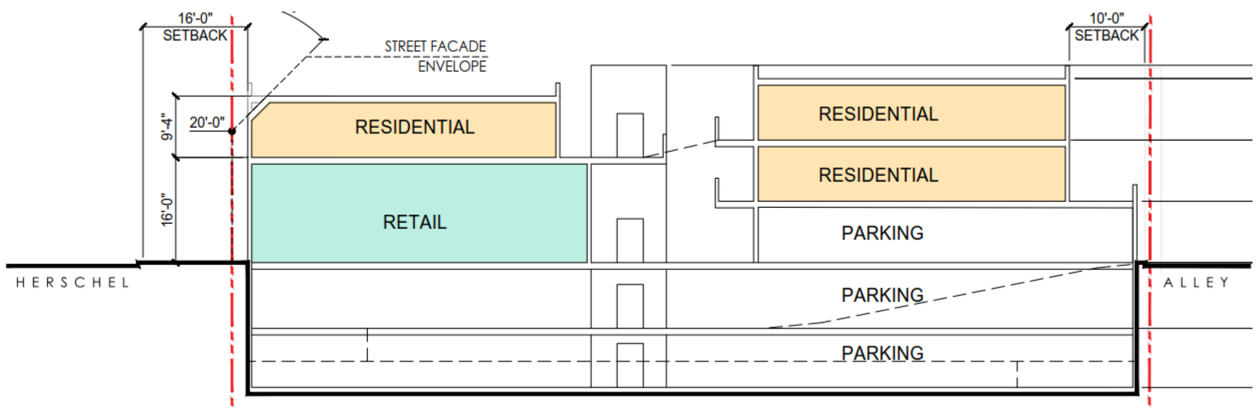
Report
Geotechnical Investigation

La Valencia Mixed-Use Building
Herschel Avenue, La Jolla, CA



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July 15, 2020

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July 15, 2020
NOVA Project 2020093

Attention: Akhil Israni, Director, Development and Acquisitions

Subject: Report
Geotechnical Investigation
La Valencia Mixed-Use Building
Herschel Avenue, La Jolla, CA

Dear Mr. Israni:

Attached hereto please find the above-referenced report. The work reported herein was completed by NOVA Services, Inc. for Pacifica Companies in accordance with NOVA's proposal dated June 2, 2020, as authorized on that date.

NOVA appreciates the opportunity to be of continued service to Pacifica Companies. Should you have any questions regarding this report or other matters, please do not hesitate to call us at 858.292.7575.

Sincerely,
NOVA Services, Inc.

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Report
Geotechnical Investigation

La Valencia Mixed-Use Building
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1.0 INTRODUCTION

1.1 Terms of Reference

This report presents the findings of a geotechnical investigation for a proposed mixed residential and commercial use building now known as 'La Valencia.' La Valencia will be located on Herschel Avenue in La Jolla at APNs 350-092-0400, -0500, and -2300 (hereinafter, 'the site').

The work reported herein was completed by NOVA Services, Inc. (NOVA) for Pacifica Companies in accordance with the scope of work detailed in NOVA's proposal dated 02 June 2020, as authorized on that date.

Figure 1-1 depicts is a vicinity map showing the location of the planned development.

