Preliminary Site Drainage Plan* (PLSA 2019) indicates biofiltration planters will be attached to the proposed buildings on the eastern and western edges. These basins should be lined to mitigate seepage of water directly under the slab and building foundations.

Permeable pavers are also shown on the plan between buildings A and B as well as the area south of building B. Due to the proximity of the pavers to slabs, footings, and retaining walls, that the areas below the pavers be lined and drained into the storm drain system.

Though structural design is incomplete, it is expected that retaining walls will be planned for the project to adapt the development to the existing groundform and to create below-grade parking areas. Both retaining walls and shallow foundations could be affected by groundwater mounding associated with attempts to infiltrate stormwater.

7.3.8 Other Factors

The site has limited space to achieve the minimum setbacks from foundations, retaining walls, and possibly underground utilities.

7.4 Suitability of the Site for Stormwater Infiltration

It is NOVA’s judgment 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, the low design infiltration rate (I) of I = 0.04 inches per hour and related potential for groundwater mounding.

Appendix D provides completed forms related to stormwater infiltration.

8.0 PERMEABLE PAVERS

8.1 Overview

The recommendations for interlocking concrete pavers provided herein have been developed in general conformance with S*tructural Design of Interlocking Concrete Pavement for Roads and Parking Lots* Interlocking Concrete Pavement Institute (ICPI), Technical Specification No. 4, May 2011.

8.2 Planned Use of Pavers

Concrete pavers are a product that substitutes for a conventional asphalt concrete or concrete structural section. By review of the civil plans it appears that permeable pavers are proposed at several areas within the project.

8.3 Recommendations

8.3.1 General

Concrete paver units should be at least 80 millimeters (3 ⅛-inches) thick for vehicular concrete pavers. Interlocking concrete pavement can be constructed by placing the concrete paver units over a 1-inch bedding sand layer generally conforming to ASTM C-33 sand.

8.3.2 Bedding and Joint Sand Gradation

Table 8-1 summarizes bedding sand gradation recommendations and recommended joint sand gradation. The joint sand should comply with ASTM C144 with a maximum 100 percent passing the No. 16 sieves and no more than 5 percent passing the No. 200 sieve.

Bedding sand may be used as joint sand; however, additional effort may be required due to its coarser gradation.

Table 8-1. Gradation of Sand for Paver Systems

Sieve Size	Percent Passing	
	Bedding Sand	Joint Sand
3/8 – inch	100	-
No. 4	95 - 100	100
No. 8	80 - 100	95 - 100
No. 16	50 - 85	70 - 100
No. 30	25 - 60	40 - 75
No. 50	5 - 30	20 - 40
No. 100	0 - 10	10 - 25
No. 200	0 - 1	0 - 5

8.3.3 Base and Subgrade

The bedding sand should be underlain with at least 10-inches of Class II base compacted to at least 95 percent of the maximum dry density at or slightly above optimum moisture content as determined by ASTM D 1557.

The upper 12 inches of the subgrade soil should be scarified; moisture conditioned as necessary, and compacted to a dry density of at least 95 percent of the laboratory maximum dry density at or slightly above optimum moisture content as determined by ASTM D 1557.

8.3.4 Control of Infiltration

An impermeable liner (e.g., 30-mil PVC or equivalent) should be placed surrounding the pavers to prevent soil subgrade saturation and lateral water migration. The liner should extend up to the top of the aggregate base layer and adhered to the edge restraint.

Water retained by the liner can be collected by a subdrain. The lined subgrade soils should be sloped at least one percent towards the subdrain. A 4-inch diameter, Schedule 40, perforated PVC pipe encapsulated with Caltrans Class II permeable base (or equivalent) should be suitable as a subdrain. This piping should connect to solid PVC pipe to convey the stormwater to a suitable outlet structure, i.e. area drain or storm drain structure.

Figure 8-1 depicts a design to control infiltrating surface water that reflects the above recommendations.

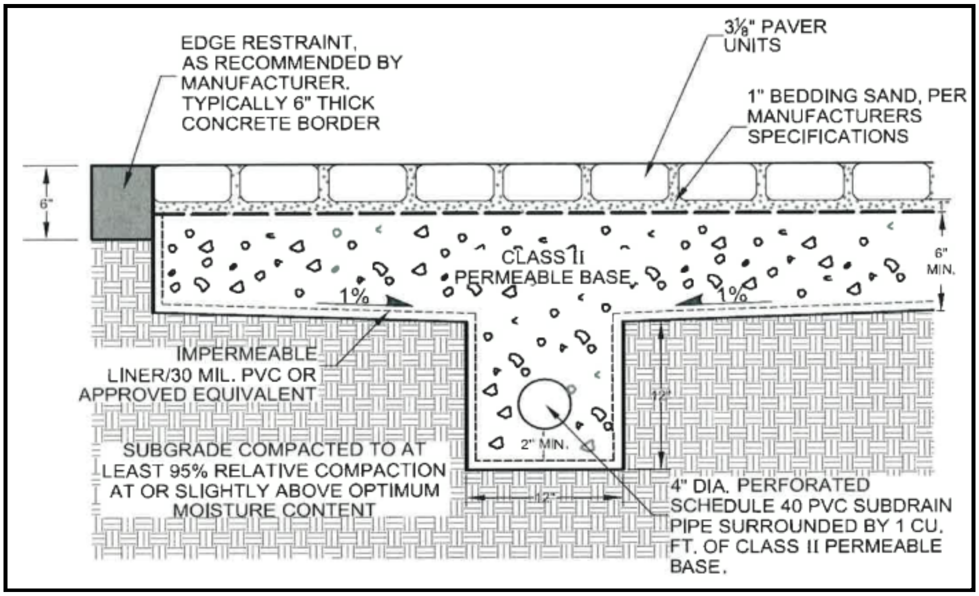


Figure 8-1. Design to Control Infiltration

8.3.5 Installation

Concrete paver installation should be performed in accordance with the manufacturer's and ICPI guidelines. Stable edge restraints such as concrete edge bands and curbs are essential to maintain horizontal interlock while the paver units are subjected to repeated vehicular loads.

8.3.6 Edge Restraint

The edge restraint may consist of a concrete pavement section. Other edge restraint recommendations can be found in the ICPI technical guidelines.

A concrete edge restraint pavement section may be designed in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 *Guide for Design and Construction of Concrete Parking Lots* using the following parameters:

Modulus of subgrade reaction, k = 100 pci
Modulus of rupture for concrete, MR= 500 psi
Traffic Category = B
Average daily truck traffic, ADTT (assumed) = 30

Based on the criteria presented above, concrete pavement should consist of a minimum of 6 inches of PCC placed over subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,200 psi (pounds per square inch).

No reinforcing steel will be necessary within the concrete for geotechnical purposes.

8.3.7 Maintenance

A maintenance schedule consisting of inspecting the pavement sections should be established. Periodic removal, replacement, and re-leveling of individual pavers may be required.

9.0 PAVEMENTS

9.1 Overview

9.1.1 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.

9.1.2 Design to Limit Infiltration

The surface grades of pavements and related design features to limit infiltration should conform with the concepts discussed in Section 7.

An important consideration with the design and construction of pavements is surface and subsurface drainage. Where standing water develops, either on the pavement surface or within the base course, softening of the subgrade and other problems related to the deterioration of the pavement can be expected.

