REPORT
GEOTECHNICAL INVESTIGATION

Proposed 32nd & Broadway Homes
1000 Block 32nd Street, San Diego, CA 92102

PREPARED FOR

32nd & Broadway, LLC
3184 Airway Avenue, Suite B
Costa Mesa, CA 92626

PREPARED BY

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

May 24, 2019
NOVA Project 2019066

City of San Diego Project No. 637438

GEOTECHNICAL ■ MATERIALS ■ SPECIAL INSPECTIONS

32nd & Broadway, LLC
3184 Airway Avenue, Suite B
Costa Mesa, CA 92626

May 24, 2019
NOVA Project 2019066

Attention: Ben C. Anderson

Subject: Report
Geotechnical Investigation
Proposed 32nd & Broadway Homes
1000 Block 32nd Street, San Diego, CA 92102
City of San Diego Project No. 637438

Dear Mr. Anderson:

NOVA Services, Inc. (NOVA) is pleased to forward herewith its report of the findings of a geotechnical investigation for the above-referenced project.

Work related to this report was completed by NOVA for 32nd & Broadway, LLC in accordance with the scope of work detailed in NOVA's proposal dated March 25, 2019.

NOVA appreciates the opportunity to provide its services to 32nd & Broadway, LLC. Should you have any questions, please do not hesitate to contact the undersigned at (858) 292-7575.

Sincerely,
NOVA Services, Inc.

Wail Mokhtar
Senior Project Manager

Bryan Miller-Hicks, C.E.G. 1323
Senior Geologist

PROFESSIONAL GEOLOGIST BRYAN MILLER-HICKS No. 1323 CERTIFIED ENGINEERING GEOLOGIST Exp 5/31/19 STATE OF CALIFORNIA

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Darius E. Mitchell
Project Geologist

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1000 Block 32nd Street, San Diego, CA 92102

TABLE OF CONTENTS

List of Plates

List of Appendices

List of Figures

List of Tables

1.0 INTRODUCTION

1.1 Terms of Reference

The work reported herein was completed by NOVA Services, Inc. (NOVA) for 32nd & Broadway LLC in accordance with the scope of work detailed in NOVA's proposal dated March 25, 2019, as authorized by 32nd & Broadway LLC on that date.

The report presents the findings of a geotechnical investigation for a development planned for a site located southeast of the intersection of 32nd Street and C Street in San Diego, California.

Figure 1-1 provides a graphic that depicts the site vicinity.

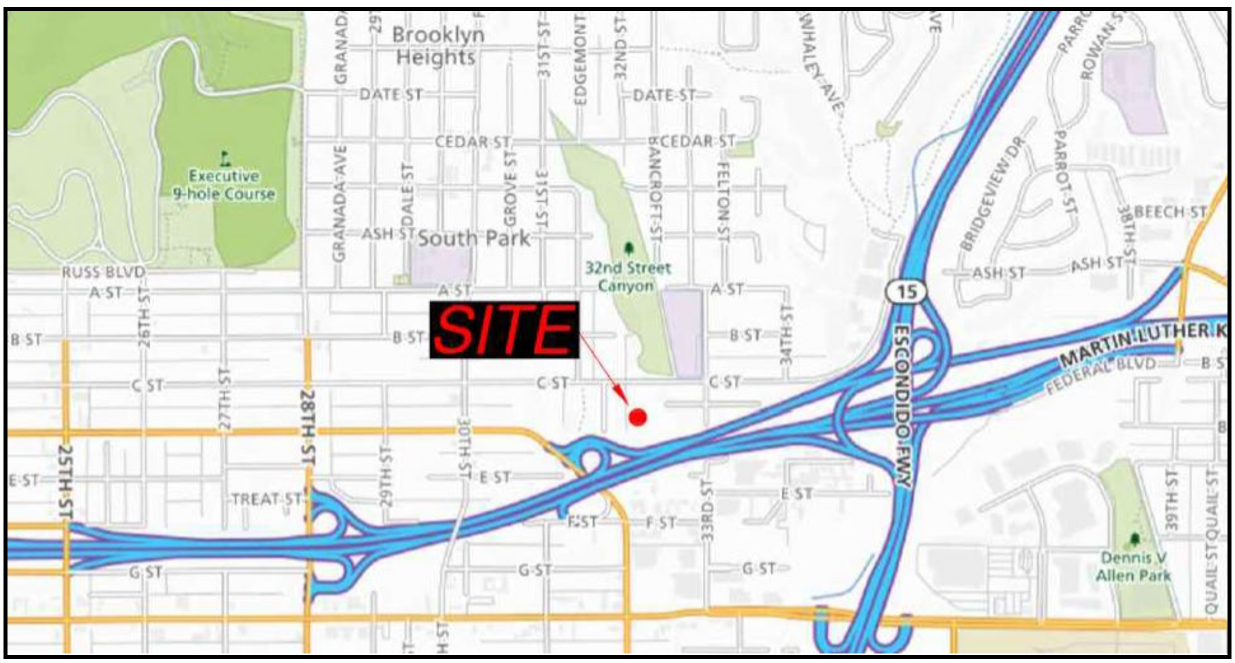


Figure 1-1. Vicinity Map

1.2 Previous Geotechnical Characterization

This site was the object of a 2006 geotechnical investigation by Construction Testing & Engineering, Inc. (reference, *Preliminary Geotechnical Investigation, proposed 28 Row Homes Northeast Corner of 32nd Street and Broadway (Proposed), San Diego, California,* Construction Testing & Engineering, Inc., Job No. 10-8520, 29 August 2006, hereinafter, 'CTE 2006').

Development of this report has utilized the indications of the subsurface exploration reported in CTE 2006 to supplement the work reported herein.

The recommendations reported herein supersede the recommendations of CTE 2006.

1.3 Objectives, Scope, and Limitations of This Work

1.3.1 Objectives

The objectives of the investigation were threefold, namely: (i) to characterize the subsurface conditions at the site; (ii) to develop recommendations for geotechnical-related design and construction; and, (iii) to develop recommendations for siting and design of permanent stormwater infiltration Best Management Practices ('BMPs').

1.3.2 Scope

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

- Task 1, Background Review. Reviewed background data, prior geotechnical reporting (particularly, CTE 2006), topographic maps, geologic reports, fault maps, and available development plans for the project.

- Task 2, Field Exploration. Completed a subsurface exploration that included the following elements of work.

    - *Subtask 2-1, Reconnaissance*. Conducted a site reconnaissance, including layout of exploratory trenches and boring. Dig Alert was notified for underground utility mark-out services.

    - *Subtask 2-2, Test Trenches*. Excavated and sampled six (6) test trenches. The trenching was completed to depths of between 3 and 10 feet below existing ground surface (bgs).

    - *Subtask 2-3, Percolation Testing.* Drilled a single percolation test boring to a depth of 3.5 bgs, following which the boring was converted to a well, and percolation testing was completed in conformance with City of San Diego standards.

    - *Subtask 2-4, Closure*. The test trenches and well were backfilled with soil cuttings, lightly compacted, and the area of work restored to the degree practical.

- Task 3, Laboratory Testing. Laboratory testing addressed soil gradation, *in-situ* moisture content, and density, expansion potential, strength and corrosivity.

- Task 4, Engineering Evaluations. Utilizing data developed by the preceding tasks, NOVA completed geotechnical and stormwater infiltration-focused engineering evaluations.

- Task 5, Reporting. Preparation of this report presenting NOVA's findings and recommendations completes the scope of work described in NOVA's March 25, 2019 proposal.

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 trenches.

NOVA can finalize 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 by the Design Team in decision-making regarding design and construction of the planned development.

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

1.5 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, considering each for its potential to affect the site.
- Section 6 provides recommendations for earthwork and foundation design.
- Section 7 provides an evaluation of the suitability of the site for development of stormwater infiltration Best Management Practices.
- Section 8 addresses design for flexible and rigid pavements.
- Section 9 provides a list of references utilized in the development of this report.

Tables and figures that amplify discussion in the text are embedded at the point at which they are referenced. Plates that present larger detail of certain graphics and discussion are provided following the text of the report.

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 test trenches. Appendix C provides records of percolation testing. Appendix D provides records of the geotechnical laboratory testing.

2.0 PROJECT INFORMATION

2.1 Site Description

2.1.1 Location

The subject site is located southeast of the intersection of 32nd Street and C Street in San Diego, California (hereinafter, 'the site'). The site is bounded to the north by a vacant lot, to the west by 32nd street, to the south by vacant land, and to the east by an existing apartment development. Figure 2-1 depicts the location and limits of the site on a recent aerial view.

Figure 2-1. Site Location and Limits
(source: adapted from Google Earth 2018)

2.1.2 Site Use

The site is currently vacant, cleared of structures and covered with light grasses and vegetation.

Review of aerial photography dating to 1994 indicates that the site has been vacant since at least that time. Figure 2-2 (following page) provides an aerial photograph depicting the site in 1994.

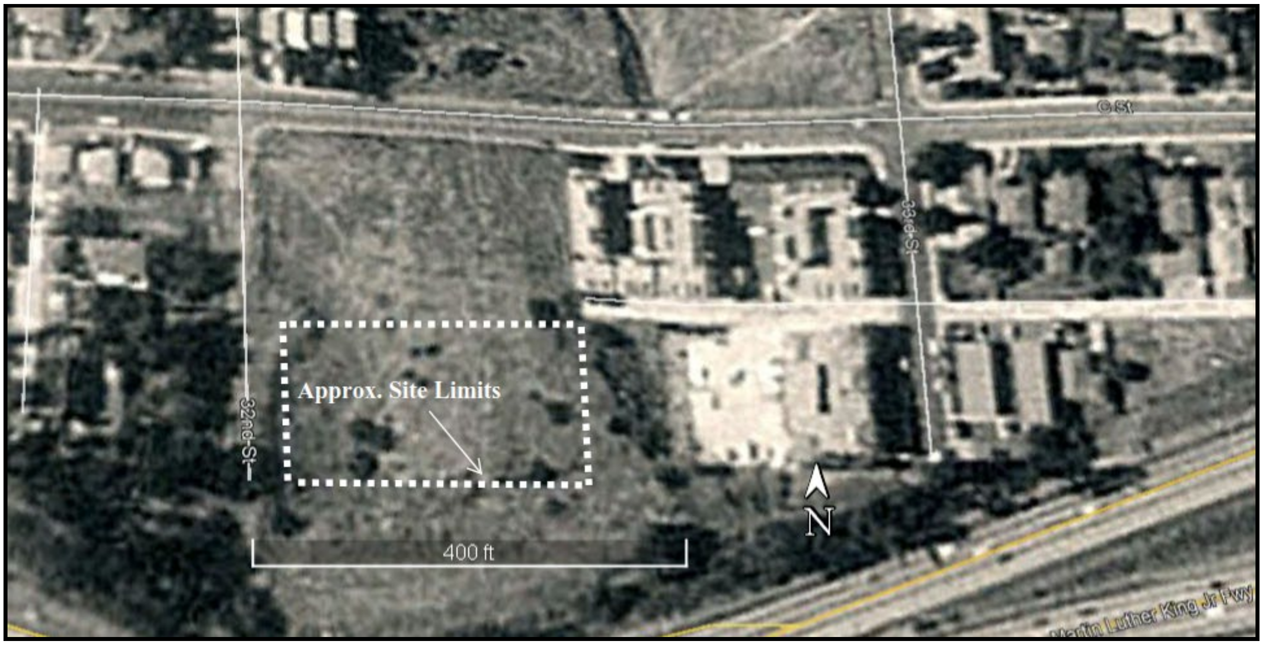


Figure 2-2. 1994 Aerial Photo
(source: adapted from Google Earth 2019)

2.2 Planned Development

2.2.1 General

NOVA's understanding of current planning for the development is based upon review of concept/ feasibility level architectural design by Woodley Architectural Group, Inc. (reference, *Concept Design Development Package, 32nd Street, BCA Development,* Woodley Architectural Group, Inc., March 6, 2019, hereinafter, 'WAG 2019').

WAG 2019 indicates that the development will consist of ± 42 zero lot line, three-level residences. Parking at each of the residences will be developed at grade, with the living space developed in two levels above the parking. Figure 2-3 reproduces an architectural graphic that depicts the planning concept.

Figure 2-3. Conceptual Development Plan
(source: WAG 2019)

2.2.2 Civil

Civil related drawings developed by Coffey Engineering, Inc depicts planning for the proposed residential development (reference, *Tentative Map, 32nd & Broadway Rowhomes, 1000 Block 32nd Street San Diego, California,* Sheet 1 of 1, Coffey Engineering, Inc., February 2019). Figure 2-4 provides a plan view of the development, indicating the layout of residences and associated infrastructure.

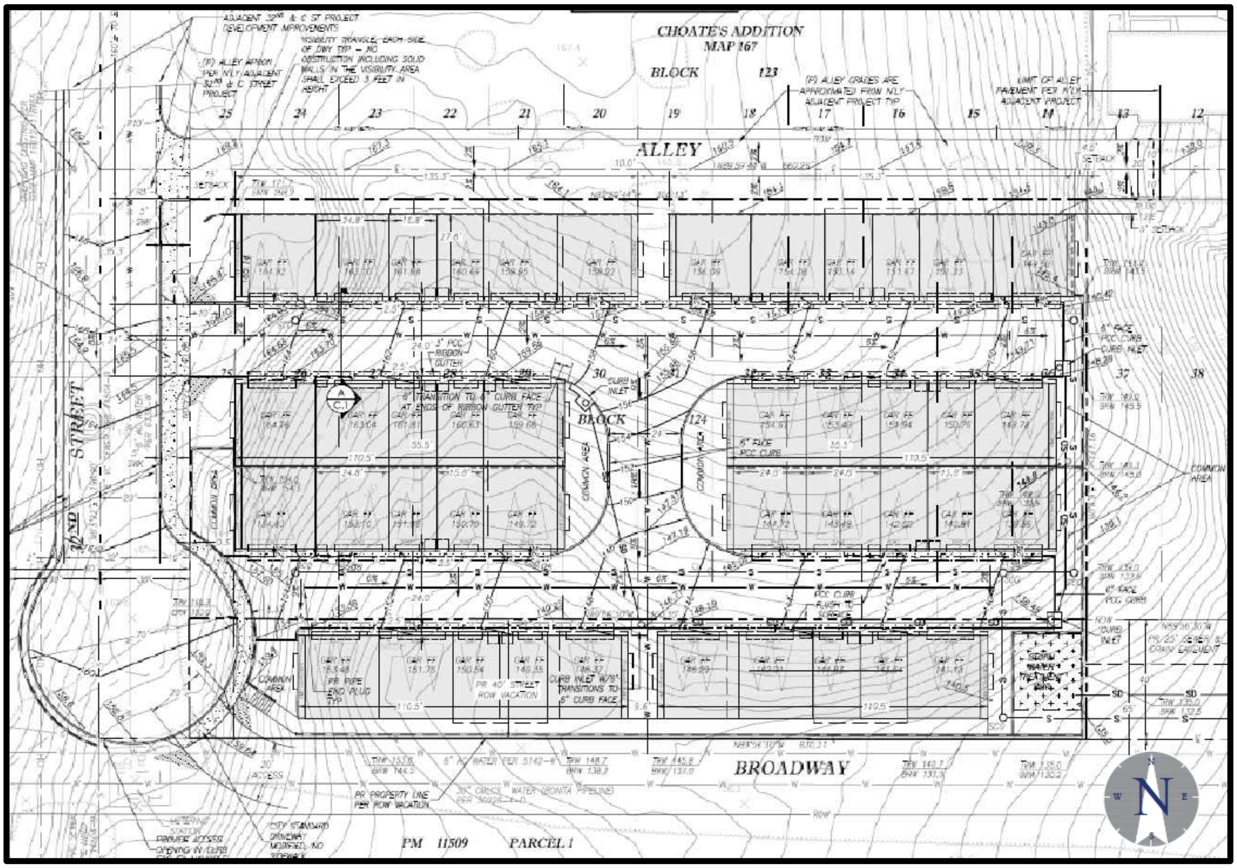


Figure 2-4. Preliminary Civil Design
(source: Coffey 2019)

2.2.3 Structural

Though structural design has not been completed, NOVA expects that the buildings will be wood-framed. The buildings will likely be founded on conventionally reinforced, ground-supported slab foundations.