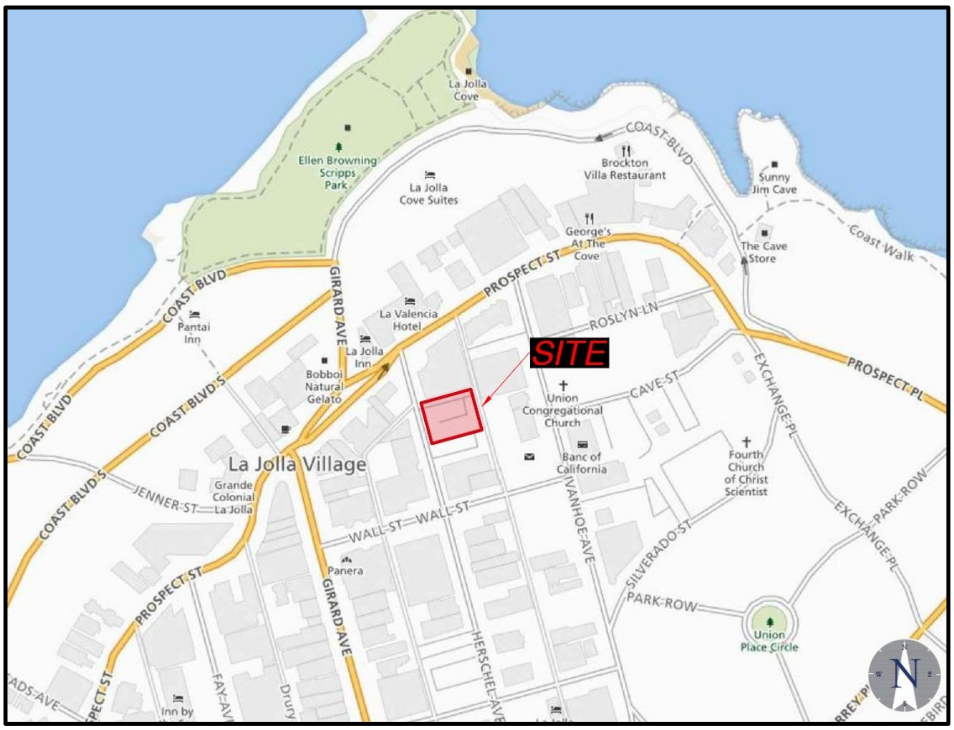

Figure 1-1. Vicinity Map, La Valencia, La Jolla, CA

1.2 Objective, Scope, and Limitations of This Work

1.2.1 Objectives

The objectives of the work reported herein are (i) to characterize the subsurface conditions within the limits of the site in a manner sufficient to develop recommendations for geotechnical-related development, including foundations, walls, temporary shoring, and earthwork; and (ii) to provide the Civil Engineer with infiltration rates for guidance in design of permanent stormwater infiltration Best Management Practices ('stormwater BMPs').

1.2.2 Scope

In order to accomplish the above objective, NOVA undertook the task-based scope of work described on the following page.

1. Task 1, Background Review. Readily available background data regarding the site area was reviewed, including geotechnical reports, topographic maps, geologic data, fault maps, etc. Architectural schematics were reviewed.

2. Task 2, Subsurface Exploration. A NOVA geologist directed a subsurface exploration comprised of the subtasks listed below.

   - *Subtask 2-1, Reconnaissance.* Prior to undertaking any exploratory work, a site reconnaissance was conducted, including layout of engineering and percolation test borings. Underground Service Alert and a utility location contractor were notified for underground utility mark-out services.

   - *Subtask 2-2, Permits and Coordination.* Specialty subcontractors were retained to conduct the drilling. Boring permits will be obtained from the County of San Diego Department of Environmental Health. NOVA coordinated with you regarding access for fieldwork.

   - *Subtask 2-3, Engineering Borings*. The geologist directed drilling of four (4) hollow stem auger borings. Soils were sampled and tested *in situ* by means of the Standard Penetration Test ('SPT', after ASTM D1586) and the California Modified sampler ('ring sampler', after ASTM D 3550).

   - *Subtask 2-4, Percolation Testing*. Two (2) hollow stem auger borings were extended to a depth of 30 feet below ground surface. Thereafter, the borings were converted to percolation test wells and percolation testing completed using procedures described in the City of San Diego BMP Design Manual, October 2018 edition.

3. Task 3, Laboratory Testing. Laboratory testing of samples recovered from the borings was completed address soil index and mechanical characteristics. Chemical testing addresses the potential that soils may be corrosive to embedded concrete or metals.

4. Task 4, Engineering Evaluations. The findings of Tasks 1-3 were utilized to support evaluations directed toward recommendations for geotechnical-related development, including foundations, earthwork, and excavation.

5. Task 5, Reporting. NOVA's scope of services will conclude with preparation of a formal written report that will provide a record of all work and geotechnical-related recommendations for foundations, walls, temporary shoring, and earthwork.

1.2.3 Limitations

Assessment of the subsurface in geological and geotechnical engineering is characterized by uncertainty. Opinions relating to environmental, geologic, and geotechnical conditions are based

on limited data, such that actual conditions may vary from those encountered at the times and locations where the data are obtained, despite the use of due professional care.
The judgments provided in this report are based upon NOVA's understanding of the planned construction, its experience with similar work, and its judgments regarding subsurface conditions indicated by the methods of subsurface exploration described in the report.

Conditions exposed by construction may vary from those disclosed by the borings. NOVA should be retained for design review and for surveillance to observe subsurface conditions revealed during construction. NOVA cannot assume responsibility for the recommendations of this report if NOVA does not perform construction observation. Section 9 of this report addresses this consideration in more detail.