Furthermore, good drainage should minimize the risk of the subgrade materials becoming saturated 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:

- site grading at a minimum 2% grade away from the pavements;
- compaction of any utility trenches for landscaped areas to the same criteria as the pavement subgrade;
- sealing all landscaped areas in or adjacent to pavements to minimize or prevent moisture migration to subgrade soils near pavements; 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).

9.1.3 Maintenance

Preventative maintenance should be planned and provided for. 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.

9.1.4 Review and Surveillance

The Geotechnical Engineer-of-Record should review the planning and design for pavement to confirm that the recommendations presented in this report have been incorporated into the plans prepared for the project. The preparation of subgrades for roadways should be observed on a full-time basis by a representative of the Geotechnical Engineer-of-Record.

9.2 Pavement Subgrade Preparation

Remedial grading for paved areas should consist oif removing the upper 12 inches of the Unit 1, compacting the bottom of the removals to at least 90% relative compaction after ASTM D 1557 (the 'modified Proctor'). The removed soils should be replaced with "Select" fill and densified to at least 95% relative compaction after ASTM D 1557 (the 'modified Proctor').

After the completion of compaction/densification, 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. The Geotechnical Engineer can provide alternative options such as using geogrid and/or geotextile to stabilize the subgrade at the time of construction, if necessary.

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.

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.

9.3 Flexible Pavements

The structural design of flexible pavement depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. Table 9-1 provides preliminary flexible pavement sections using an assumed R-value of 25.

Table 9-1. Preliminary Pavement Sections, R = 25

Area	Subgrade R-Value	Traffic Index	Asphalt Thickness (in)	Base Course Thickness (in)
Auto Parking	25	5	4.0	6.0
Roadways/Fire Lane/Driveways	25	6	4.0	7.5

1. The above sections assume properly prepared subgrade consisting of at least 12 inches of subgrade compacted to a minimum of 95% relative compaction after ASTM D1557, with EI <50.

2. The aggregate base materials should be placed at a minimum of 95% relative compaction after ASTM D1557.

9.4 Rigid Pavements

9.4.1 General

Concrete pavement sections should be developed in the same manner as undertaken for all other slabs and pavements: removal of the Unit 1 and replacement of that material in an engineered manner as described in Section 9.2.

Concrete pavement sections consisting of 7 inches of Portland cement concrete over a base course of 6 inches and a properly prepared subgrade support a wide range of traffic indices.

Where rigid pavements are used, the concrete should be obtained from an approved mix design with the minimum properties of Table 9-2.

Table 9-2. Recommended Concrete Requirements

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 I Portland
Concrete Aggregate	ASTM C33 and CalTrans Section 703
Aggregate Size	1-inch maximum
Maximum Water Content	0.50 lb/lb of cement
Maximum Allowable Slump	4 inches

9.4.2 Jointing and Reinforcement

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.

Load transfer devices, such as dowels or keys are recommended at joints in the paving to reduce possible offsets. 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.

10.0 REFERENCES

10.1 Site Specific

10.1.1 Design

Preliminary Site Drainage Plan, 17 on Voltaire, 4103 Voltaire Street, San Diego, California, Pasco Laret Suiter & Associates (PLSA), Job No. PLSA 3090, 07 June 2019.

17 on Voltaire, CityMark, Architectural Submittal Package, The McKinley Associates, Inc., 14 June 2019.

10.1.2 Previous Geotechnical

Soil Investigation, Proposed Mixed Use Apartment/Retail Complex Site, Southwest Corner of Voltaire Street and San Clemente Street, San Diego, Allied Earth Technology, Project No. 07-1167B7, 25 July 2007.

10.2 Design

American Concrete Institute, 2002, Building Code Requirements for Structural Concrete, ACI 318-02.

American Concrete Institute, 2015, Guide to Concrete Floor and Slab Construction, ACI 302.1R-15.

ASCE, Minimum Design Load for Buildings and Other Structures, ASCE 7-10.

ASCE, ASCE 7-10 Hazard Tool, found at: https://asce7hazardtool.online/

APWA, 2015 Standard Specifications for Public Works Construction ('Greenbook')

California Code of Regulations, 2016 California Residential Code.

California Department of Transportation (Caltrans), 2003, *Corrosion Guidelines*, Version 1.0, available at http://www.dot.ca.gov/hq/esc/ttsb/corrosion/pdf/2012-11-19-Corrosion-Guidelines.pdf.

California Division of Mines and Geology (CDMG), 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117A.

City of San Diego, 2011, *Guidelines for Geotechnical Reports*, Development Services Department.

Interlocking Concrete Pavement Institute (ICPI), S*tructural Design of Interlocking Concrete Pavement For Roads and Parking Lots*, Technical Specification No. 4, May 2011.

10.3 Site Setting

Bryant, W.A., (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, Version 2.0 California Geological Survey, found at http://www.conservation.ca.gov/cgs/information/publications/Pages/QuaternaryFaults_ver2.aspx.

Bryant, W.A. and Hart, E.W., 2007, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps, California Division of Mines and Geology Special Publication 42, Interim Revision 2007, found at: ftp://ftp.consrv.ca.gov/pub/dmg/pubs/sp/Sp42.pdf.

California Department of Conservation, California Geological Survey, 2010, *Fault Activity Map of California*, found at http://maps.conservation.ca.gov/cgs/fam/.

California Geological Survey, State of California Special Studies Zones, Chapter 7.5, Division 2 of the California Public Resources Code, Point Loma Quadrangle, official map, May 2003

California Geological Survey, State of California, County of San Diego, *Tsunami Inundation Map for Emergency Planning, Point Loma Quadrangle*, June 1, 2009.

California Department of Water Resources, Water Data Library: found at http://www.water.ca.gov/waterdatalibrary/

City of San Diego, 2008, *Seismic Safety Study, Geologic Hazards and Faults*, Grid Tile 16, 4/23/2008.

Hart, E. W. and Bryant, W. A., 1997, *Fault-Rupture Hazard Zones In California*, California Department of Conservation, Division of Mines and Geology, Special Publication 42.

Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California*, California Geological Survey, Geologic Data Map No. 6.

Kennedy, M.P., 1975, reprint 2001, *Geology of the San Diego Metropolitan Area, California*, Bulletin 200, California Dept. of Mines and Geology.

Kennedy, M.P. and Tan, 2008, *Geologic Map of the San Diego 30' x 60' Quadrangle, California*.

Norris, R. M. and Webb, R. W., 1990, Geology of California, Second Edition: John Wiley & Sons, Inc.

Rockwell, T.K., and Murbach, M., *1996 Holocene Earthquake History of the Rose Canyon Fault Zone*, U.S. Geological Survey Final Technical Report for Grant No. 1434-95-2613.

San Diego Association of Geologists. *DRAFT Provisional Fault Map for San Diego/Tijuana Earthquake Scenario*, found at http://sandiegogeologists.org/Faults_map.html)

United States Federal Emergency Management Agency (FEMA), 2012, Flood Insurance Rate Map (FIRM), Panel 06073C1880G, effective 5/16/2012.