NOVA expects the average bearing stress across ground supported foundations of similar structures will be in the range of 200 to 300 pounds per square foot (psf). Maximum wall loads will be on the order of 500 pounds per linear foot. No below grade construction is planned.

2.2.4 Potential for Earthwork

It is anticipated that grading will be directed toward 'balanced' cutting and filling- cutting areas of higher elevation and placing this soil as fill in lower portions. The resultant filling will involve placement of engineered fill to about 12 feet thickness. NOVA's understanding of the planned earthwork at the site is depicted in Figure 2-5 on the following page.

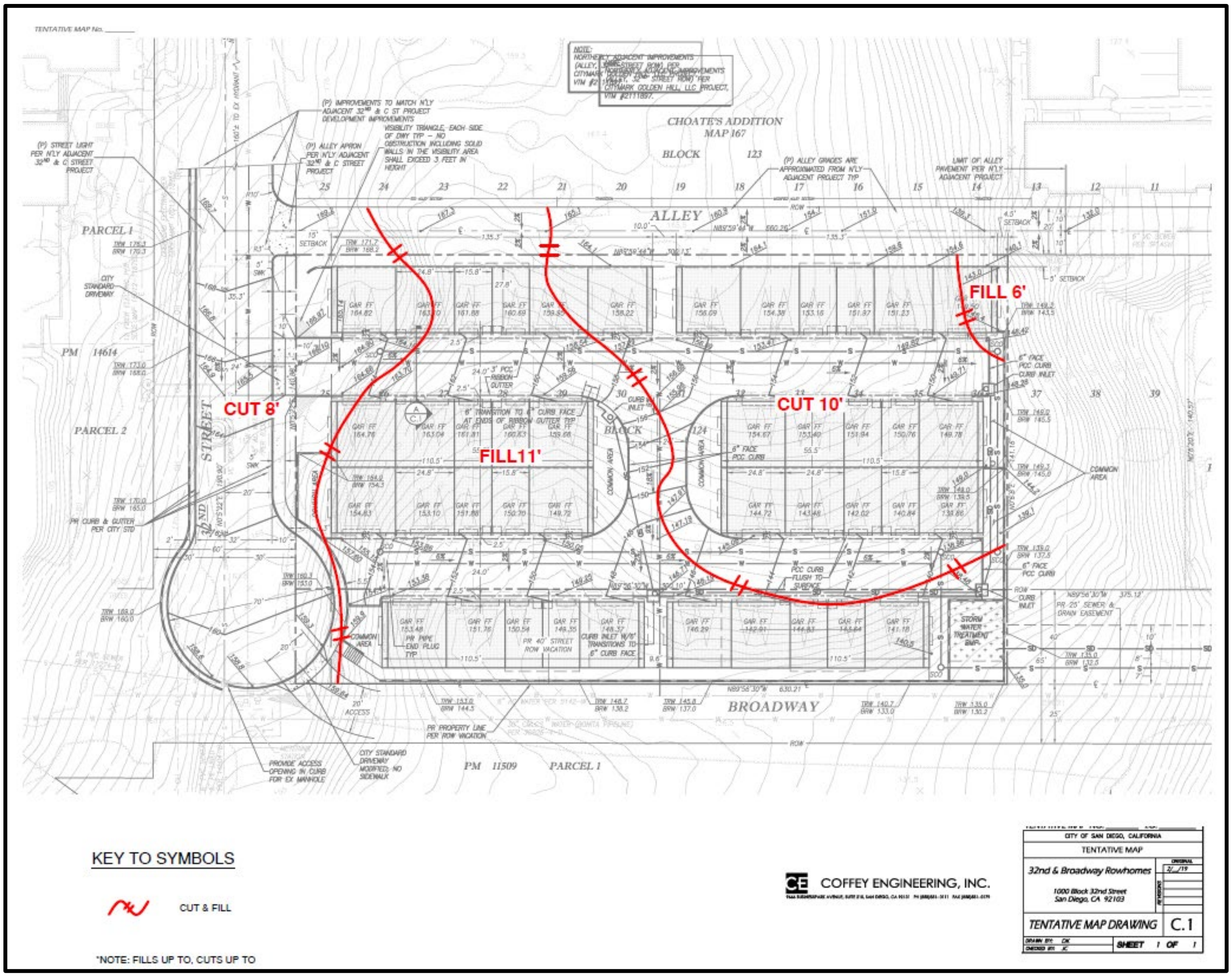


Figure 2-5. Anticipated Earthwork at The Site
(source: Coffey Engineering 2019)

3.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

3.1 Overview

NOVA's subsurface exploration was completed on April 1-2, 2019. The work included six (6) exploratory test trenches (referenced herein as 'T-1' through 'T-6') and a single percolation test well (referenced as 'P-1'). The test trenches and percolation well were completed by specialty subcontractors retained by NOVA. All work was completed under the continuous supervision of a NOVA geologist.

Figure 3-1 presents a plan view of the site indicating the location of the test trenches and percolation test well.

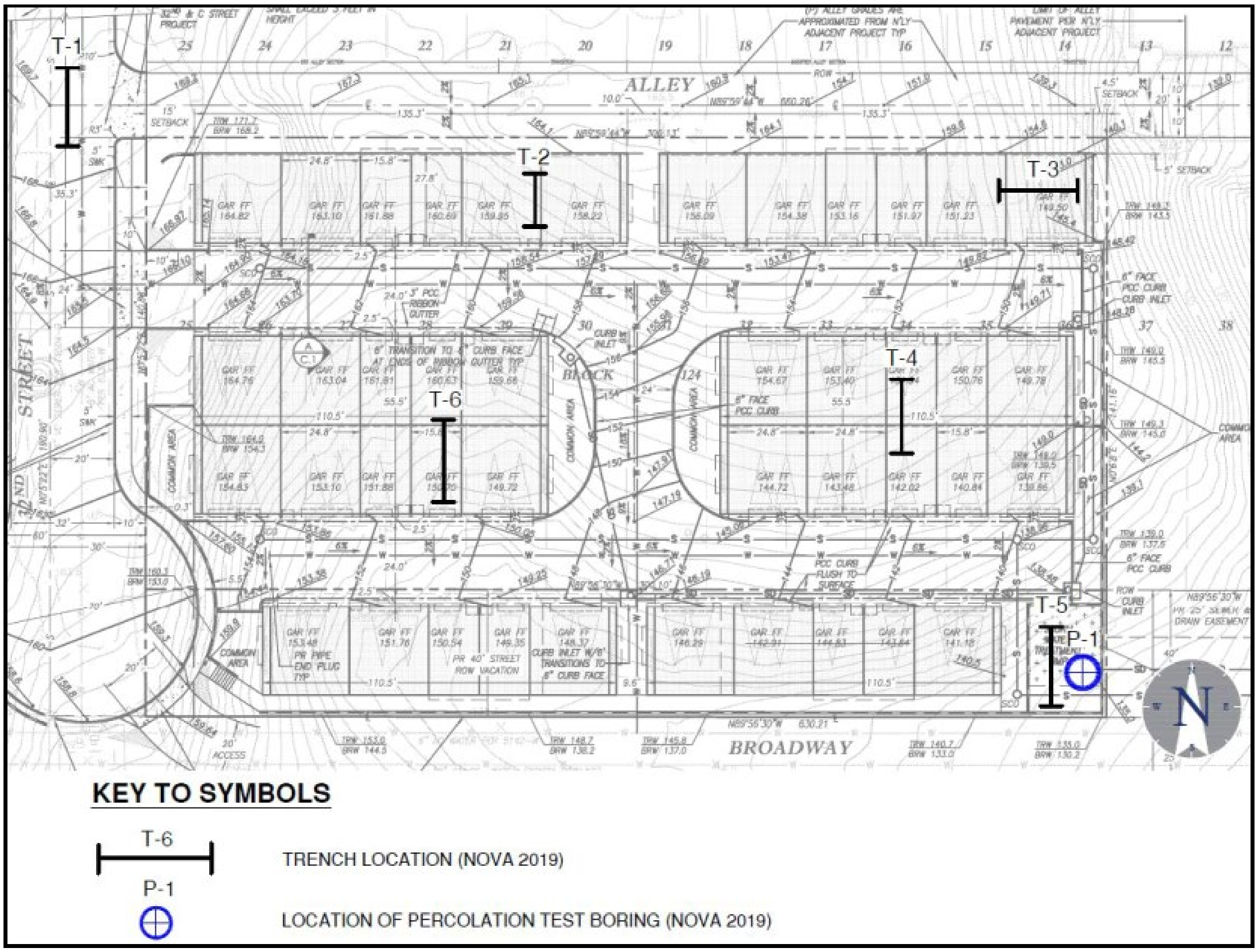


Figure 3-1. Location of the Test Trenches and Percolation Testing by NOVA

Soil samples recovered from the test trenches 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.

3.2 Test Trenches

3.2.1 General

The test trenches were excavated by a rubber-tired backhoe. Trench locations were determined in the field by the NOVA geologist. Elevations of the ground surface at the trench locations were estimated.

Table 3-1 provides an abstract of the test trenches.

Table 3-1. Abstract of the Test Trenches

Boring Reference	Approximate Ground Surface Elevation (feet, msl)	Total Depth Below Ground Surface (feet)	Approximate Depth to Formation (feet)
T-1	+173	10	5.5
T-2	+160	3	1
T-3	+150	3.5	2
T-4	+150	3	1.5
T-5	+132	5	2
T-6	+147	10	4

Figure 3-2 (following page) provides a photograph depicting backhoe operations

3.2.2 In Situ Testing

The soils in the test trenches were tested by two means, as described below.

- *In situ* testing by use of the dynamic cone penetrometer ('DCP', after ASTM D6951) testing was used to determine the consistency of the subsurface soils.

- Bulk samples were returned to NOVA's materials laboratory. Laboratory testing was undertaken to determine index and strength characteristics of the subsurface soils.

The DCP is widely used to assess the quality of subgrades. As utilized for this assessment, a 60º cone was driven at the end of connecting rods. The upper rod includes a slide assembly that allows a 17.6 pound (8 kg) drop hammer to slide 22.6 inches, striking an anvil to drive the cone in the ground. The test is initiated by inserting the cone into the ground, 'seating' it until the widest part of the cone is just below the testing surface. Thereafter, incremental cone penetration is recorded as hammer blows are applied. The depth of penetration for a set number of blows is measured.

Data from the DCP was processed to produce a 'DCP Index' expressed in inches per blow (then converted to millimeters per blow). The DCP Index (DCPI) can be correlated to California Bearing Ratio (CBR) utilizing guidance provided on Table 2 of ASTM D6951.

Logs of the trenches are provided in Appendix B.

3.2.3 Closure

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

Figure 3-2. Backhoe Operations, Trench T-4 (4/1/19)

3.3 Percolation Testing

3.3.1 General

NOVA directed the excavation and construction of one (1) percolation test boring 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.

Figure 3-3 (following page) depicts percolation test well P-1.

3.3.2 Drilling

The boring was drilled with an 8-inch hand auger to a depth of 3.5 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.

Figure 3-3. Percolation Test Well P-1

3.3.3 Conversion to a 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.

3.3.4 Percolation Testing

The percolation test well was pre-soaked by filling the well with water to at least 5 times the well's radius. The pre-soak water did not percolate at least 6 inches into the soil unit within 25 minutes; therefore, the well 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 (minimum of 12 readings), or until the water percolation stabilized after each reading. The water level was raised to close to the previous water level to maintain a near constant head before subsequent readings.

Table 3-2 (following page) abstracts the indications of the percolation testing.

Table 3-2. Abstract of the Percolation Testing

Boring	Approx. Elevation (feet, msl)	Total Depth (feet)	Approximate Percolation Test Elev. (feet, msl)	Percolation Rate (in/hour) [2]	Subsurface Units Tested[1]
P-1	+134	3.5	+130.5	18.72	Qvop8

Note: The referenced geologic unit is Very Old Paralic Deposits (Qvop8).

3.3.5 Closure

At the conclusion of the percolation testing, the PVC pipe was removed and the resulting hole backfilled with soil cuttings to match the existing surfacing.

3.4 Laboratory Testing

3.4.1 General

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

3.4.2 Soil Gradation

The visual classifications were further evaluated by gradation testing. Table 3-3 provides an abstract of this testing.

Table 3-3. Abstract of the Soil Gradation Testing

Sample Ref		Percent by Weight Passing the U.S. #200 Sieve	Classification after ASTM D2488
Trench	Depth (feet)		
1	0 - 3	19	SM
4	2 - 3	22	SM
5	2 - 3	8	SP-SM
6	2 - 3	19	SM

Note:
1. 'Passing #200' percent by weight passing the U.S. # 200 sieve (0.074 mm), after ASTM D6913.
2. All testing on bulk samples recovered over the identified depth interval.

3.4.3 Moisture-Density

A single sample of the near-surface soil was tested to determine its moisture-density relationship after ASTM D 1557 (the 'modified Proctor'). This testing indicated an optimum dry density (γ_{dry}) of γ_{dry} = 132 lb/ft^3 at an optimum moisture content (w) of w = 8.6%.

3.4.4 R-Value

The Resistance Value (R-value) test is a material stiffness test, demonstrating a material's resistance to deformation as a function of the ratio of transmitted lateral pressure to applied vertical pressure. The purpose of this test is to determine the suitability of prospective subgrade soils and road aggregates for use in the pavement sections of roadways. The test is used by Caltrans for pavement design, replacing the California Bearing Ratio (CBR) test. A saturated cylindrical soil sample is placed in a Hveem Stabilometer device and then compressed. The stabilometer measures the horizontal pressure that is produced while the specimen is under compression.

A sample representative of soils from the upper soil horizon was selected for this testing. Testing after ASTM D 2844 indicated an R-value of 28.

3.4.5 Expansion Potential

An Expansion Index ('EI', after ASTM D4829) test was performed to evaluate the potential for expansion of the fill that overlies the site. The expansion test was performed on a remolded sample.

EI has been adopted by the California Building Code ('CBC', Section 1803.5.3) for characterization of expansive soils. The listing below tabulates the qualitative descriptors of expansion potential as included with ASTM D 4829 and the CBC.

Tests of three remolded samples of the fill indicated these soils have "Low' to 'Very Low' expansion potential after ASTM D 4829.