This report addresses geotechnical considerations only. The report does not provide any environmental assessment or investigation of the presence or absence of hazardous or toxic materials in the soil, soil gas, 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.

1.3 Understood Use of This Report

NOVA expects that the findings and recommendations provided herein will be utilized by Pacifica Companies and its Design Team in decision-making regarding design and construction of the geotechnical related aspects of La Valencia.

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

1.4 Report Organization

The remainder of this report is organized as abstracted below.

- Section 2 reviews the presently available project information.
- Section 3 describes the subsurface investigation.
- Section 4 describes geologic and soil conditions.
- Section 5 reviews geologic, soil, and siting-related hazards common to civil works in this region, considering each for its potential to affect La Valencia during construction and its expected useful life.
- Section 6 provides recommendations for earthwork and foundation design.
- Section 7 provides recommendations for design of temporary excavations.
- Section 8 addresses stormwater infiltration for the development of dry wells.
- Section 9 provides recommendations for geotechnical observation during construction.
- Section 10 lists the principal references cited or employed in this report.

Figures and tables intended to amplify the discussions in the text are embedded therein. Plates providing large-scale presentations of certain graphics are provided immediately 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 the engineering borings.
- Appendix C provides records of laboratory testing.
- Appendix D provides documents related to stormwater infiltration.

2.0 PROJECT INFORMATION

2.1 Site Description

2.1.1 Location

The 0.4-acre site is located on Herschel Avenue in La Jolla at APNs 350-092-0400, -0500, and -2300. The site is bounded by commercial development on the north and east, Herschel Avenue on the west, and an at-grade parking lot to the south.

2.1.2 Current Site Use

The site is currently an at-grade parking lot, surfaced with asphalt. It is generally level, with surface elevations ranging from +115 feet mean sea level (msl) at the eastern property line, to about +111 feet msl at the west.

Figure 2-1 shows the location and limits of the site on a recent aerial view

Figure 2-1. Site Location and Limits

2.1.3 Historical Site Use

A review of historical aerial photos shows that the site has been used as a parking lot since at least 1953, the date of the earliest available historical imagery.

2.2 Planned Development

2.2.1 Architectural

Design is still in its preliminary stages. NOVA's understanding of current planning for development of La Valencia is based upon review of *Entitlement Draft, La Valencia, Herschel Avenue, La Jolla, CA 92037*, Joseph Wong Design Associates, Inc., September 10, 2019 (hereinafter, 'JWDA 2019').

JWDA 2019 depicts planning for a three-story mixed-use building with two levels of underground parking. The second and third stories will provide several residential units. Additionally, the at-grade level of the building will contain common areas such as a residential court and 5,152 SF of retail space. A two-level below-grade secured and enclosed parking garage will provide 83 parking spaces.

Figure 2-2 reproduces an architectural concept for the building.