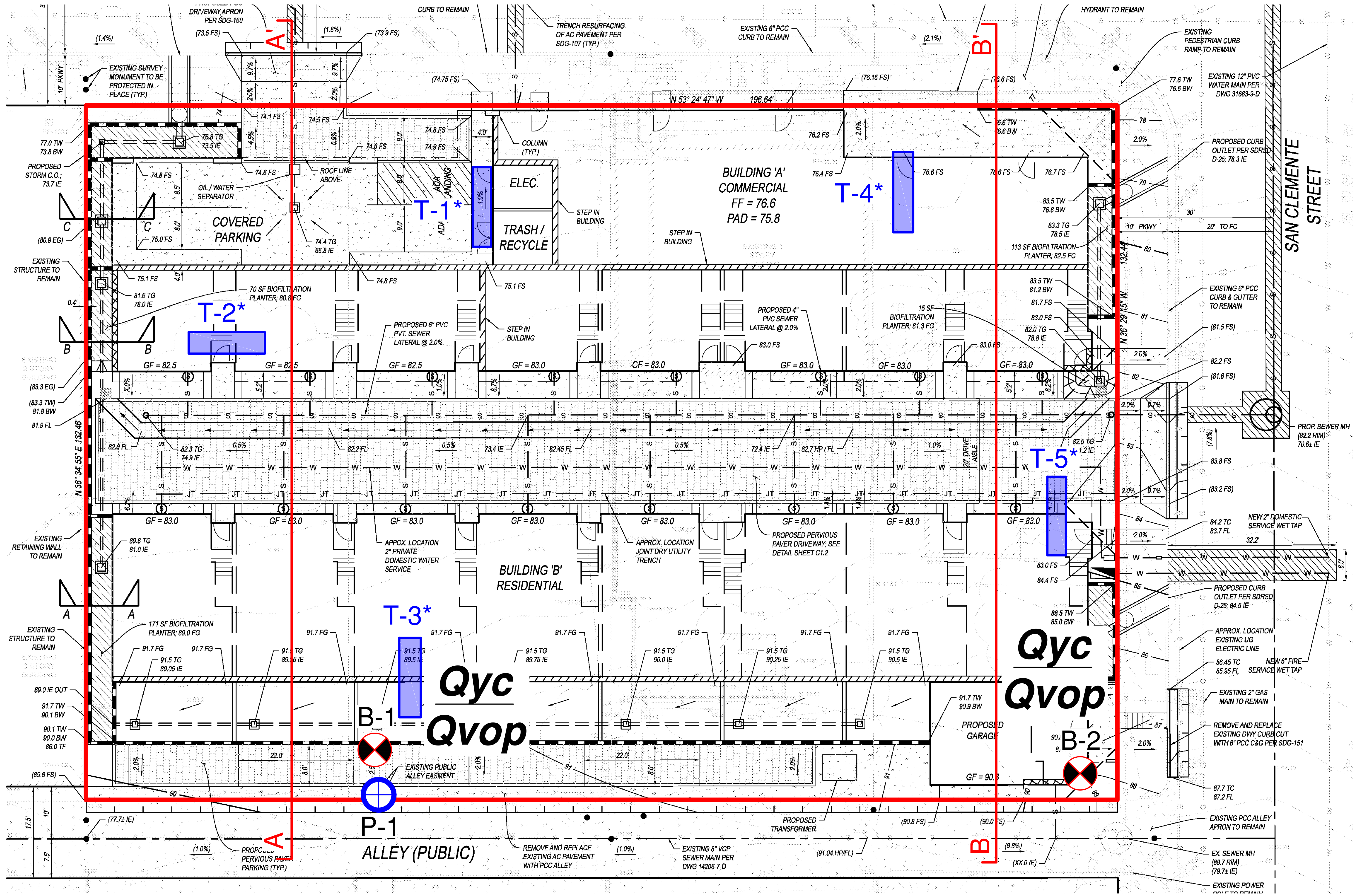
United States Geological Survey and California Geological Survey, 2011, *Quaternary Fault and Fold database for the United States*, http://earthquake.usgs.gov/regional/qfaults/.

PLATES

Plate 1: Subsurface Investigation Map

Plate 2: Geologic Cross Sections Map

NOVA



KEY TO SYMBOLS

Qyc	YOUNG COLLUVIAL DEPOSITS
Qvop	VERY OLD PARALIC DEPOSITS
B-2	LOCATION OF GEOTECHNICAL BORING
P-1	LOCATION OF PERCOLATION TEST BORING
T-5	APPROXIMATE TRENCH LOCATION (AET 2007) *ASTERISKED WHERE PROJECTED
?	GEOLOGIC CONTACT INFERRED WERE NOT ENCOUNTERED
B B'	GEOLOGIC CROSS SECTION

NOVA
4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA

858-292-7575
858-292-7570 (FAX)

WWW.USA-NOVA.COM

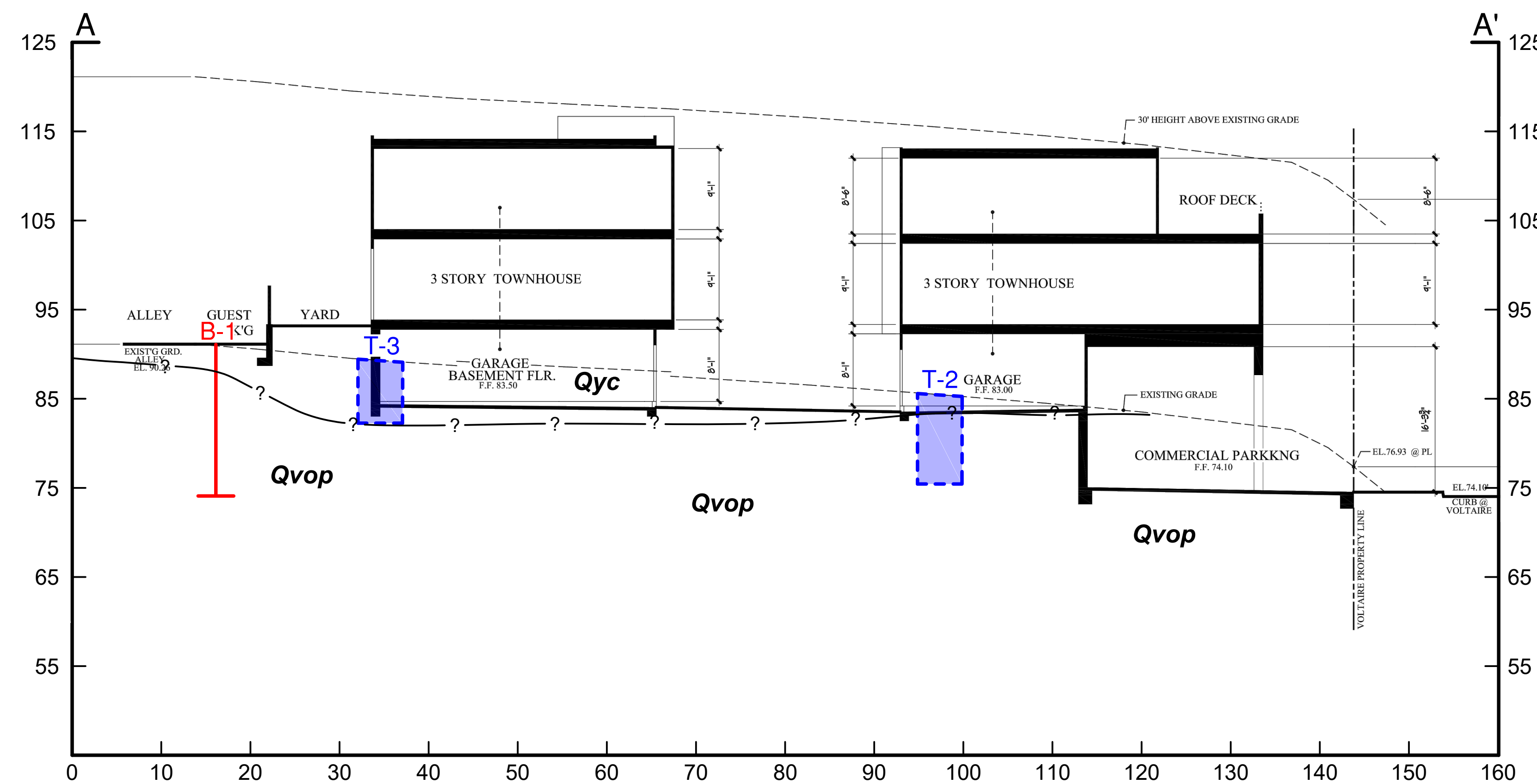
VOLTAIRE
4103 VOLATAIRE ST
SAN DIEGO, CA