3.4.6 Corrosivity Testing

Resistivity, sulfate content and chloride contents were determined to estimate the potential corrosivity of onsite soils- the potential that the on-site soils may corrode/chemically attack embedded metals and concrete. These chemical tests were performed on a representative sample of the near-surface soils by Clarkson Laboratory and Supply, Inc.

Chemical testing of the near-surface soils indicates the soils may be corrosive. Embedded concrete will not be at risk for sulfate attack.

Section 6 addresses this consideration in more detail. Records of this testing are provided in Appendix D.

4.0 SITE CONDITIONS

4.1 Geologic Setting

4.1.1 Regional

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

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

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

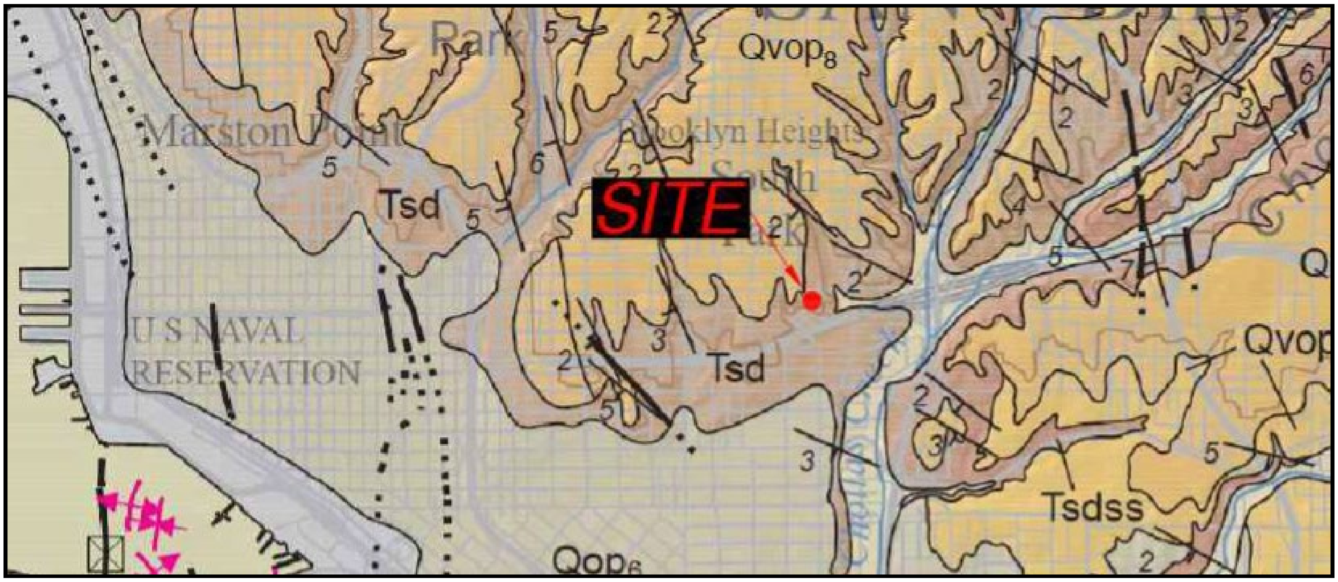
4.1.2 Site Specific

The geology of the Coastal Plain zone of the Peninsular Ranges geomorphic province is controlled by both alluvial and marine influences. This plain is underlain by near-shore marine sedimentary rocks deposited at various intervals from the late-Mesozoic through Quaternary ages. The Coastal Plain increases in elevation from west to east across marine terrace surfaces uplifted during Pleistocene time. Sedimentary rocks consist of sandstones, siltstones, and claystones that were deposited during the Cretaceous, Tertiary, and Quaternary periods.

Geologic units encountered by the subsurface investigation include a veneer of artificial fill (Qaf), 1 to 5.5 feet in thickness as encountered in the trenches. Underlying the fill are sandstones of the Very Old Paralic Deposits Formation (Qvop8) and the San Diego Formation (Tsd), encountered only in trench T-6 at 9 feet bgs. Figure 4-1 (following page) depicts the geology of the site area.

The Very Old Paralic deposits are interfingered strandline, beach, estuarine and colluvial deposits of middle to early Pleistocene age. Sedimentary rocks of this Formation are primarily siltstone, sandstone and conglomerate.

The San Diego Formation is largely comprised of poorly indurated fossiliferous marine sandstones, and marine and nonmarine pebble and cobble conglomerates of early Pleistocene and late Pliocene age.



KEY TO SYMBOLS

Tsd	SAN DIEGO FORMATION, UNDIVIDED	Tsdss	SAN DIEGO FORMATION, MARINE SANDSTONE	Qop_6	OLD PARALIC DEPOSITS, UNIT 6
$Qvop_8$	VERY OLD PARALIC DEPOSITS, UNIT 8	Qya	YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS	Qvop	VERY OLD PARALIC DEPOSITS

Figure 4-1. Geologic Mapping of the Site Vicinity

4.2 Site Conditions

4.2.1 Surface

The site is undeveloped, covered with light grasses and sparse trees. The site descends in elevation generally from northwest to southeast. The ground surface at the northwest of the site is graded to about El +175 feet msl. The ground surface at the southeast edge of the site is about El +135 feet msl. This 40-foot differential occurs over a distance of about 300 feet, a surface gradient of about 13%.

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, Fill. The site is covered by a mantle of fill approximately 1 to 5 feet in thickness. The fill is comprised of silty to clayey sands of loose to medium dense consistency and sandy clays of firm consistency. Because no records exist regarding the placement of this fill, the fill is considered 'undocumented,' and at risk for wide variations in quality.

2. Unit 2, Very Old Paralics. Beneath the fill, the site is underlain by Quaternary-aged Very Old Paralic deposits (Qvop8). Locally, this unit outcrops at the ground surface. As encountered in the

trenches, the unit is characteristically well cemented silty sandstone of dense to very dense consistency. The upper surface (about the upper 1’-2’) of this unit includes abundant cobbles to typically 4”.

The unit is very dense, characterized by high resistance to the Dynamic Cone Penetrometer. The backhoe met refusal on very dense Paralics in trenches T-3, T-4 and T-5. The unit will be incompressible under loads from future earth fills and structures.

The Paralics extend to below the depths explored in trenches T-1 through T-5. Figure 4-2 depicts the Paralics at T-2, depicting the upper ‘cobbly’ zone and dense sandstones below that level.

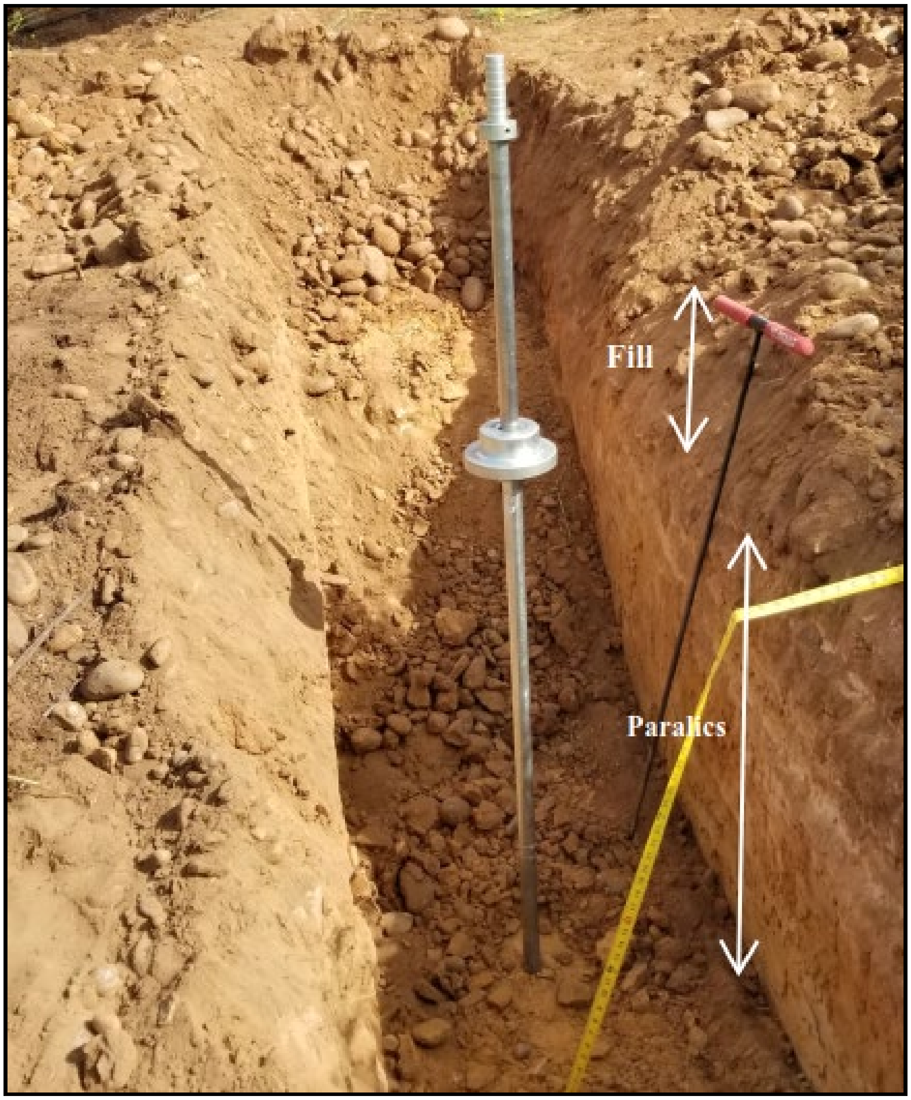


Figure 4-2. DCP Testing and Unit 2 Paralics Exposures at Trench T-2

3. Unit 3, San Diego Formation. Trench T-6 disclosed that the site is underlain by the Pleistocene/Pliocene-aged San Diego Formation (Tsd) below the Very Old Paralics. Trenches T-1 through T-5 did not extend through the Very Old Paralics to expose this unit. As encountered at the site, San Diego Formation consists of medium dense and friable well-graded sandstone, similar in quality (low compressibility, high strength) to the Very Old Paralics. Figure 4-3 depicts soil recovered from this unit.

Figure 4-3. Unit 3 San Diego Formation

4.2.3 Groundwater

No groundwater was encountered in the borings above the maximum depth explored. As such, groundwater is expected to first occur below a depth of about 30 feet.

Infiltrating storm water from prolonged wet periods can ‘perch’ atop localized zones of lower permeability soil that exist above the static groundwater level. No perched groundwater was observed during excavation of the test trenches.

4.2.4 Surface Water

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

5.0 REVIEW OF GEOLOGIC, SOIL AND SITING HAZARDS

5.1 Overview

This section provides a review of geologic, soil and siting-related hazards common to this region of California, considering each for its potential to affect the planned development.

The primary hazard identified by this review is that the site is at risk for moderate-to-severe ground shaking in response to large-magnitude earthquakes during the lifetime of the planned development. While strong ground motion could affect the site, there is no risk of liquefaction or related seismic phenomena. The expectation of strong ground motion is common to all civil works in this area of California.

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 not located within a currently designated Alquist-Priolo Earthquake Zone. No known active faults are mapped on the site area. The nearest known active faults are within the Rose Canyon fault system. The closest faults within this system lie in the downtown graben, located approximately 1.5 miles west of the site. This system has the potential to be a source of strong ground motion. The potentially active (pre-Holocene) Texas Street Fault is about 0.4 miles west of the site. There is no evidence of movement on this fault within the last 11,700 years.

The seismicity of the site was evaluated utilizing a web-based analytical tool provided by the USGS. This evaluation shows the site may be subjected to a Magnitude 7 seismic event, with a corresponding risk-based Peak Ground Acceleration (PGA_M) of $PGA_M \sim 0.49$ g.

5.2.2 Fault Rupture

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

Figure 5-1 (following page) reproduces published mapping of faulting in the site vicinity.

5.2.3 Geologic and Seismic Hazards

Seismic risk was further reviewed by review of mapping developed by the City of San Diego in 2008. The *Seismic Safety Study* published by the City indicates that the site is located in an area of favorable geologic setting.

Figure 5-2 (following page) reproduces mapping from the *Seismic Safety Study* that includes the site vicinity.

Figure 5-1. Faulting in the Site Vicinity

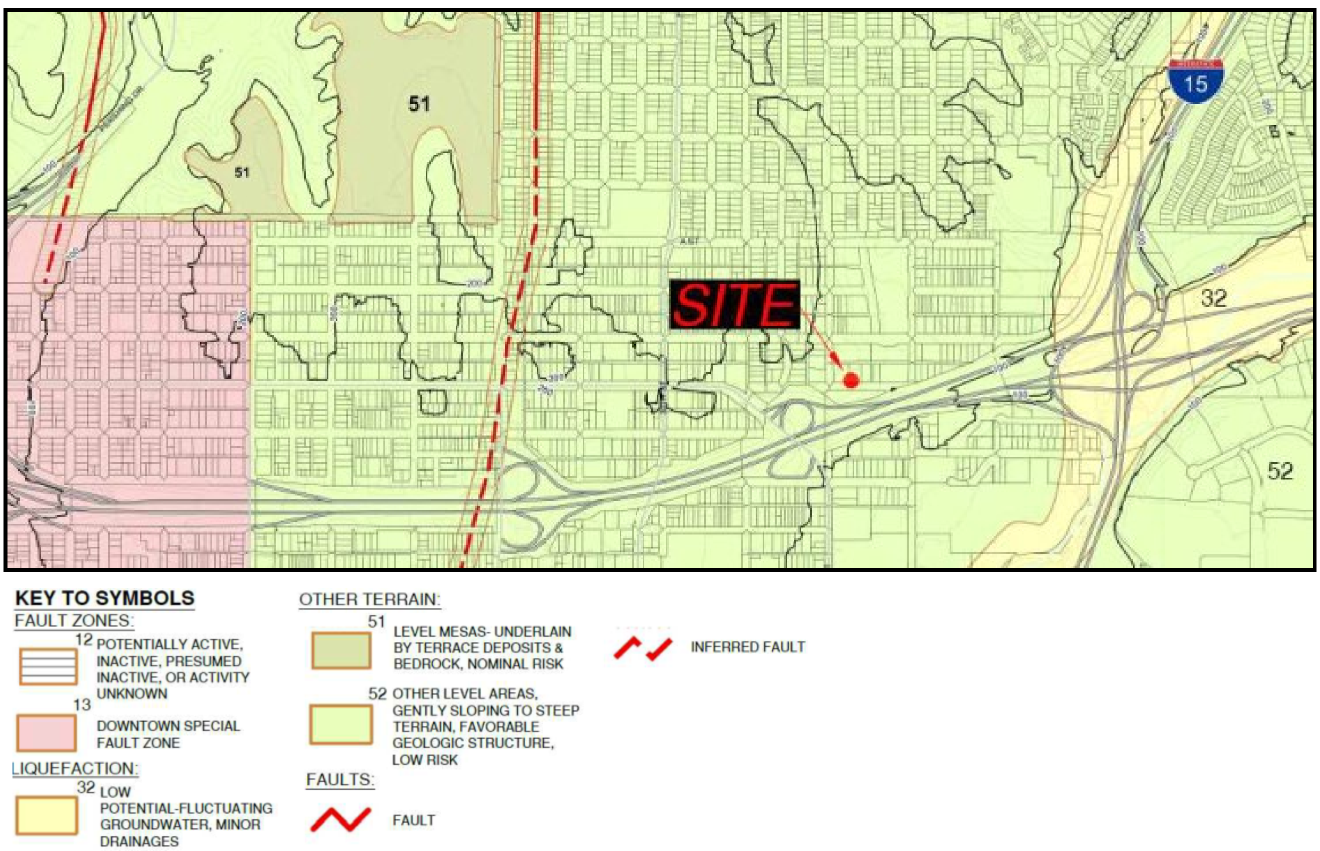


Figure 5-2. Geologic and Seismic Hazard Mapping of the Site Vicinity
(source: adapted from *Seismic Safety Study, Geologic Hazards and Faults,*
City of San Diego, Grid Tile 17, April 2008)

5.2.4 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/failure surfaces. These mass displacements can also include 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.). World-wide and within California, the most common initiator of landslides is wet weather/precipitation.