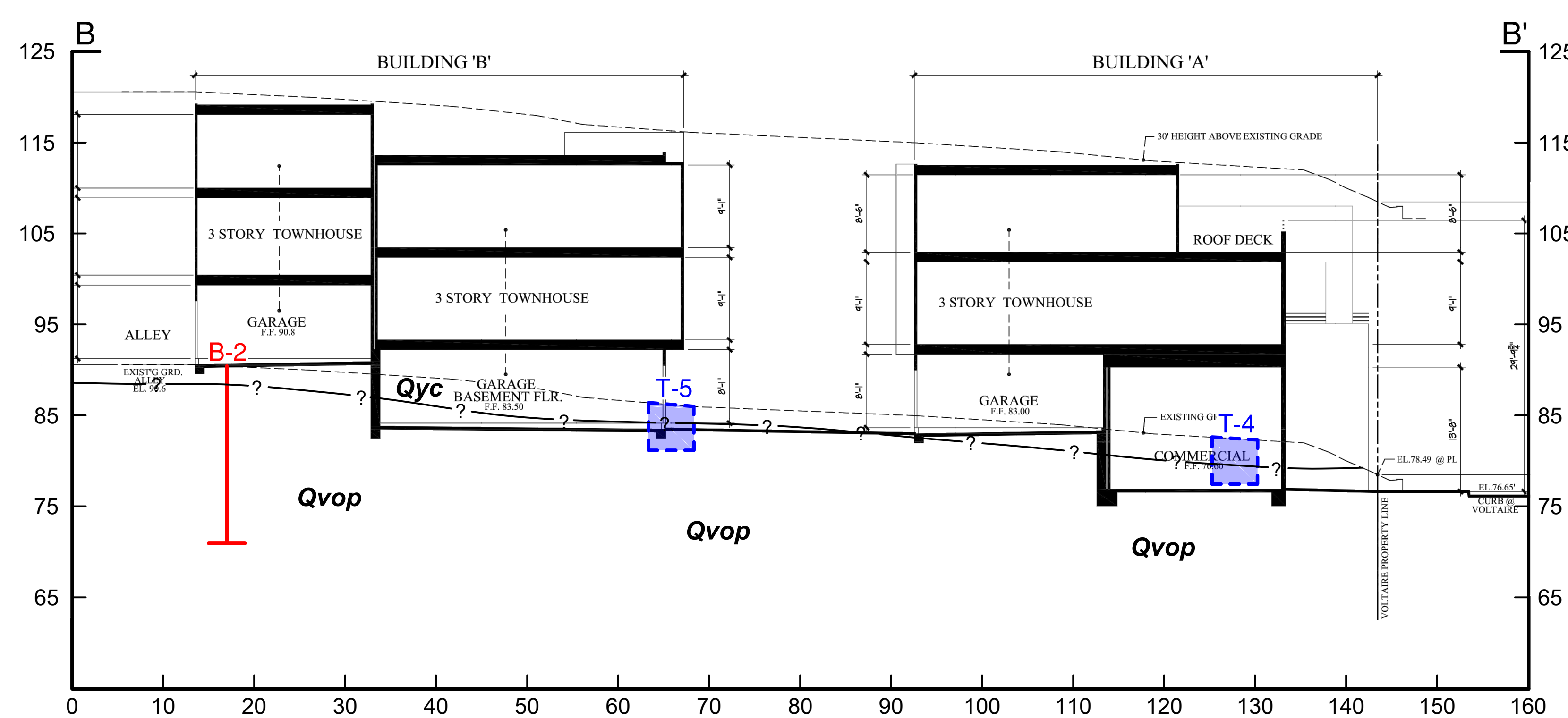
PROJECT NO:	2019147
DATE:	AUG 2019
DRAWN BY:	HM
REVIEWED BY:	WM

SUBSURFACE INVESTIGATION MAP

0 20 40

PLATE 1





KEY TO SYMBOLS

Qyc	YOUNG COLLUVIAL DEPOSITS
Qvop	VERY OLD PARALIC DEPOSITS
B-2	APPROXIMATE LOCATION OF GEOTECHNICAL BORING
T-5	APPROXIMATE TRENCH LOCATION (AET 2007)
?	GEOLOGIC CONTACT INFERRED WERE NOT ENCOUNTERED

NOVA
4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA

858-292-7575
858-292-7570 (FAX)

WWW.USA-NOVA.COM

VOLTAIRE
4103 VOLATAIRE ST
SAN DIEGO, CA

PROJECT NO:	2019147
DATE:	AUG 2019
DRAWN BY:	HM
REVIEWED BY:	WM

GEOLOGIC CROSS SECTION MAP

0 20 40

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

Copyright 2004 by ASFE, Inc. Duplication, reproduction, or copying of this document, in whole or in part, by any means whatsoever, is strictly prohibited, except with ASFE's specific written permission. Excerpting, quoting, or otherwise extracting wording from this document is permitted only with the express written permission of ASFE, and only for purposes of scholarly research or book review. Only members of ASFE may use this document as a complement to or as an element of a geotechnical engineering report. Any other firm, individual, or other entity that so uses this document without being an ASFE member could be commiting negligent or intentional (fraudulent) misrepresentation.

APPENDIX B

LOGS OF BORINGS AND TRENCHES

NOVA

BORING LOG B-1

DATE EXCAVATED:	JULY 11, 2019	**EQUIPMENT:**	IR A300
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 89 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	SM		**COLLUVIUM (Qyc):** *SILTY SAND; BROWN, DRY, MEDIUM DENSE, FINE GRAINED*		
	SC-CL	19	*CLAYEY SAND-SANDY CLAY; OLIVE GRAY MOTTLED BROWN, DAMP TO MOIST, MEDIUM DENSE-STIFF TO VERY STIFF, FINE GRAINED*	AL SA	
	SM	50/3"	**VERY OLD PARALIC DEPOSITS (Qvop):** *SILTY SANDSTONE; ORANGE BROWN, DRY TO DAMP, VERY DENSE, FINE GRAINED*		
5		50/5.5"			
			LIGHT YELLOW BROWN	SA	
		50/5.5"	*GRAY CLAYSTONE LENSE*		
10	SM-ML	50/5"	*SILTY SANDSTONE-SANDY SILTSTONE; GRAY BROWN, DAMP, VERY DENSE-HARD, FINE GRAINED*		
			SOME CLAY		
		55			
15	SM	56	*SILTY SANDSTONE; GRAY BROWN, DAMP, VERY DENSE, FINE GRAINED*		
			BORING TERMINATED AT 17 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

17 ON VOLTAIRE
VOLTAIRE STREET AND SAN CLEMENTE STREET
SAN DIEGO, CALIFORNIA

NOVA

LOGGED BY:	DEM	DATE:	AUG 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019147	APPENDIX B.1

BORING LOG B-2

DATE EXCAVATED:	JULY 11, 2019	**EQUIPMENT:**	IR A300
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 89 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 SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0						*2 INCHES OF ASPHALT CONCRETE OVER 10 INCHES OF AGGREGATE BASE*		
				SM-SC	75	**COLLUVIUM (Qyc):** *SILTY CLAYEY SAND; DARK ORANGE BROWN, DAMP, VERY DENSE, FINE TO MEDIUM GRAINED*		
				SM	50/5.5"	**VERY OLD PARALIC DEPOSITS (Qvop):** *SILTY SANDSTONE; ORANGE BROWN, DAMP, VERY DENSE, FINE GRAINED*		
5					50/3.5"	*ORANGE BROWN MOTTLED WITH LIGHT BROWN*		
							CR	
10					50/5.5"			
15					55	*LIGHT BROWN*		
					50/4"			
					61			
20						*BORING TERMINATED AT 19.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
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

17 ON VOLTAIRE
VOLTAIRE STREET AND SAN CLEMENTE STREET
SAN DIEGO, CALIFORNIA

LOGGED BY:	DEM	DATE:	AUG 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019147	

NOVA

APPENDIX B.2

PERCOLATION BORING LOG P-1

DATE EXCAVATED:	JULY 11, 2019	**EQUIPMENT:**	IR A300
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 89 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** *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0				SM		**COLLUVIUM (Qyc):** *SILTY SAND; BROWN, DRY, MEDIUM DENSE, FINE GRAINED*		
				SC-CL		*CLAYEY SAND-SANDY CLAY; OLIVE GRAY MOTTLED BROWN, DAMP TO MOIST, MEDIUM DENSE-STIFF TO VERY STIFF, FINE GRAINED*		
5				SM		**VERY OLD PARALIC DEPOSITS (Qvop):** *SILTY SANDSTONE; ORANGE BROWN, DRY TO DAMP, VERY DENSE, FINE GRAINED*		
10 15 20 25 30						*BORING TERMINATED AT 5.5 FT AND CONVERTED TO A PERCOLATION WELL.*		

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

17 ON VOLTAIRE
VOLTAIRE STREET AND SAN CLEMENTE STREET
SAN DIEGO, CALIFORNIA

LOGGED BY:	DEM	DATE:	AUG 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019147	

NOVA

APPENDIX B.3

TRENCH LOG SHEET

TRENCH NO. 1

FT.		DESCRIPTION	SOIL TYPE
0		Brown, very dry, loose	SILTY FINE SAND (SM)
1			
2	1	Very light brown	
3	2	Light brown, moist, medium dense (colluvium)	CLAYEY SAND (SC) 9.6* 109.9*
4		Light brown, moist, medium dense (Bay Point Formation)	SILTY FINE SAND (SM)
5	3		
6			
7			
8			
9		Dense	
10			
11			
12			

BOTTOM OF TRENCH (NO REFUSAL)

LEGEND

* – Indicates in-situ density test
○ – Indicates representative sample

Project No. 07-1268B7

Figure No. 3

TRENCH LOG SHEET

TRENCH NO. 2

FT.	DESCRIPTION	SOIL TYPE
0 1	Brown, very dry, loose (collivium)	SILTY FINE SAND (SM)
2 3 4 5 6 7 8 9 10	Light brown, moist, medium dense (Bay Point Formation) Dense Very dense	SILTY FINE SAND (SM)

BOTTOM OF TRENCH (NO REFUSAL)

TRENCH LOG SHEET

TRENCH NO. 3

FT.		DESCRIPTION	SOIL TYPE
0 1		Brown, very dry, loose	SILTY FINE SAND (SM)
2 3 4	1	Light brown/gray, moist dense (colluvium)	CLAYEY SAND (SC)
5 6 7		Light brown, damp, dense Very dense	SILTY FINE SAND (SM)

BOTTOM OF TRENCH (REFUSAL ON DENSE FORMATION)

TRENCH LOG SHEET

TRENCH NO. 4

FT.	DESCRIPTION	SOIL TYPE
0	Brown, dry, loose (colluvium)	SILTY SANDS (SM)
1	①	11.0*107.3*86.5%*
2	Medium dense	
3		
4	Light reddish brown, moist, dense, cemented (Bay Point Formation)	SILTY FINE SAND (SM)
5	②	

Bottom of Trench (Refusal in dense formational soil)

TRENCH LOG SHEET

TRENCH NO. 5

FT.	DESCRIPTION	SOIL TYPE
0 1 2	Light brown, dry, loose (colluvium) 1	SILTY SANDS (SM)
3 4 5	Light brown/medium gray moist, dense (Bay Point Formation) Cemented 2 Very dense	SILTY SANDS (SM)

Bottom of Trench (Refusal in dense formational soil)

APPENDIX C

RECORDS OF LABORATORY TESTING

NOVA

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.*
- **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.*
- **CORROSIVITY TEST (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 C.3 and Appendix C.4.*

Atterberg Limits (ASTM D4318)

Sample Location	Sample Depth (ft.)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI	USCS (% Finer than No. 40)
B-1	1.5 - 3	33	13	20	CL