Though the site is set in an area where the ground surface slopes on the order of 10% to 20%, the formational geologic units that underlie the area are not associated with landslide susceptibility. Figure 5-3 reproduces landslide susceptibility mapping for the site area.

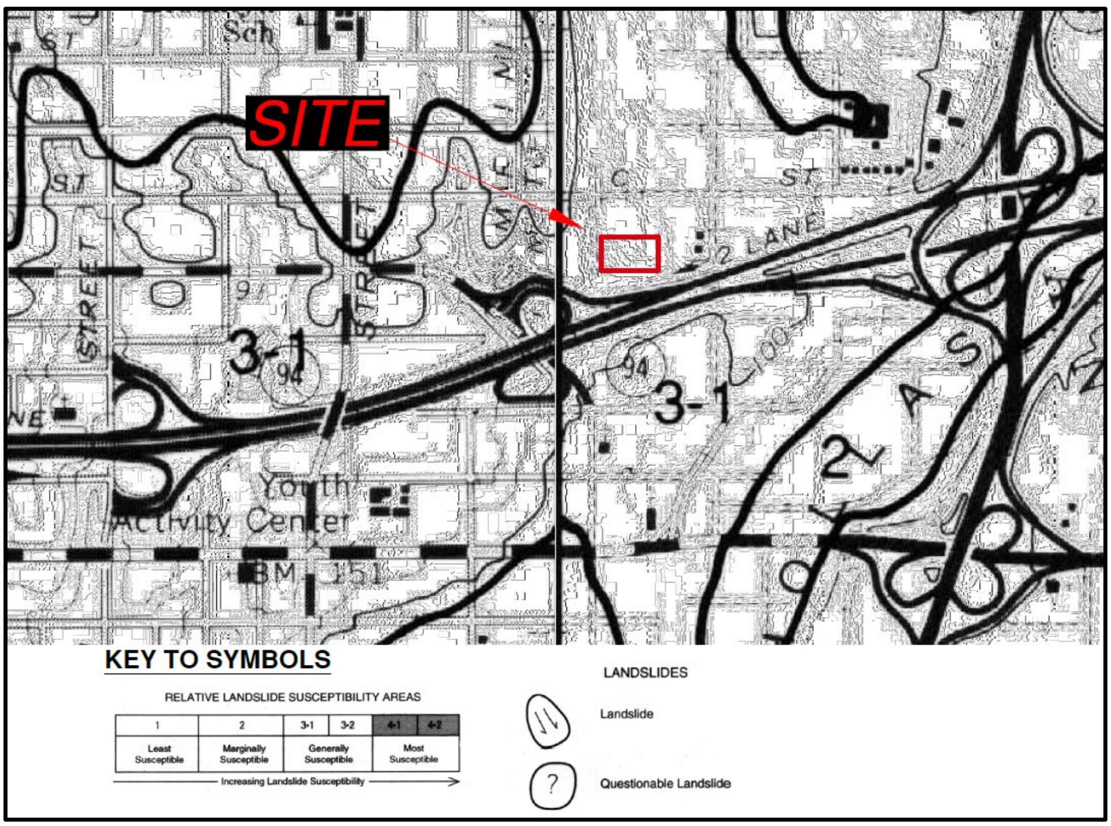


Figure 5-3. Landslide Susceptibility Mapping of the Site Vicinity

In consideration of the foregoing, including the indications of Figure 5-2 and Figure 5-3, it is NOVA's judgment that the landslide hazard is 'low' for the site and its immediately 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.'

There is no detailed planning available to review the extent of expected embankments. However, anticipating engineered fills that may be as thick as 12 feet, such slopes will have to be engineered and properly protected to mitigate this future risk.

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.

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

Seismically Induced Settlement

Apart from liquefaction, a strong seismic event can induce settlement within loose to moderately dense, unsaturated granular soils. The soils of this site will not be prone to seismic settlement.

Lateral Spreading

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

5.3.3 Expansive Soil

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

As is discussed in Section 3, the soils have been characterized by testing to determine Expansion Index ('EI' after ASTM D 4829). Originally developed in Orange County in the 1960s, EI is a basic soil index property, comparable to indices such as the Atterberg limits of soils. The expansion index has been judged by ASTM "*… to have a greater range and better sensitivity of expansion potential than other* *indices...*" EI has been adopted by the California Building Code ('CBC', Section 1803.5.3) for characterization of expansive soils.

EI testing of three remolded samples of the Unit 1 fill indicated 'Low' to 'Very Low' expansion potential. However, clay layers maybe encountered during mass grading at the site that will require mixing with other non-expansive soils to create a low expansive material (E.I<50).

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 2/Unit 3 eliminate concern for hydro-collapse.

5.3.5 Undocumented Fill

As is discussed in Section 4, the undocumented fill is considered potentially compressible. The fill is characteristically less than six feet in thickness.

Section 6 discusses design to adapt to the undocumented fill.

5.3.6 Corrosive Soils

Chemical testing of the near-surface soils indicates the soils may be corrosive. Embedded concrete will not be at risk for sulfate attack.

Section 6 addresses this consideration in more detail.

5.4 Siting Hazards

5.4.1 Effect on Adjacent Properties

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

5.4.2 Flood

The site is located within a FEMA-designated flood zone, flood map No. 06073C1903G dated May 16, 2012. The site area is designated "Zone X," indicating the site is within an area of minimal flood hazard. Figure 5-4 reproduces flood mapping by FEMA of the site area.

Figure 5-4. Flood Mapping of the Site Area
(source: adapted from FEMA Flood Map 06073C1903G, Revised May 16, 2012)

5.4.3 Tsunami

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

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

6.0 EARTHWORK AND FOUNDATIONS

6.1 Overview

6.1.1 Review of Site Hazards

Section 5 provides a review of soil and geologic hazards common to development of civil works in the project area. The primary hazard identified by that review is that the site is at risk for moderate-to-severe ground shaking in response to a large-magnitude earthquake during the lifetime of the planned development. While strong ground motion could affect the site, there is no risk of liquefaction or related seismic phenomena. The expectation of strong ground motion is common to all civil works in this area of California.

Section 6.2 addresses seismic design parameters.

6.1.2 Site Suitability.

Based upon the indications of the field and laboratory data developed for this investigation, as well as review of previously developed subsurface information, it is the opinion of NOVA that the site is suitable for development utilizing shallow foundations, provided the geotechnical recommendations described herein are followed.

Development as presently envisioned 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.

6.1.3 Review and Surveillance

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

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

6.2 Seismic Design Parameters

6.2.1 Site Class

The site-specific data used to determine the Site Class typically includes borings drilled to refusal materials to determine Standard Penetration resistances (N-values). The depth of soil information available for this site is limited, such that the site is classified as Site Class C per ASCE 7 (Table 20.3-1).

6.2.2 Seismic Design Parameters

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

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

Parameter	Value
Site Soil Class	C
Site Latitude (decimal degrees)	32.716219
Site Longitude (decimal degrees)	-117.124713
Site Coefficient, F_a	1.
Site Coefficient, F_v	1.369
Mapped Short Period Spectral Acceleration, S_S	1.123 g
Mapped One-Second Period Spectral Acceleration, S_1	0.431 g
Short Period Spectral Acceleration Adjusted For Site Class, S_{MS}	1.123 g
One-Second Period Spectral Acceleration Adjusted For Site Class, S_{M1}	0.59 g
Design Short Period Spectral Acceleration, S_{DS}	0.748 g
Design One-Second Period Spectral Acceleration, S_{D1}	0.393 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. These chemical tests were performed on a representative sample of the near-surface soils by Clarkson Laboratory and Supply, Inc. The results of the testing are tabulated on Table 6-2.

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

Parameter	Units	Value
pH	standard unit	5.4
Resistivity	Ω-cm	270
Water Soluble Chloride	ppm	880
Water Soluble Sulfate	ppm	260

6.3.2 Metals

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

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

Based on the Caltrans criteria, the on-site soils would be considered 'corrosive'. Appendix D 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

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 on Table 6-2, the soil sample tested indicated water-soluble sulfate (SO_4) content of 260 parts per million ('ppm,' 0.026% by weight). With $SO_4 < 0.10$ percent by weight, the American Concrete Institute (ACI) 318-08 considers a soil to have no potential (S0) for sulfate attack.

Table 6-4 reproduces the Exposure Categories considered by ACI.

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

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

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

6.3.4 Limitations

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

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

6.4 Earthwork

6.4.1 General

Based upon the known condition of the site and the design concept that is currently considered, NOVA expects that earthwork will include:

- relatively 'balanced' mass grading operations to adapt the sloping site to the development, cutting in the northwest and filling to the southeast; and,
- excavations for foundations and utilities.

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

6.4.2 Select Fill

Material Requirements

All fill or backfill for structures, utilities, walls, and pavements should be 'Select Fill', a mineral soil free of organics and regulated constituents with the characteristics listed below:

- with at least 40 percent by weight finer than ¼-inches in size,
- maximum particle size of 4 inches;
- classified as SW, SM, GW, GM after ASTM D 2487; and
- expansion index (EI) less than 10 (i.e., EI < 50, after ASTM D 4829).

The majority of the Unit 1 fill will conform to the above criteria.

Placement Requirements

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

Effective densification of the cohesionless (i.e., 'sandy') Select Fill will require the use of vibratory compaction methods, using equipment designed for this application. Anticipating the need to properly construct engineered fills to thicknesses of up to about 12 feet, prospective earthwork subcontractors should plan to employ self-propelled vibratory compactors with a minimum operating weight of 8 tons. Static compaction of these larger engineered fills that will support houses should not be allowed.

Select Fill should be placed in loose lifts no thicker than the ability of the vibratory compaction equipment to thoroughly densify the lift. For self-propelled construction equipment that conforms to the criteria of this section, this will limit loose lifts to on the order of 10-inches or less, applying a minimum of four passes in the forward direction.

Lift thickness for hand-operated vibratory equipment (tampers, walked behind compactors, etc.) will be limited to on the order of 4 inches or less.

6.4.3 Site Preparation

Erosion & Sedimentation Control BMPs

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.

Clearing

Prior to the start of earthwork, the site should be cleared of vegetation, organics-affected top soil, existing pavements and relic foundations. The deleterious materials should be disposed of in approved off-site or on-site locations.

6.4.4 Excavation Characteristics

The Unit 1 fill and Unit 2 Paralics will be readily excavated by earthwork equipment usual for developments of this nature. Locally, some ripping or other special excavation techniques (for example, the use of hoe rams to loosen Unit 2/Unit 3 in utility excavations) will be required.

6.4.5 Earthwork and Grading

General

Design to adapt the existing site groundform to the planned development with approximately balanced earthwork (i.e., the volume of cuts and fill across the site will be approximately equal) will require cuts of up to about 10 feet in the northwest portion of the site, with filling up to about 11 feet at the central and southern portion of the site. Of particular concern in this regard will be attention to the following:

1. Cut/Fill Transitions. Building areas that are underlain by both cuts to the formational sandstones and fill must have earthwork design to adapt to the differential stiffnesses of these foundation materials.

2. Engineered Fill Areas. Care must be taken in development of areas of engineered fill, taking care to ensure that these fills are developed with careful attention to creation of a thorough and high degree of compacted soil. Section 6.4.2 addresses equipment requirements for these thicker engineered fills.

The following subsections address these considerations.

Cut/Fill Transitions

Where the building or wall foundations are underlain by a combination of fill and Unit 2 or Unit 3 formational materials, the Unit 2/Unit 3 formational materials should be over-excavated to at least 18 inches below the base of the building foundation and backfilled with Select Fill that conforms with the material and placement requirements identified in Section 6.4.2.

Engineered Fill Areas

In areas to receive new fills, the existing undocumented fills should be removed to contact with the underlying Unit 2 or 3 (formational soils). The bottom of removals should be approved by NOVA. After approval, engineered fill beneath the buildings should be placed in conformance with the criteria identified in Section 6.4.2. Of particular consequence in this regard will be the need for densification of the Select Fill at 'molding' moisture contents about 2% above the optimum and application of thorough densification by heavy vibratory compacted equipment. NOVA expects that throw compaction of a 10" lift of Select Fill will require a minimum of 4 passes in the forward direction of a vibratory compactor that conforms with the criteria of Section 6.4.2.

6.4.6 Maintenance of Moisture in Soils During Construction

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

6.4.7 Trenching and Backfilling for Utilities

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

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

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

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

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

6.4.8 Slope Construction

In areas to support fill slopes, keys should be cut into competent soils. Keys should be at least five feet wide and be sloped back into the hillside at least two percent. The keys should extend at least one foot into the competent supporting materials. Where the existing ground has a slope of 5:1 (horizontal to vertical) or steeper, it should be benched into as the fill extends upward from the keyway.

Compaction of fill slopes should be performed by back-rolling with a sheepsfoot compactor at vertical intervals of four feet or less as the fill is being placed, and track-walking the face of the slope when the slope is completed. If space allows, the fill slopes may alternatively be overfilled by at least three feet and then cut back to the compacted core at the design line and grade.

6.4.9 Flatwork

Prior to casting exterior flatwork, the upper 12” of subgrade soils should be removed and replaced with “Select” fill, moisture conditioned and recompacted, as recommended in Section 6.4.5. Concrete slabs for pedestrian traffic or landscaping should be at least four (4) inches thick.

6.5 Shallow Foundations

6.5.1 General

Foundations for the planned buildings can be supported on shallow foundations provided the earthwork is completed as described in Section 6.4. The following subsections provide recommendations for shallow foundations.

6.5.2 Conventionally Reinforced Concrete Slab

The ground level slab of the building and garage structures may employ conventional on-grade (ground-supported) slab, designed using a modulus of subgrade reaction (k) of 180 pounds per cubic inch (i.e., k = 180 pci).

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

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).

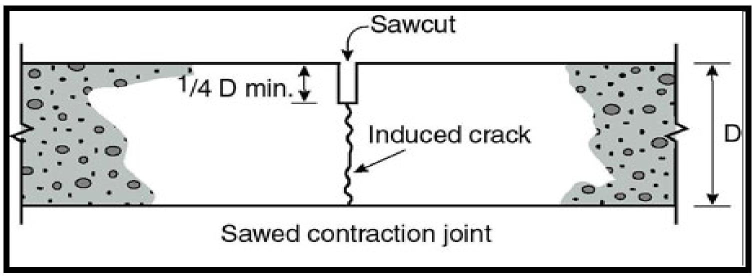


Figure 6-1. Sawed Contraction Joint

6.5.3 Conventional Foundations

Conventional foundations, either isolated and continuous footings, may be employed as described below.