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

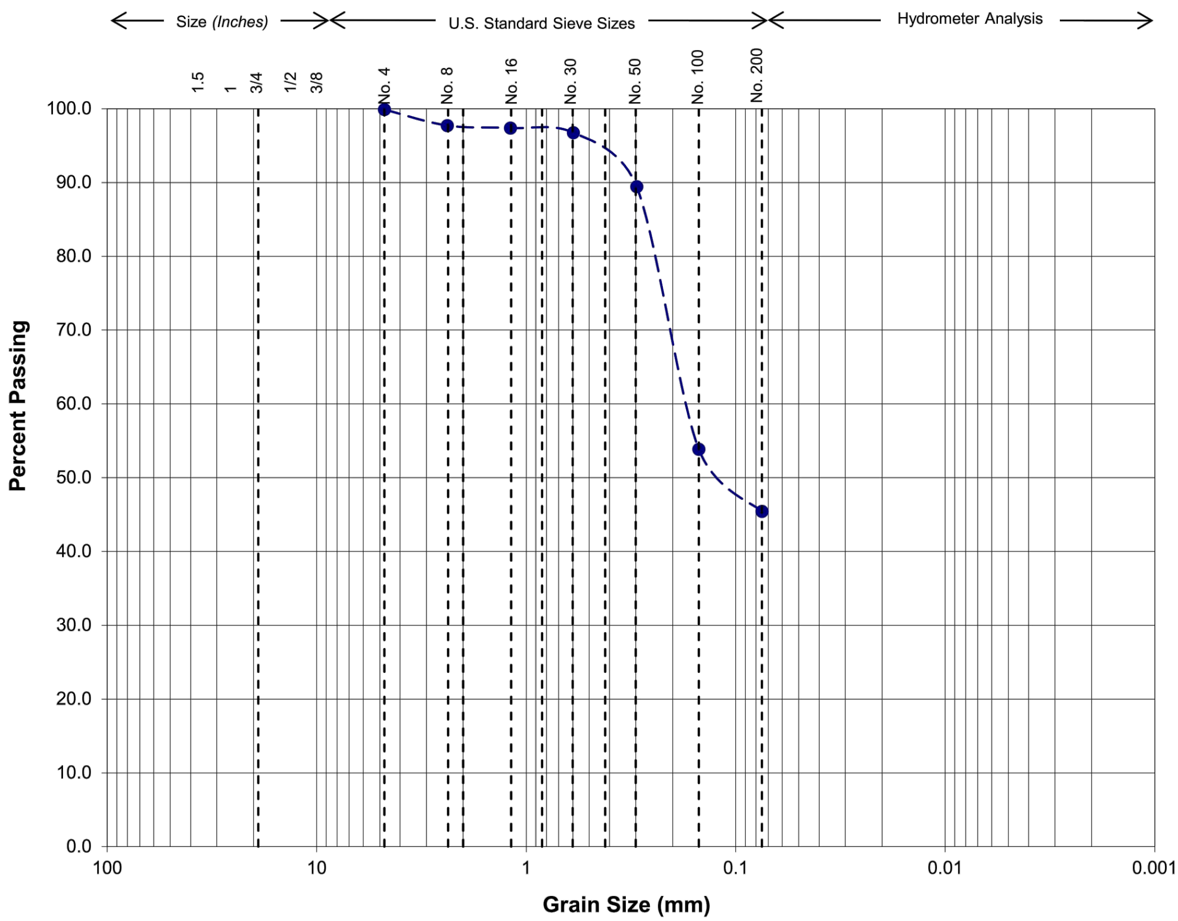
Sample Location	Sample Depth (ft.)	pH	Resistivity (Ohm-cm)	Sulfate Content (ppm)	Sulfate Content (%)	Chloride Content (ppm)	Chloride Content (%)
B-2	8 - 10	6.9	540	150	0.015	280	0.028

NOVA
4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

LAB TEST RESULTS

17 ON VOLTAIRE
VOLTAIRE STREET AND SAN CLEMENTE STREET
SAN DIEGO, CALIFORNIA

BY: HP | DATE: AUG 2019 | PROJECT: 2019147 | APPENDIX: C.2



<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): 1.5 - 3

USCS Soil Type: SC-CL

Passing No. 200 (%): 45

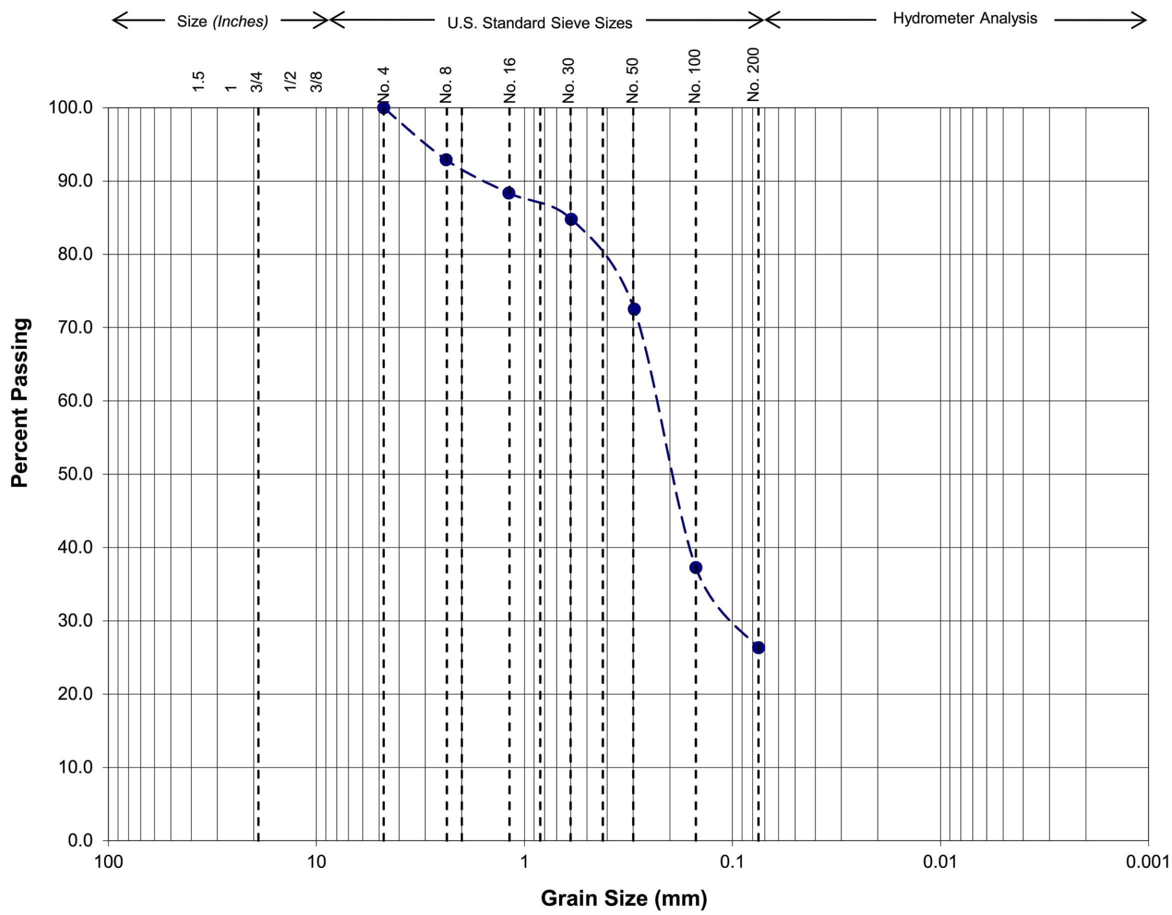
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

17 ON VOLTAIRE
VOLTAIRE STREET AND SAN CLEMENTE STREET
SAN DIEGO, CALIFORNIA

BY: HP	DATE: AUG 2019	PROJECT: 2019147	APPENDIX: C.3



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-2

Depth (ft): 5 - 7

USCS Soil Type: SM

Passing No. 200 (%): 26

NOVA

4373 VIEWRIDGE AVENUE, SUITE B

SAN DIEGO, CALIFORNIA

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

GRADATION ANALYSIS TEST RESULTS

17 ON VOLTAIRE

VOLTAIRE STREET AND SAN CLEMENTE STREET

SAN DIEGO, CALIFORNIA

BY: HP	DATE: AUG 2019	PROJECT: 2019147	APPENDIX: C.4

APPENDIX D

STORMWATER INFILTRATION

(Infiltration Feasibility Condition Letter and Worksheet C.4-1: Form I-8A)

NOVA

NOVA

GEOTECHNICAL ■ MATERIALS ■ SPECIAL INSPECTIONS

SBE ■ SLBE ■ SCOOP

4373 Viewridge Avenue, Ste. B
San Diego, CA 92123
858.292.7575

CityMark Communities, LLC
3818 Park Boulevard
San Diego, CA 92103

August 02, 2019
NOVA Project No. 2019147

Attention Mr. Rich Gustafson

Subject: Assessment of Infiltration Feasibility
Proposed 17 on Voltaire Townhomes
Voltaire Street and San Clemente Street, San Diego, California

References: See Attachment.

Dear Mr. Gustafson:

The intent of this letter is to address the infiltration conditions and related feasibility for permanent stormwater Best Management Practices ('stormwater BMPs') for drainage management areas (DMAs) at the above-referenced site.

This letter has been prepared by NOVA Services, Inc. (NOVA) for CityMark Communities, LLC. NOVA is retained by CityMark Communities as Geotechnical Engineer-of-Record (GEOR) for the project.

Background

Current Site Use

The site is comprised of a collection of four parcels with the following APNs: 449-251-05, -06, -07 and -08-00. The eastern parcels are currently occupied by a pet care business and a surfboard repair business. The western parcels are vacant, used by the neighborhood as community gardens.

Review of aerial photography dating to 1994 indicates that the eastern parcels have been developed since at least 1994. The western parcels have been vacant since 2012, when the gardens were planted.

Planned Development

NOVA's understanding of current planning for the development is based upon review of architectural documentation developed by The McKinley Associates (TMA 2019).

TMA 2019 indicates planning for a proposed residential townhouse and commercial development that will include the construction of two 3-story townhouse buildings and commercial space. The buildings will accommodate a total of 17 townhouses, ranging from 1,375 sf to 1,662 sf. Commercial space will be about 2,879 sf. The development will provide for parking for 44 vehicles in a partially below-grade basement garage. Figure 1 shows an elevation view of the development, depicting the manner by which the buildings will be adapted to the existing groundform.

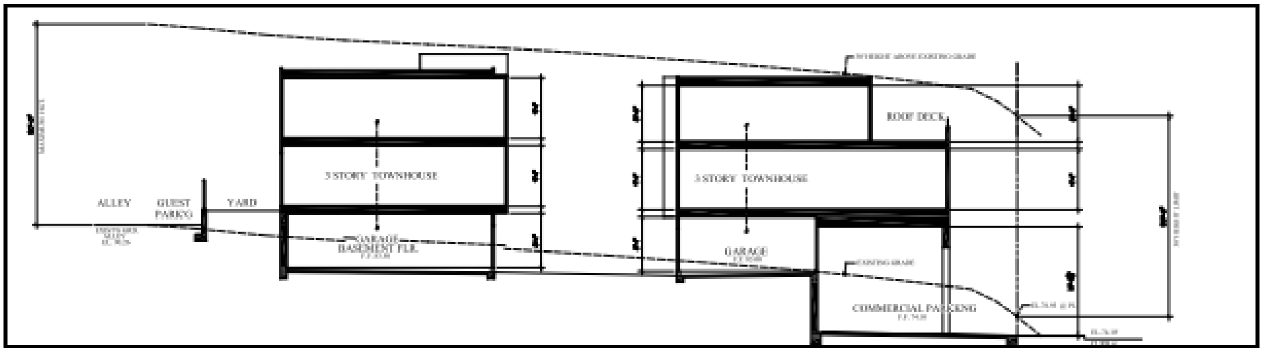


Figure 1. Elevation View of the Proposed Structure
(source: *TMA 2019)*

Proposed DMA

As the project plans are conceptual, permanent stormwater infiltration Best Management Practices (‘stormwater BMP’) locations are not identified. Figure 2 depicts the tested location.

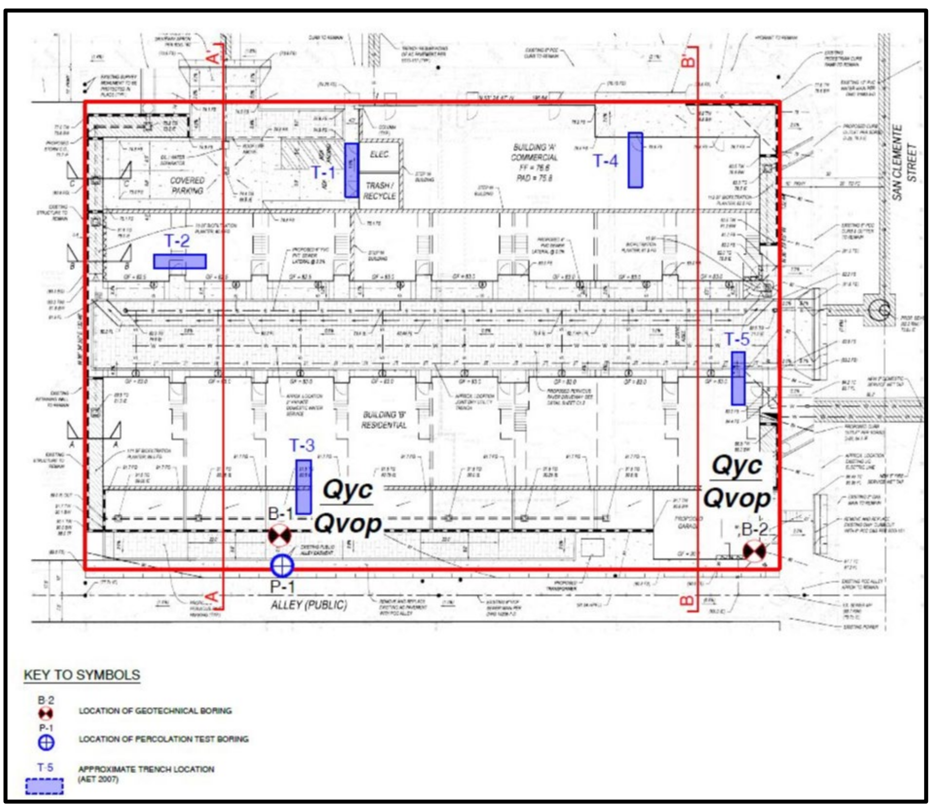


Figure 2. Percolation Test and Engineering Boring Locations
(source: adapted from *SDA 2019)*

Percolation Testing by NOVA

This site and the planned development have been the object of a prior geotechnical study by Allied Earth Technology (AET 2007). NOVA's work follows initial exploration of the site by excavation of five test trenches. Percolation testing was not completed by AET.

NOVA conducted percolation testing in the preliminary stages of planning for the site's development on July 11, 2019 and July 12, 2019. Testing was completed in accordance with procedures detailed in the referenced City of San Diego Storm Water Standards, Part 1 BMP Design Manual, October 2018 edition (San Diego 2018).

One percolation test boring ('P-1') was drilled to a depth of 5.5 feet below ground surface (bgs), into the formational soils. An exploratory engineering boring ('B-1') was drilled to 17 feet bgs near P-1. Table 1 summarizes the infiltration rate determined by the percolation testing at P-1.

Table 1. Infiltration Rate 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	+89	5.5	+83.5	0.08	0.04

Notes: (1) 'F' indicates 'Factor of Safety' (2) elevations are approximate.