Isolated Foundations

Isolated foundations for interior columns may be designed for an allowable contact stress of 3,000 psf. This value may be increased by one-third for transient loads such as wind and seismic. These foundation units should have a minimum width of 30 inches, embedded a minimum of 24 inches below lowest adjacent grade.

Continuous Foundations

Continuous foundations may be designed for an allowable contact stress of 2,500 psf, for footings with a minimum of 18 inches in width and embedded 24 inches below lowest adjacent grade. 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 may be resisted by passive earth pressure against the face of the footing, calculated as a fluid density of 250 psf per foot of depth, neglecting the upper 1 foot of soil below surrounding grade in this calculation. Additionally, a coefficient of friction of 0.35 between soil and the concrete base of the footing may be used with dead loads.

Settlement

If the building is supported as recommended above, it will settle on the order of 0.5 inch. This movement will occur elastically, as dead load (DL) and permanent live loads (LL) are applied. In usual circumstance, about 80% 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).

Setback from Slopes

The face of foundations near descending 2:1 (H:V) slopes should be set back at least 7 feet from the face of the slope at that point. The project structural engineer should include footing daylight requirements in their plans.

6.6 Capillary Break and Underslab Vapor Retarder

6.6.1 Capillary Break

NOVA recommends that the requirements for a capillary break ('sand layer') 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.

6.6.2 Vapor Retarder

Design Responsibility

Soil moisture vapor that penetrates ground-supported concrete slabs can result in damage to moisture-sensitive floor covering, some floor sealers, or sensitive equipment in direct contact with the floor.

It is not the responsibility of the geotechnical consultant to provide recommendations for vapor retarders to address this concern. This responsibility usually falls to the Architect. Decisions regarding the appropriate vapor retarder are principally driven by the nature of the building space above the slab, floor coverings, anticipated penetrations, concerns for mold or soil gas, and a variety of other environmental, aesthetic and materials factors known only to the Architect.

Design Guidance

A variety of specialty polyethylene (polyolefin)-based vapor retarding products are available to retard moisture transmission into and through concrete slabs.

Guidance 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*

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 are specified by the ASTM E1745 class rating.

Quality Assurance

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 Quality Assurance (QA) surveillance.

6.7 Control of Moisture Around Foundations

6.7.1 General

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

6.7.2 Erosion and Moisture Control During Construction

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

6.7.3 Design

Civil, structural, architectural and landscaping design for the areas around foundations should be undertaken with a view to the maintenance of an environment that encourages drainage away from below grade walls. Roof and surface drainage, landscaping, and utility connections should be designed to limit the potential for mounding of water near subterranean walls. In particular, rainfall to roofs should be collected in gutters and discharged away from foundations.

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

6.8 Retaining Walls

6.8.1 Lateral Pressures

Lateral earth pressures to permanent below grade garage walls are related to the type of backfill, drainage conditions, slope of the backfill surface, and the allowable rotation of the wall. The groundwater level will be well below the garage wall levels.

Table 6-5 (following page) provides recommendations for lateral soil and groundwater wall loading to below grade walls with level backfill for varying conditions of wall yield.

Table 6-5. Wall Lateral Loads from Soil

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: site-sourced Unit 1 sands 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. Surcharge loading should consider wall loads that may develop from adjacent streets and sidewalks.

6.8.2 Seismic Increment

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

6.8.3 Foundation Uplift

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

6.8.4 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.

6.8.5 Wall Drainage

The above recommendations assume a wall drainage panel or a properly compacted granular free-draining backfill material (EI <50).

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

6.9 Temporary Slopes

6.9.1 Conformance with OSHA and Cal/OSHA

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

Based on the data interpreted from the borings, the design of temporary slopes in Unit 2 and Unit 3 may assume California Occupational Safety and Health Administration (Cal/OSHA) Soil Type B for planning purposes. The Unit 1 soils may be assumed to be Type C.

6.9.2 Excavation Planning and Control

The face of temporary excavations 5 feet deep or less in the Unit 1 fill should not be steeper than 1:1 (horizontal: vertical). Temporary excavations in Unit 2/Unit 3 should not be steeper than ¾:1.

Surcharge loads to temporary slopes should not be permitted within a distance equal to the height of the excavation measured from the top of the excavation. The top of the excavation should be a minimum of 15 feet to the edge of existing improvements. Excavations (i) steeper than those recommended; or, (ii) closer than 15 feet from an existing service improvement, should be shored in accordance with applicable OSHA regulations and codes.

The faces of 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 raveling should be brought to the attention of the Engineer and corrective action implemented before personnel begin working in the excavation. Excavated materials 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 the rainy season, berms are recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Slopes steeper than those described above or temporary excavations that extend below a plane inclined at 1½:1 (horizontal: vertical) downward from the outside bottom edge of existing structures or improvements will require shoring.

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 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 3.3 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. The proximity of the planned BMP location to foundations is further complicated by the expected need for widespread fill. In consideration of these factors, NOVA concludes that the site is not feasible for development of permanent stormwater infiltration Best Management Practices ('stormwater BMPs').

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

7.2 Proposed DMA

Coffey 2019 depicts planning for the proposed residential development indicating the proposed location for the stormwater BMPs in a Drainage Management Area ('DMA') at the southeast corner of the site.

Figure 7-1 (following page) depicts the location of the DMA. The figure also shows the location of related percolation testing ('P-1) and test trenches ('T-1' through 'T-6') by NOVA.

7.3 Infiltration Rates

7.3.1 General

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

Table 7-1. 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	+134	3.5	+130.5	0.16	0.08

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

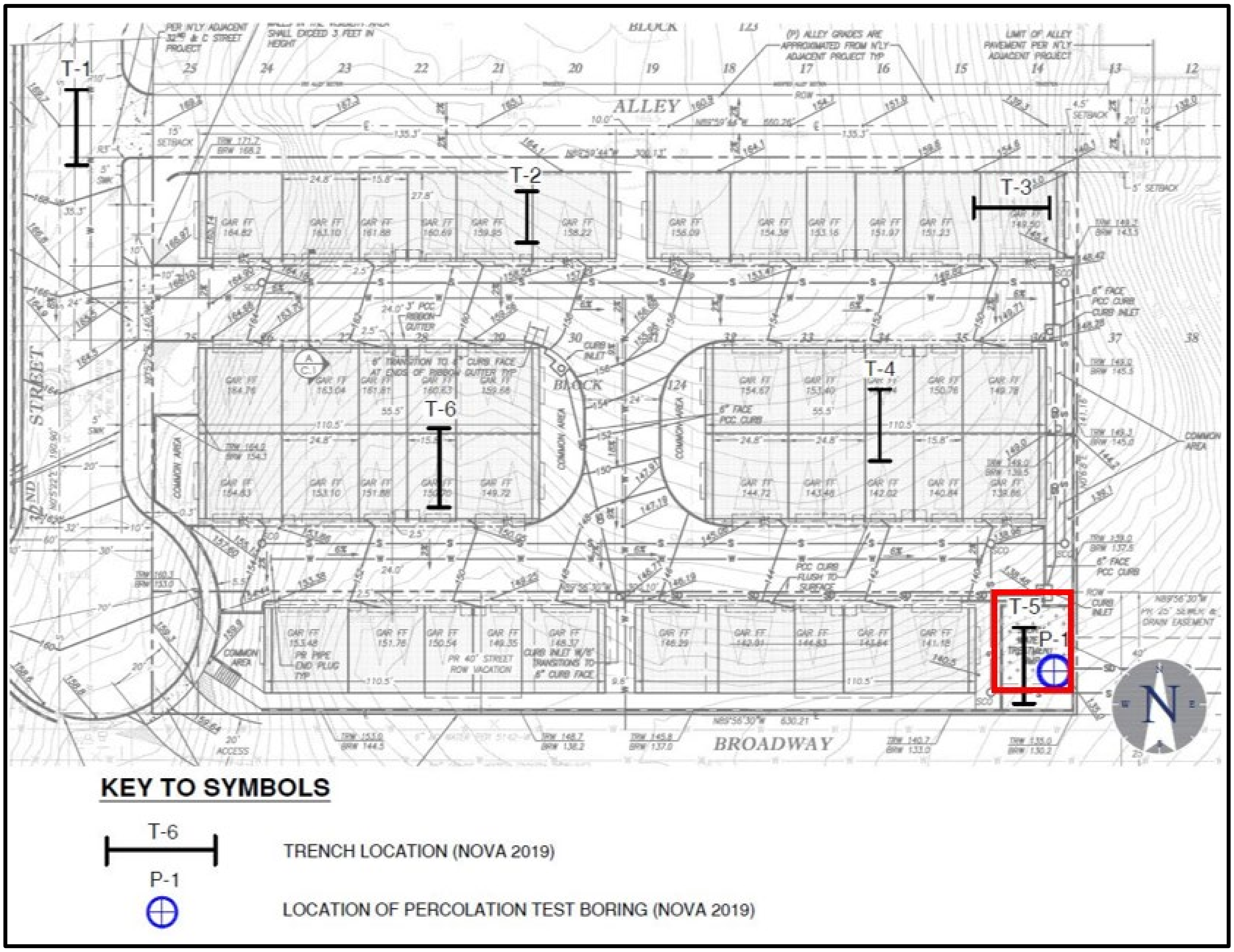


Figure 7-1. Proposed DMA and Percolation Test Location
(source: adapted from Coffey 2019)

7.3.2 Design Infiltration Rate

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 = 0.08 inches per hour.

7.4 Review of Conditions for Storm Water Infiltration

7.4.1 Overview of Subsurface Conditions

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

1. Unit 1, Fill. The site is covered by a mantle of fill approximately 1 to 5 feet in thickness. The fill is comprised of silty to clayey sands of loose to medium dense consistency and sandy clays of

firm consistency. Because no records exist regarding the placement of this fill, the fill is considered 'undocumented,' and at risk for wide variations in quality.

2. Unit 2, Very Old Paralics. Beneath the fill, the site is underlain by Quaternary-aged Very Old Paralic deposits (Qvop8). Locally, this unit outcrops at the ground surface. As encountered in the trenches, the unit is characteristically well cemented silty sandstone of dense to very dense consistency. The upper surface (about the upper 1'-2') of this unit includes abundant cobbles to typically 4".

3. Unit 3, San Diego Formation. Trench T-6, disclosed that the site is underlain by the Tertiary-aged San Diego Formation (Tsd) below the Very Old Paralics. Trenches T-1 through T-5 did not extend through the Very Old Paralics to expose this unit. As encountered at the site, San Diego Formation consists of medium dense and friable well-graded sandstone, similar in quality (low compressibility, high strength) to the Very Old Paralics.

7.4.2 Review of Feasibility Criteria

As stated in the BMP Design Manual, when one standard setback in the simple feasibility criteria cannot be achieved, the DMA is classified in a 'no infiltration' condition. At a minimum, the site fails the feasibility criteria listed below.

1. Foundations and Structures. Full or partial infiltration BMPs may not be proposed within 10 feet of structures or retaining walls. The planned BMP is located adjacent to the neighboring structure. Water infiltrating through soil may weaken foundation soils/rock.

2. Fill Depth. The proposed BMP and much of the site is located in an area that will receive approximately 11 feet of fill. Extension of the BMP down to natural soil may prove infeasible in areas of considerable fill depth.

7.5 Recommendation for 'No Infiltration'

In consideration of the foregoing, it is the judgment of NOVA that the site is not suitable for full or partial BMPs.

8.0 PAVEMENTS

8.1 Overview

8.1.1 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.

8.1.2 Design to Limit Infiltration

The surface grades of pavements and related design features to limit infiltration should conform with the concepts discussed in Section 6.

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).

8.1.3 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.

8.1.4 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.

8.2 Pavement Subgrade Preparation

Remedial grading for paved areas should be as described in Section 6.4.3, removing the upper 2 feet of the Unit 1 undocumented fill, 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.

8.3 Flexible Pavements

The structural design of flexible pavement depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. Table 8-1 (following page) provides preliminary flexible pavement sections using an R-value of 28. This R-value was indicated by laboratory testing described in Section 3.

8.4 Rigid Pavements

8.4.1 General

Concrete pavement sections should be developed in the same manner as undertaken for all other slabs and pavements: removal of the Unit 1 undocumented fill and replacement of that material in an engineered manner as described in Section 8.2.

Concrete pavement sections consisting of 7 inches of Portland cement concrete over a base course of 4 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 8-2 (following page).

Table 8-1. Preliminary Pavement Sections, R = 28

Area	Subgrade R-Value	Traffic Index	Asphalt Thickness (in)	Base Course Thickness (in)
Auto Parking	28	5	4.0	5.0
Roadways/Fire Lane	28	6	4.0	7.0

1. The above sections assume properly prepared subgrade consisting of at least 12 inches of subgrade compacted to a minimum of 90% 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.

Table 8-2. 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

8.4.2 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.

9.0 REFERENCES

9.1 Site Specific

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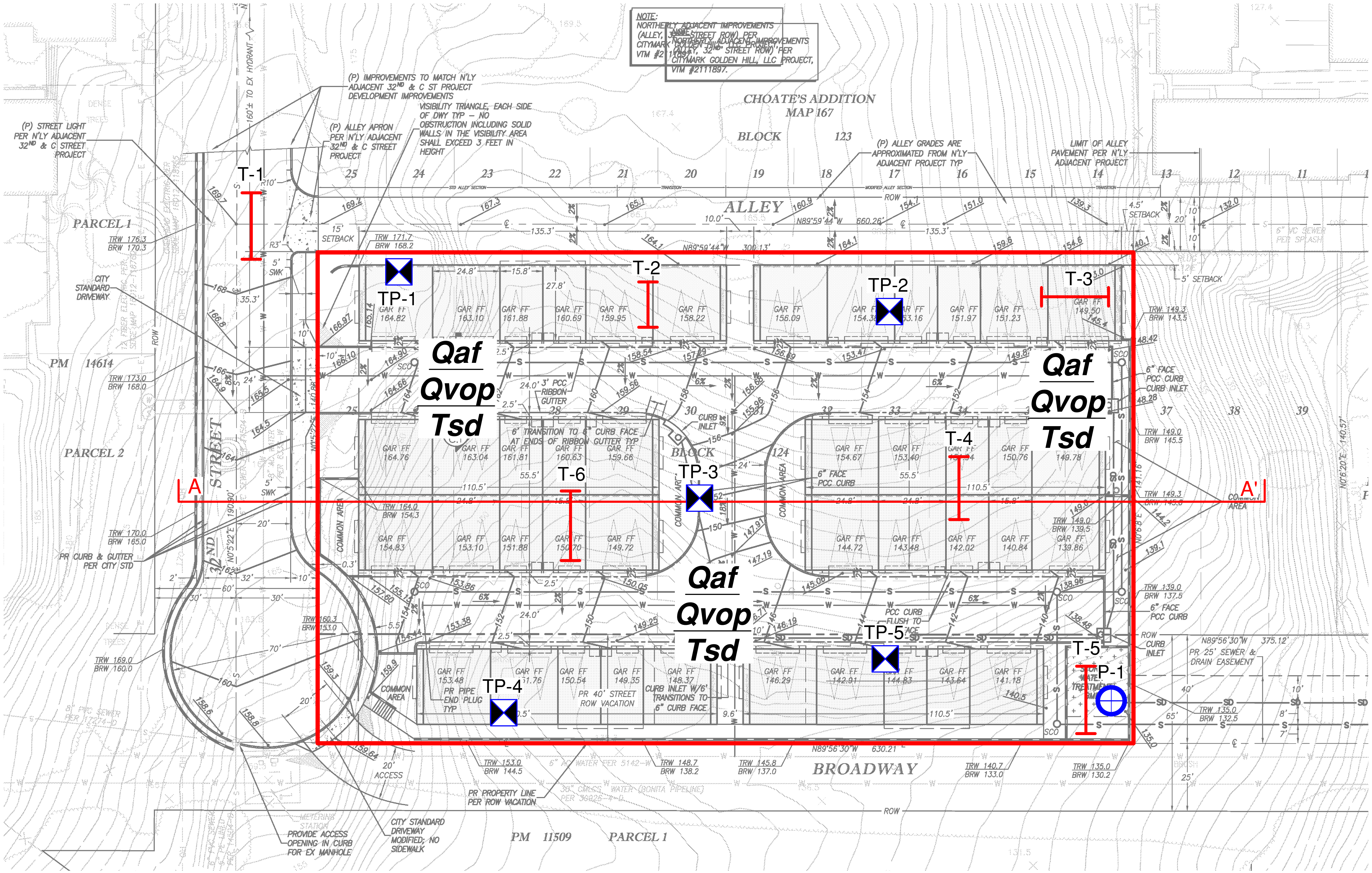
PLATES

NOVA



KEY TO SYMBOLS

Qaf	FILL	(red wavy line)	LIMITS OF ANTICIPATED REMEDIAL GRADING	TP-5	TEST PIT LOCATION (CTE 2006)
Qvop	VERY OLD PARALIC DEPOSITS	T-6	TRENCH LOCATION (NOVA 2019)	A A'	GEOLOGIC CROSS-SECTION
Tsd	SAN DIEGO FORMATION	P-1	LOCATION OF PERCOLATION TEST BORING (NOVA 2019)		

CITY OF SAN DIEGO, CALIFORNIA
TENTATIVE MAP
32nd & Broadway Rowhomes
1000 Block 32nd Street San Diego, CA 92103
TENTATIVE MAP DRAWING C.1
SHEET 1 OF 1

COFFEY ENGINEERING, INC.
9666 BUSINESSPARK AVENUE, SUITE 210, SAN DIEGO, CA 92131 PH (858)831-0111 FAX (858)831-0179

NOVA
4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA

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

WWW.USA-NOVA.COM

32ND & BROADWAY
1000 BLOCK & 32ND STREET
SAN DIEGO, CALIFORNIA

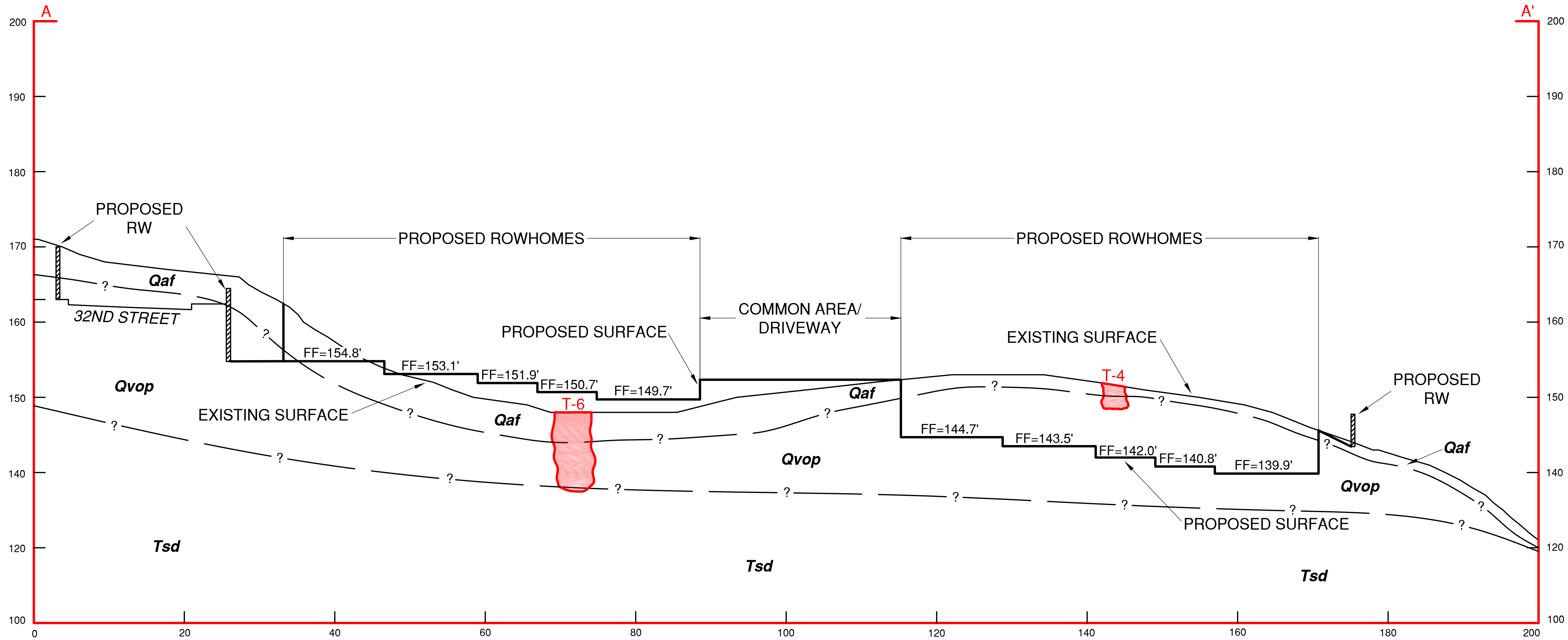
PROJECT NO:	2019066
DATE:	MAY 2019
DRAWN BY:	DTW
REVIEWED BY:	DM

SUBSURFACE INVESTIGATION MAP

N

0 20' 40'

PLATE 1



KEY TO SYMBOLS

Qaf	FILL
Tsd	SAN DIEGO FORMATION
Qvop	VERY OLD PARALIC DEPOSITS
(red box)	TRENCH LOCATION (NOVA 2019)
?	GEOLOGIC CONTACT
FF	FINISHED FLOOR

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1000 BLOCK & 32ND STREET
SAN DIEGO, CALIFORNIA

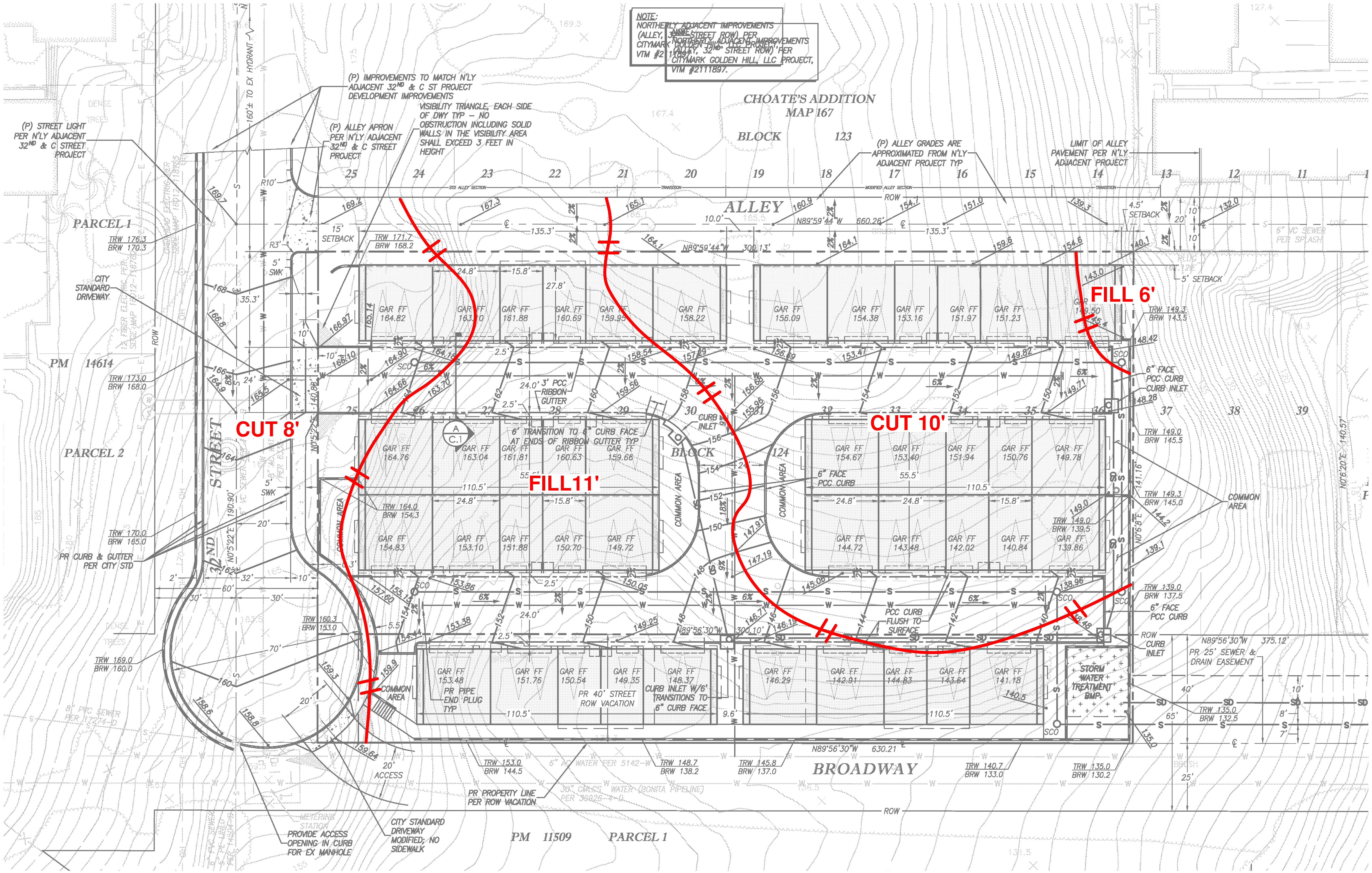
PROJECT NO: 2019066
DATE: MAY 2019
DRAWN BY: DTW
REVIEWED BY: WM

GEOLOGIC CROSS-SECTION AA'

0 16' 32'

PLATE 2

TENTATIVE MAP No. ________



KEY TO SYMBOLS

CUT & FILL

*NOTE: FILLS UP TO, CUTS UP TO

COFFEY ENGINEERING, INC.

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CITY OF SAN DIEGO, CALIFORNIA	
TENTATIVE MAP	
32nd & Broadway Rowhomes 1000 Block 32nd Street San Diego, CA 92103	ORIGINAL 2/__/19 REVISIONS
TENTATIVE MAP DRAWING	C.1
DRAWN BY: DK CHECKED BY: JC	SHEET 1 OF 1

NOVA

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32ND & BROADWAY

1000 BLOCK & 32ND STREET
SAN DIEGO, CALIFORNIA

PROJECT NO:	2019066
DATE:	MAY 2019
DRAWN BY:	DTW
REVIEWED BY:	DM

PLANNED EARTHWORK

0 20' 40'

PLATE 3

APPENDIX A

USE OF THE GEOTECHNICAL REPORT

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

APPENDIX B

LOGS OF TRENCHES

NOVA

TRENCH LOG T-1

DATE EXCAVATED:	APRIL 1, 2019	**EQUIPMENT:**	BACKHOE
EXCAVATION DESCRIPTION:	TRENCH EXCAVATION	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 173 FT MSL (OBTAINED FROM GOOGLE EARTH)

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)	DCP INCHES PER 25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0	SM	5.75 3.5	**FILL (Qaf):** *SILTY SAND; YELLOW BROWN TO BROWN, DRY TO DAMP, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL, SCATTERED COBBLE ≤ 12", TRACE ASPHALT CONCRETE AND CONCRETE DEBRIS*	MD SA RV	
5	CL	4	*SANDY CLAY; BROWN TO DARK BROWN, DAMP TO MOIST, FIRM TO STIFF*	EI	*28 LOW*
	SM	2	**VERY OLD PARALIC DEPOSITS (Qvop):** *SILTY SANDSTONE; LIGHT BROWN TO ORANGE, DAMP, MEDIUM DENSE TO DENSE, FINE TO MEDIUM GRAINED, ABUNDANT GRAVEL, ABUNDANT COBBLE ≤ 10"* *TRACE GRAVEL, TRACE COBBLE FROM 8-9'* *ABUNDANT GRAVEL, SOME COBBLE ≤ 10"*		
10			*TRENCH TERMINATED AT 10 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
15					

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
◪	DCPT (ASTM D6951)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

32ND & BROADWAY HOMES
1000 BLOCK 32ND STREET
SAN DIEGO, CALIFORNIA

LOGGED BY:	DEM	DATE:	MAY 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019066	

NOVA

APPENDIX B.1

TRENCH LOG T-2

DATE EXCAVATED:	APRIL 1, 2019	**EQUIPMENT:**	BACKHOE
EXCAVATION DESCRIPTION:	TRENCH EXCAVATION	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 160 FT MSL (OBTAINED FROM GOOGLE EARTH)

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	DCP TEST	SOIL CLASS. (USCS)	DCP INCHES PER 25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0		X	DCPT	SM	3	**FILL (Qaf):** *SILTY SAND WITH COBBLE; BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, ABUNDANT COBBLE ≤ 10", SOME GRAVEL*		
		X	DCPT	SM	1.5	**VERY OLD PARALIC DEPOSITS (Qvop):** *SILTY SANDSTONE; LIGHT BROWN, DAMP, DENSE TO VERY DENSE, FINE TO MEDIUM GRAINED, TRACE GRAVEL*		
						TRENCH TERMINATED AT 3 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5								
10								
15								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
⊠	BULK SAMPLE	*	NO SAMPLE RECOVERY
◺	DCPT (ASTM D6951)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

32ND & BROADWAY HOMES
1000 BLOCK 32ND STREET
SAN DIEGO, CALIFORNIA

NOVA

LOGGED BY:	DEM	DATE:	MAY 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019066	

APPENDIX B.2

TRENCH LOG T-3

DATE EXCAVATED: APRIL 1, 2019
EQUIPMENT: BACKHOE
EXCAVATION DESCRIPTION: TRENCH EXCAVATION
GPS COORD.: N/A
GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED
ELEVATION: ± 150 FT MSL (OBTAINED FROM GOOGLE EARTH)

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	DCP TEST	SOIL CLASS. (USCS)	DCP INCHES PER 25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0				SM		**FILL (Qaf):** *SILTY SAND; BROWN TO GRAY BROWN, DRY, LOOSE, FINE TO MEDIUM GRAINED, GRAVEL, ABUNDANT COBBLE ≤ 12"* *DRY TO DAMP*		
				SM	1	**VERY OLD PARALIC DEPOSITS (Qvop):** *SILTY SANDSTONE; LIGHT BROWN, DAMP, DENSE TO VERY DENSE, FINE TO MEDIUM GRAINED, TRACE GRAVEL*		
						TRENCH TERMINATED AT 3.5 FT DUE TO REFUSAL OF THE EXCAVATION EQUIPMENT ON VERY DENSE Qvop. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5								
10								
15								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
⊠	BULK SAMPLE	*	NO SAMPLE RECOVERY
◿	DCPT (ASTM D6951)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