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 rate after applying F = 2 is I = 0.04 inches per hour. Full and partial BMPs are not required on sites with infiltration rates of less than 0.05 inches per hour.

Review of Geotechnical Feasibility Criteria

Overview

Section C.2.1 of Appendix C of the BMP Manual provides seven factors that should be considered by the project geotechnical professional while assessing the feasibility of infiltration related to geotechnical conditions. These factors are listed below.

- C.2.1.1 Soil and Geologic Conditions
- C.2.1.2 Settlement and Volume Change
- C.2.1.3 Slope Stability
- C.2.1.4 Utility Considerations
- C.2.1.5 Groundwater Mounding

- C.2.1.6 Retaining Walls and Foundations
- C.2.1.7 Other Factors

The above geotechnical feasibility criteria are reviewed in the following subsections.

Soil and Geologic Conditions

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

1. Unit 1, Colluvium. The site is covered by a mantle of 3 to 4.5 feet of clayey and sandy colluvium of medium dense consistency. Testing to determine expansion potential reported in AET 2007 shows the clayey zones of this unit to have Medium expansion potential after ASTM D 4829.

2. Unit 2, Paralics. The colluvium is underlain by dense sandstones of the Quaternary-aged Very Old Paralic deposits (Qvop). The unit is characteristically silty sandstone of very dense consistency. The locally extensive paralic deposits extend beyond the maximum depth explored by this work.

Settlement and Volume Change

The Unit 1 colluvium has Medium expansion potential, prone to swelling upon wetting and shrinkage upon drying. Introduction of water to this unit could create damaging foundation movement.

Slope Stability

Embankment stability for this site is not a constraint to BMPs.

Utilities

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

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 likely result in damaging groundwater mounding during wet periods, affecting utilities, pavements, flat work, and foundations.

Retaining Wall and Foundations

Though structural design is incomplete, it is expected that retaining walls will be planned for the project to adapt the development to the existing groundform and to create below grade parking areas. Both retaining walls and shallow foundations could be affected by groundwater mounding associated with attempts to infiltrate stormwater.

Other Factors

The site has limited space to achieve the minimum setbacks from foundations, retaining walls, and possibly underground utilities.

Recommendation for 'No Infiltration'

It is NOVA's judgment 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, the low design infiltration rate (I) of I = 0.04 inches per hour and related potential for groundwater mounding.

Closure

NOVA appreciates the opportunity to be of continued support to CityMark and its commitment to the San Diego area. Should you have any questions regarding this letter or other matters, please contact the undersigned at (858) 292-7575.

Sincerely,
NOVA Services, Inc.

Wail Mokhtar
Project Manager

Hillary A. Price
Staff Geologist

John F. O'Brien, P.E., G.E.
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
JOHN F. O'BRIEN
651
EXPIRES 3-31-2021
GEOTECHNICAL
STATE OF CALIFORNIA

ATTACHMENT

REFERENCES

1. AET 2007. *Soil Investigation, Proposed Mixed Use Apartment/Retail Complex Site, Southwest Corner of Voltaire Street And San Clemente St., San Diego, California,* Allied Earth Technology, Project 07-116B7, July 25, 2007.

2. San Diego 2018. *The City of San Diego Storm Water Standards, Part 1 BMP Design Manual,* October 2018 Edition, The City of San Diego.

3. TMA 2019. *17 on Voltaire, CityMark, Architectural Submittal Package,* The McKinley Associates, Inc., 14 June 2019.

4. NOVA 2019. *Report, Geotechnical Investigation, Proposed 17 on Voltaire Townhomes, Voltaire Street and San Clemente Street, San Diego, California,* NOVA Services, Inc., NOVA Project No. 2019147, August 02, 2019.

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions[9]

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
Locations at P-1		Planning Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[11]? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ☐ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ☒ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ☐ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ☐ Yes; Continue to Step 1C. ☐ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ☐ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ☐ Yes; continue to Step 1E. ☐ No; select an appropriate infiltration testing method.	

[9] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[10] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[11] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[10]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ☐ Yes; continue to Step 1F. ☐ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ☐ Yes; continue to Step 1G. ☐ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ☐ Yes; answer "Yes" to Criteria 1 Result. ☐ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ☐ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ☒ No; full infiltration is not required. Skip to Part 1 Result.
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report.	

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	☐ Yes	☐ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐ Yes	☐ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐ Yes	☐ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐ Yes	☐ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐ Yes	☐ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐ Yes	☐ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐ Yes	☐ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	☐ Yes	☐ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☐ No
Summarize findings and basis; provide references to related reports or exhibits.			

Part 1 Result – Full Infiltration Geotechnical Screening [12]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	☐ Full infiltration Condition ☒ Complete Part 2

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

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions</th><th>Worksheet C.4-1: Form I-8A[10]</th></tr>
<tr><td colspan="3">Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria</td></tr>
<tr><td colspan="2">DMA(s) Being Analyzed:</td><td>Project Phase:</td></tr>
<tr><td colspan="2">Locations at P-1</td><td>Planning Phase</td></tr>
<tr><td colspan="3">Criteria 3 : Infiltration Rate Screening</td></tr>
<tr><td>3A</td><td colspan="2">NRCS Type C, D, or “urban/unclassified”: Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or “urban/unclassified” and corroborated by available site soil data?
☐ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer “Yes” to Criteria 3 Result.
☐ Yes; the site is mapped as D soils or “urban/unclassified” and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer “Yes” to Criteria 3 Result.
☒ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.</td></tr>
<tr><td>3B</td><td colspan="2">Infiltration Testing Result: Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr?
☐ Yes; the site may support partial infiltration. Answer “Yes” to Criteria 3 Result.
☒ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer “No” to Criteria 3 Result.</td></tr>
<tr><td>Criteria 3 Result</td><td colspan="2">Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP?
☐ Yes; Continue to Criteria 4.
☒ No: Skip to Part 2 Result.</td></tr>
<tr><td colspan="3">Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).</td></tr>
</table>

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions</th><th colspan="2">Worksheet C.4-1: Form I-8A[10]</th></tr>
<tr><td colspan="4">Criteria 4: Geologic/Geotechnical Screening</td></tr>
<tr><td>4A</td><td colspan="3">If all questions in Step 4A are answered "Yes," continue to Step 2B.

For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.</td></tr>
<tr><td>4A-1</td><td>Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td>4A-2</td><td>Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td>4A-3</td><td>Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td>4B</td><td colspan="3">When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1

If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result.
If there are any "No" answers continue to Step 4C.</td></tr>
<tr><td>4B-1</td><td>Hydroconsolidation. Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP.

Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td>4B-2</td><td>Expansive Soils. Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs.

Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?</td><td>☐ Yes</td><td>☐ No</td></tr>
</table>

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐ Yes	☐ No
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐ Yes	☐ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐ Yes	☐ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	☐ Yes	☐ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☐ No

Summarize findings and basis; provide references to related reports or exhibits.

For the complete infiltration feasibility evaluation see NOVA Services Inc., geotechnical study (reference, *Report, Geotechnical Investigation, Proposed 17 on Voltaire Townhouses, Voltaire Street and San Clemente Street, San Diego, CA,* NOVA Services Inc., Project No. 2019147, August 02, 2019.)

Part 2 – Partial Infiltration Geotechnical Screening Result[13]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	☐ Partial Infiltration Condition ☒ No Infiltration Condition

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