32ND & BROADWAY HOMES
1000 BLOCK 32ND STREET
SAN DIEGO, CALIFORNIA

LOGGED BY:	DEM	DATE:	MAY 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019066	

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APPENDIX B.3

TRENCH LOG T-4

DATE EXCAVATED:	APRIL 1, 2019	**EQUIPMENT:**	BACKHOE
EXCAVATION DESCRIPTION:	TRENCH EXCAVATION	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 150 FT MSL (OBTAINED FROM GOOGLE EARTH)

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	DCP TEST	SOIL CLASS. (USCS)	DCP INCHES PER 25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0		☒	◩	CL	5.75	**FILL (Qaf):** *SANDY CLAY; DAMP, SOFT TO FIRM, SCATTERED GRAVEL, COBBLE ≤ 10" WITHIN TOP 6"*		
		☒	◩	SM	1.25	**VERY OLD PARALIC DEPOSITS (Qvop):** *SILTY SANDSTONE; LIGHT GRAY BROWN, DRY TO DAMP, DENSE TO VERY DENSE, FINE TO MEDIUM GRAINED*	CR SA	
						TRENCH TERMINATED AT 3 FT DUE TO REFUSAL OF THE EXCAVATION EQUIPMENT ON VERY DENSE Qvop. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5								
10								
15								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
◩	DCPT (ASTM D6951)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

32ND & BROADWAY HOMES
1000 BLOCK 32ND STREET
SAN DIEGO, CALIFORNIA

LOGGED BY:	DEM	DATE:	MAY 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019066	

NOVA

APPENDIX B.4

TRENCH LOG T-5

DATE EXCAVATED:	APRIL 1, 2019	**EQUIPMENT:**	BACKHOE
EXCAVATION DESCRIPTION:	TRENCH EXCAVATION	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 132 FT MSL (OBTAINED FROM GOOGLE EARTH)

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	DCP TEST	SOIL CLASS. (USCS)	DCP INCHES PER 25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0		☒		CL		**FILL (Qaf):** *SANDY CLAY; DAMP, SOFT TO FIRM, SCATTERED GRAVEL, COBBLE ≤ 10" WITHIN TOP 6"*	EI	*16 VERY LOW*
		☒		SP-SM		**VERY OLD PARALIC DEPOSITS (Qvop):** *POORLY-GRADED SANDSTONE-SILTY SANDSTONE; LIGHT BROWN, DRY TO DAMP, DENSE TO VERY DENSE, FINE TO MEDIUM GRAINED*	SA	
5						*TRENCH TERMINATED AT 5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
10								
15								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	DCPT (ASTM D6951)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

32ND & BROADWAY HOMES
1000 BLOCK 32ND STREET
SAN DIEGO, CALIFORNIA

LOGGED BY:	DEM	DATE:	MAY 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019066	

NOVA

APPENDIX B.5

TRENCH LOG T-6

DATE EXCAVATED:	APRIL 1, 2019	**EQUIPMENT:**	BACKHOE
EXCAVATION DESCRIPTION:	TRENCH EXCAVATION	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 147 FT MSL (OBTAINED FROM GOOGLE EARTH)

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	DCP TEST	SOIL CLASS. (USCS)	DCP INCHES PER 25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0		X	X	CL	7	**FILL (Qaf):** *SILTY CLAY; BROWN, DAMP TO MOIST, SOFT TO FIRM, SOME COBBLE ≤ 12" WITHIN THE TOP 6"*		
		X		SM		*SILTY SAND; YELLOW BROWN MOTTLED GRAY, DAMP TO MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*	EI SA	*6 VERY LOW*
5		X	X	CL	7.5	**VERY OLD PARALIC DEPOSITS (Qvop):** *SILTY CLAY; GRAY MOTTLED YELLOW BROWN, DAMP TO MOIST, SOFT TO FIRM*		
						DAMP		
		X		SW		**SAN DIEGO FORMATION (Tsd):** *WELL-GRADED SAND; LIGHT BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, TRACE GRAVEL*		
10						*TRENCH TERMINATED AT 10 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
15								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
⊠	BULK SAMPLE	*	NO SAMPLE RECOVERY
◪	DCPT (ASTM D6951)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

32ND & BROADWAY HOMES
1000 BLOCK 32ND STREET
SAN DIEGO, CALIFORNIA

NOVA

LOGGED BY:	DEM	DATE:	MAY 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019066	

APPENDIX B.6

PERCOLATION BORING LOG P-1

DATE EXCAVATED:	APRIL 1, 2019	**EQUIPMENT:**	HAND TOOLS
EXCAVATION DESCRIPTION:	8-INCH DIAMETER HAND AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 132 FT MSL (OBTAINED FROM GOOGLE EARTH)

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	DCP TEST	SOIL CLASS. (USCS)	DCP INCHES PER 25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0				CL		**FILL (Qaf):** *SANDY CLAY; DAMP, SOFT TO FIRM, SCATTERED GRAVEL, COBBLE ≤ 10" WITHIN TOP 6"*		
				SP-SM		**VERY OLD PARALIC DEPOSITS (Qvop):** *POORLY-GRADED SANDSTONE-SILTY SANDSTONE; LIGHT BROWN, DRY TO DAMP, DENSE TO VERY DENSE, FINE TO MEDIUM GRAINED*		
						BORING TERMINATED AT 3.5 FT AND CONVERTED TO A PERCOLATION WELL.		
5								
10								
15								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
◿	DCPT (ASTM D6951)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

32ND & BROADWAY HOMES
1000 BLOCK 32ND STREET
SAN DIEGO, CALIFORNIA

LOGGED BY:	DEM	DATE:	MAY 2019
REVIEWED BY:	BMH	PROJECT NO.: 2019066	

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APPENDIX B.7

APPENDIX C

RECORDS OF PERCOLATION TESTING

NOVA

NOVA

GEOTECHNICAL ■ MATERIALS ■ SPECIAL INSPECTIONS

SBE ■ SLBE ■ SCOOP

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

32nd & Broadway, LLC
3184 Airway Avenue, Suite B
Costa Mesa, CA 92626

May 24, 2019
NOVA Project No. 2019066

Attention Ben C. Anderson

Subject: Assessment of Infiltration Feasibility
Proposed 32nd & Broadway Homes
1000 Block 32nd Street, San Diego, California

References: See Attachment.

Dear Mr. Anderson:

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 32nd & Broadway, LLC. NOVA is retained by 32nd & Broadway as Geotechnical Engineer-of-Record (GEOR) for the project.

Background

Current Site Conditions

Location
The residential development is proposed to be developed on a vacant parcel located southeast of the intersection of 32nd Street and C Street (hereafter, 'the site'). It is bounded to the north by a vacant lot, to the west by 32nd street, to the south by vacant land, and to the east by an existing apartment development. The apartment development abuts the property line to the east of the site. Figure 1 (following page) provides a recent aerial image depicting the site location.

Surface
The undeveloped site is currently lightly vegetated. Ground surface elevations across the site vary from 177 feet msl at the northwest corner to 130 feet msl at the southeast corner. The ground surface descends to the east and south.

Proposed BMP

Coffey 2019 depicts planning for the proposed residential development. The proposed location for the stormwater BMP is at the southeast corner at the periphery of the development. Figure 2 (following page) depicts the location of the BMPs. The figure also shows the location of related percolation testing ('P-1') and test trenches ('T-1' through 'T-6') by NOVA.

Figure 1. Site Location and Limits
(source: adapted from *Google Earth 2019)*

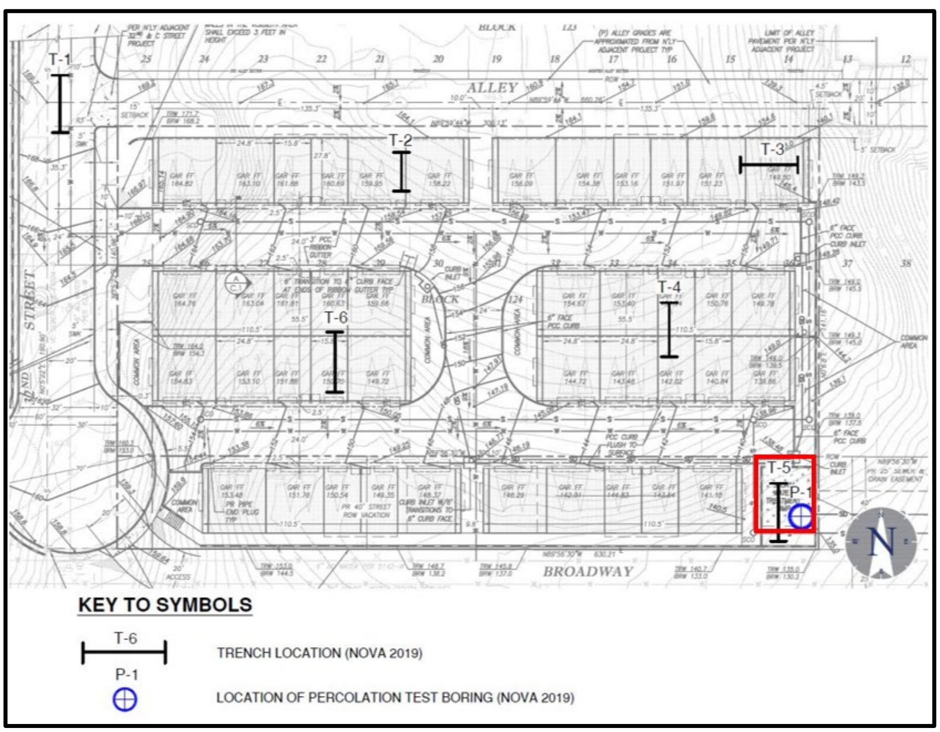

Figure 2. Proposed DMA and Percolation Test Location by NOVA
(source: adapted from Coffey 2019*)*

Percolation Testing

NOVA conducted percolation testing in the preliminary stages of planning for the site's development on April 1, 2019 and April 2, 2019. Testing was completed in accordance with procedures detailed in the referenced City of San Diego BMP Design Manual (San Diego 2018).

One percolation test boring (P-1) was drilled to 3.5 feet below ground surface (bgs). One exploratory trench (T-5) was drilled to 5 feet bgs. Table 1 summarizes the infiltration rates determined by the 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	+134	3.5	+130.5	0.16	0.08

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 = 0.08 inches per hour.

Review of Conditions for Storm Water Infiltration

Overview of Subsurface Conditions

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

1. Unit 1, Fill. The site is covered by a mantle of fill approximately 1 to 5.5 feet in thickness. The fill is comprised of silty to clayey sands of loose to medium dense consistency and sandy clays of firm consistency.

2. Unit 2, Paralics. Beneath the fill, the site is underlain by Quaternary-aged Very Old Paralic deposits (Qvop). The unit is characteristically cemented silty sandstone with gravel of dense to very dense consistency. The backhoe met refusal on very dense paralics in trenches T-3, T-4 and T-5. The paralics extend to below the depths explored in trenches T-1 through T-5.

3. Unit 3, San Diego. Trench T-6 exposed the Tertiary-aged San Diego Formation (Tsd). This formation is known to occur below the paralics across this area of San Diego. Trenches T-1 through T-5 did not extend through the Paralics to expose this unit. As encountered at the site, the San Diego Formation consists of medium dense and friable well-graded sandstone.

Review of Feasibility Criteria

As stated in the BMP Design Manual, when one standard setback in the simple feasibility criteria cannot be achieved, the DMA is classified in a 'no infiltration' condition. At a minimum, the site fails the feasibility criteria listed below.

1. Foundations and Structures. Full or partial infiltration BMPs may not be proposed within 10 feet of structures or retaining walls. The planned BMP is located adjacent to the neighboring structure. Water infiltrating through soil may weaken foundation soils/rock.

2. Fill Depth. The proposed BMP and much of the site is located in an area that will receive approximately 11 feet of fill. Extension of the BMP down to natural soil may prove infeasible in areas of considerable fill depth.

Recommendation for 'No Infiltration'

In consideration of the foregoing, it is the judgment of NOVA that the site is not suitable for full or partial BMPs.

Closure

NOVA appreciates the opportunity to be of service to 32nd & Broadway, LLC on this most interesting project. 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. San Diego 2018. *The City of San Diego Storm Water Standards, Part 1 BMP Design Manual,* October 2018 Edition, The City of San Diego.

2. CE 2019. *Tentative Map, 32nd & Broadway Rowhomes, 1000 Block 32nd Street, San Diego, California,* Coffey Engineering Inc., February 2019.

3. NOVA 2019. *Report, Geotechnical Investigation, Proposed 32nd & Broadway Homes, 1000 Block 32nd Street, San Diego, California,* NOVA Services, Inc., NOVA Project No. 2019066, May 24, 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:
Proposed BMP Location		Planning
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. See Geotechnical Investigation (NOVA 2019)	

<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 2: Geologic/Geotechnical Screening</td></tr>
<tr><td>2A</td><td colspan="3">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.</td></tr>
<tr><td>2A-1</td><td>Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td>2A-2</td><td>Can the proposed full 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>2A-3</td><td>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?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td>2B</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 2B are answered “Yes,” then answer “Yes” to Criteria 2 Result.
If there are “No” answers continue to Step 2C.</td></tr>
<tr><td>2B-1</td><td>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?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td>2B-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 full 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]	
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">Proposed BMP Location</td><td>Planning</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).
See Geotechnical Investigation (NOVA 2019)</td></tr>
</table>

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 4: Geologic/Geotechnical Screening			
4A	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.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	☒ Yes	☐ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐ Yes	☒ No
4A-3	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?	☐ Yes	☐ No
4B	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.		
4B-1	**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?	☐ Yes	☐ No
4B-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 partial 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]	
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.			
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.

APPENDIX D

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.*

- **EXPANSION INDEX (ASTM D4829):** *The expansion index of selected materials was evaluated in general accordance with ASTM D4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thich by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours.*

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

- **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.*

- **R-VALUE (ASTM D2844):** *The resistance Value, or R-Value, for near-surface site soils were evaluated in general accordance with California Test (CT) 301 and ASTM D2844. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results.*

- **GRADATION ANALYSIS (ASTM C 136 and/or ASTM D422):** *Tests were performed on selected representative soil samples in general accordance with ASTM D422. The grain size distributions of selected samples were determined in accordance with ASTM C 136 and/or ASTM D422. The results of the tests are summarized on Appendix D.3 through Appendix D.6.*

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

LAB TEST SUMMARY

32ND & BROADWAY
1000 BLOCK & 32ND STREET
SAN DIEGO, CALIFORNIA

BY: DTW	DATE: MAY 2019	PROJECT: 2019066	APPENDIX: D.1

Expansion Index (ASTM D4829)

Sample Location	Sample Depth (ft.)	Expansion Index	Expansion Potential
T-1	4.0'-5.0'	28	Low
T-5	0.0' - 2.0'	16	Very Low
T-6	2.0' - 3.0'	6	Very Low

EXPANSION INDEX	EXPANSION POTENTIAL
0 - 20	VERY LOW
21 - 50	LOW
51 - 90	MEDIUM
91 - 130	HIGH
131 AND ABOVE	VERY HIGH

Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

Sample Location	Sample Depth (ft.)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
T-1	0.0' - 3.0'	Yellow Brown to Brown Silty Sand	131.6	8.6

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

Sample Location	Sample Depth (ft.)	Soil Description	R-Value
T-1	0.0'-3.0'	Yellow Brown to Brown Silty Sand	28

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 (%)
T-4	2.0'-3.0'	5.4	270	260	0.026	880	0.088

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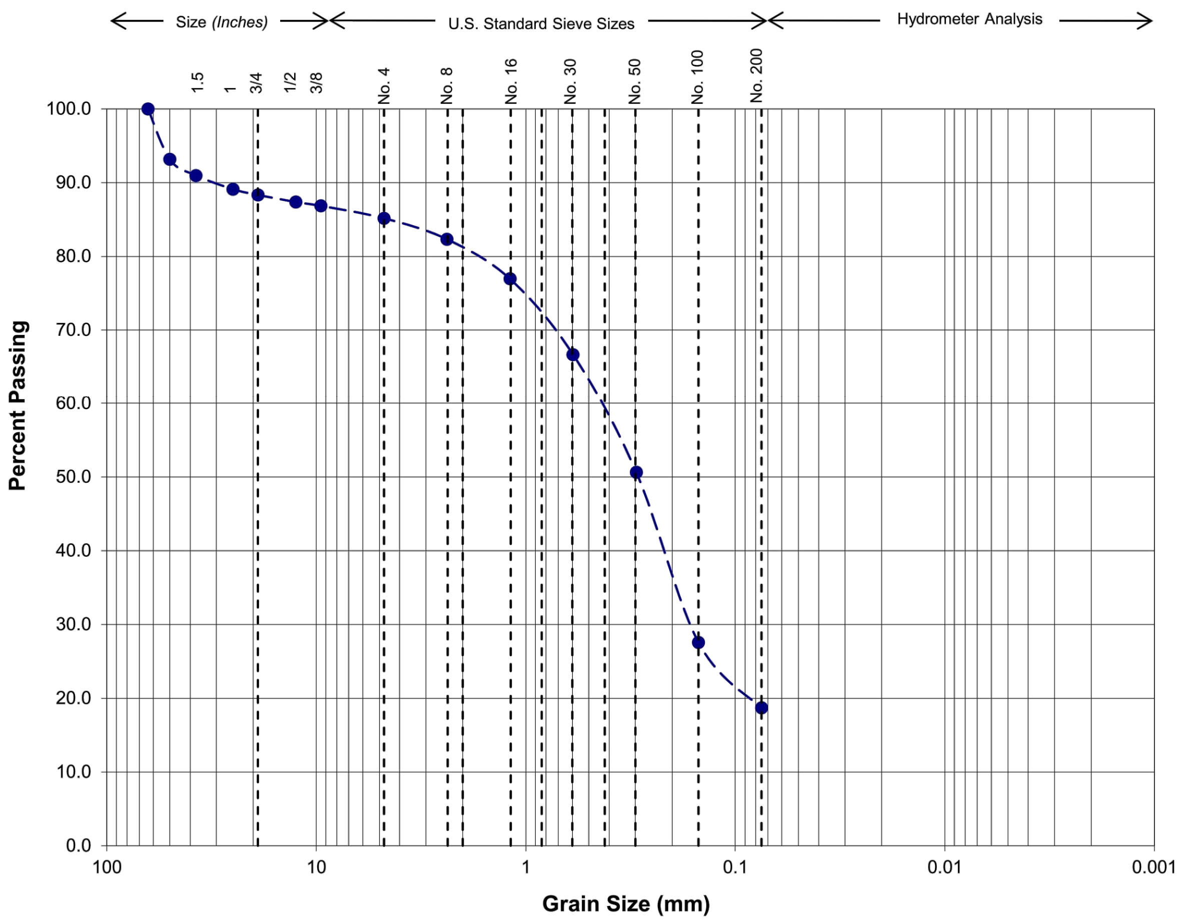
LAB TEST RESULTS

32ND & BROADWAY
1000 BLOCK & 32ND STREET
SAN DIEGO, CALIFORNIA

BY: DTW	DATE: MAY 2019	PROJECT: 2019066	APPENDIX: D.2



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: T-1

Depth (ft): 0.0'-3.0'

USCS Soil Type: SM

Passing No. 200 (%): 19

GRADATION ANALYSIS TEST RESULTS

32ND & BROADWAY

1000 BLOCK & 32ND STREET

SAN DIEGO, CALIFORNIA

BY: DTW	DATE: MAY 2019	PROJECT: 2019066	APPENDIX: D.3

NOVA
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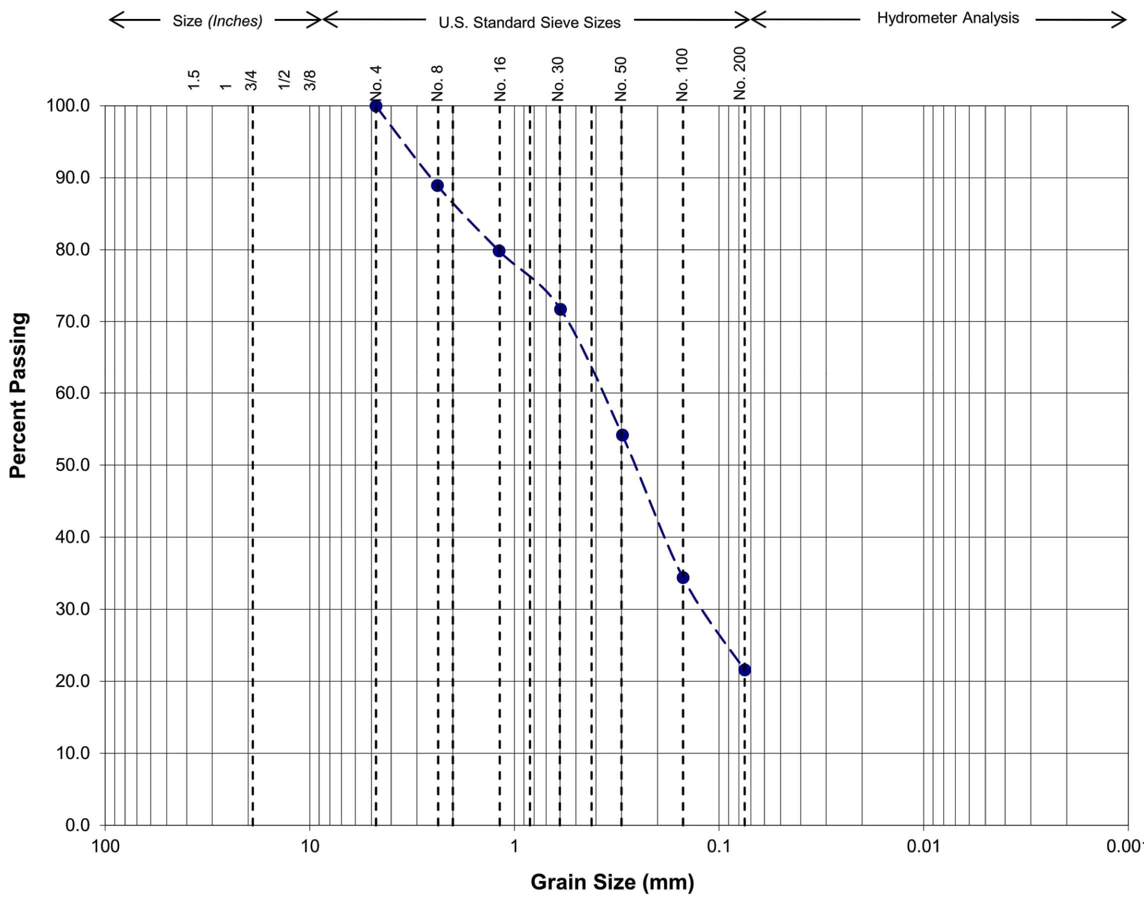
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: T-4

Depth (ft): 2.0'-3.0'

USCS Soil Type: SM

Passing No. 200 (%): 22

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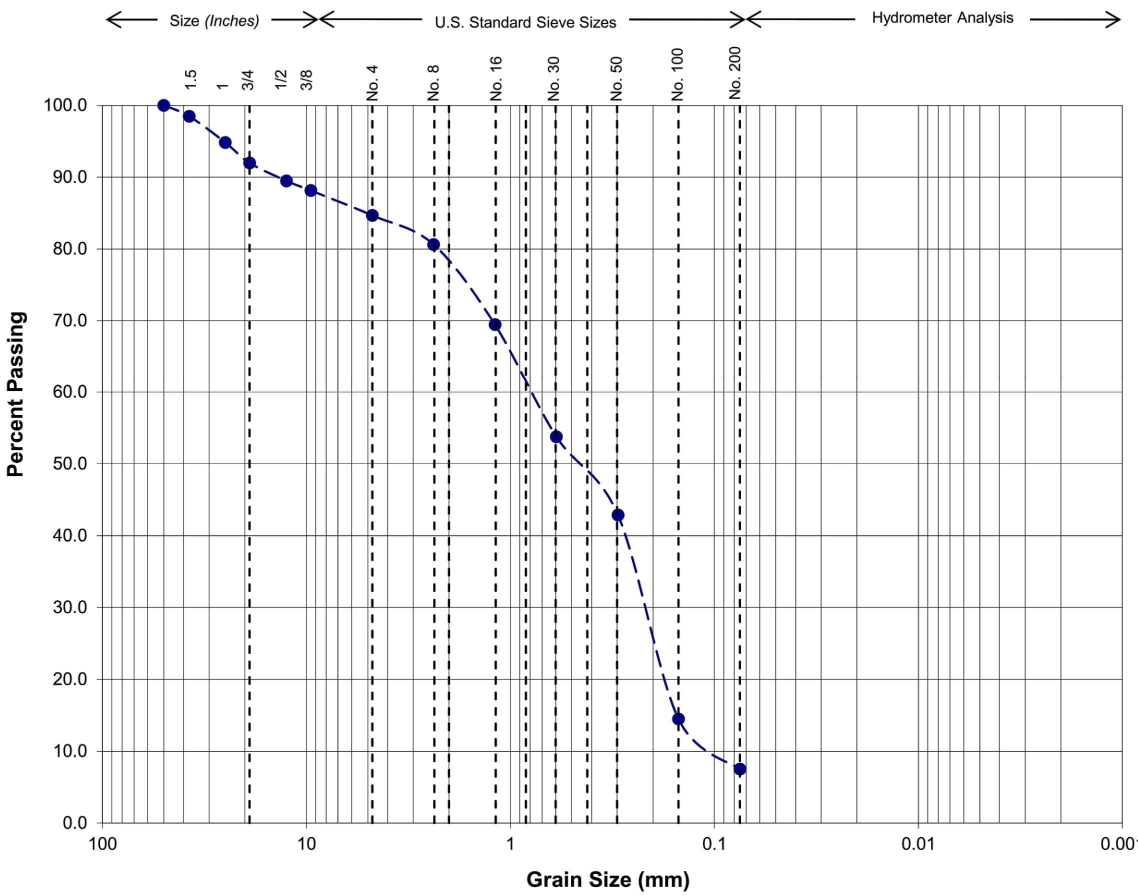
GRADATION ANALYSIS TEST RESULTS

32ND & BROADWAY

1000 BLOCK & 32ND STREET

SAN DIEGO, CALIFORNIA

BY: DTW	DATE: MAY 2019	PROJECT: 2019066	APPENDIX: D.4



<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: T-5

Depth (ft): 2.0'-3.0'

USCS Soil Type: SP-SM

Passing No. 200 (%): 8

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SAN DIEGO, CALIFORNIA

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

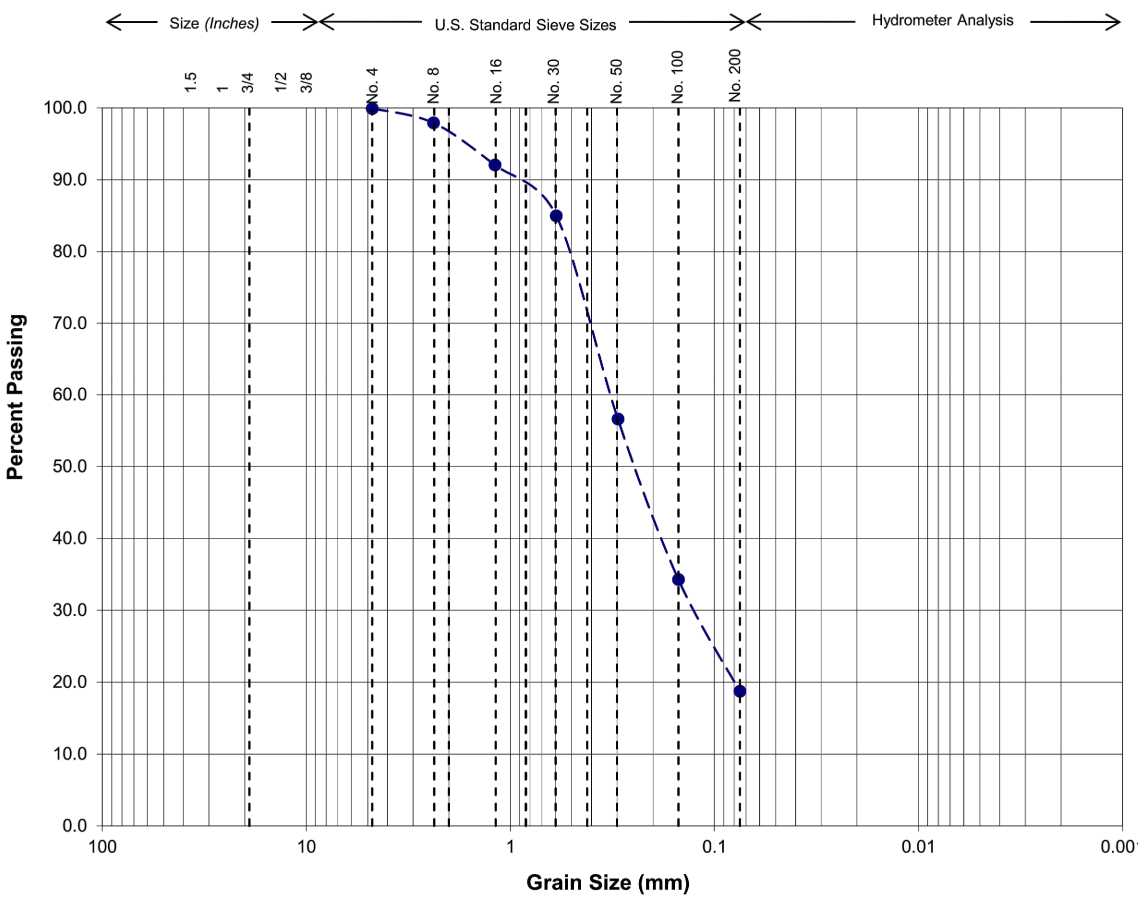
GRADATION ANALYSIS TEST RESULTS

32ND & BROADWAY

1000 BLOCK & 32ND STREET

SAN DIEGO, CALIFORNIA

BY: DTW	DATE: MAY 2019	PROJECT: 2019066	APPENDIX: D.5



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: T-6

Depth (ft): 2.0'-3.0'

USCS Soil Type: SM

Passing No. 200 (%): 19

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GRADATION ANALYSIS TEST RESULTS

32ND & BROADWAY

1000 BLOCK & 32ND STREET

SAN DIEGO, CALIFORNIA

BY: DTW	DATE: MAY 2019	PROJECT: 2019066	APPENDIX: D.6