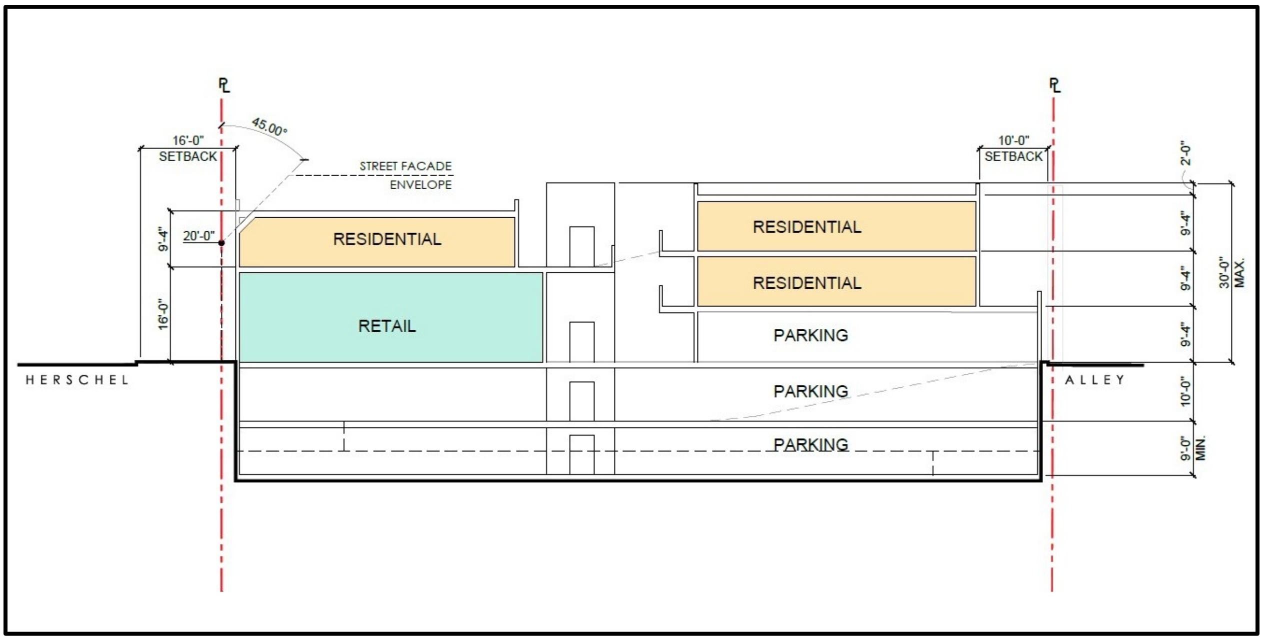

Figure 2-2. Architectural Concept
(source: JWDA 2019)

2.2.2 Structural

No structural design information is currently available. JDWA 2019 indicates that La Valencia will be developed in Type V over Type 1A construction. As may be seen by review of Figure 2-2 this planning suggests the three above-ground levels of the structure will be of Type V construction, set atop a two-level Type 1A reinforced concrete podium. The two-levels of below-grade parking will extend to about 20 feet below the surrounding ground.

Review of Figure 2-2 also suggests that design may be associated with some relatively larger column spans. Though the building includes a maximum of five-levels, the longer column spacing could drive maximum column loads (DL +LL) to about 900 kips.

Planning is currently considering an access connection with the existing retail structure to the north, connecting at the first below grade level. Structural framing concepts have not yet been developed, though it is anticipated that this design feature will be associated with higher foundation loads in the vicinity of the opening. Figure 2-3 depicts this concept.

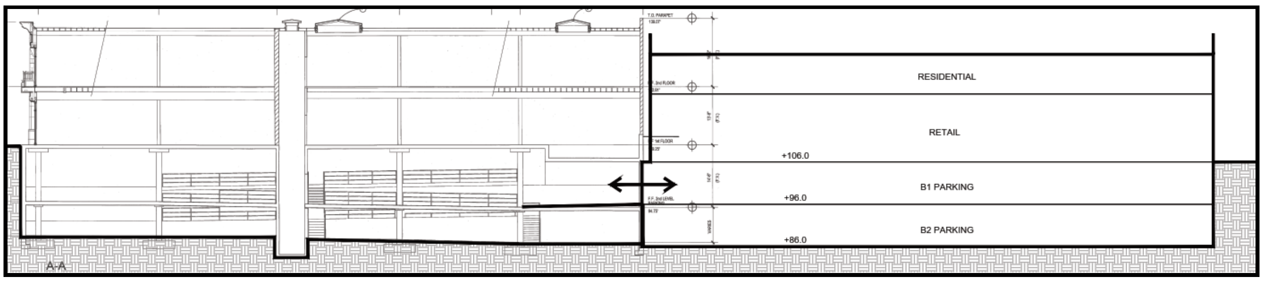

Figure 2-3. Access Connection at the First Below Grade Level
(source: JWDA 2019)

2.2.3 Civil

Development will include minimal requirements for new roadways. Current planning for permanent stormwater Best Management Practices (BMP's) structures, includes development of infiltration dry wells at the northwest corner of the structure, extending these wells to a depth of about 30 feet.

2.2.4 Potential for Earthwork

If the garage is extended two-levels below existing ground surface, it will be founded at least 20 feet in depth. Extending an excavation to about 22 feet depth across the 17,475 SF area of the site will require removal of a bank volume of about 13,600 yd^3 of soil. With an allowance for 10% swell, the Contractor could be required to remove on the order of 15,000 yd^3 of soil (about 20,300 tons hauled in perhaps 1,500 dump trucks). Note that the foregoing is not a construction estimate. Prospective earthwork contractors should make their own estimates of earthwork.

As is discussed in more detail in Section 6, the naturally occurring sandstones that underlie this site favor excavation to this level or deeper with conventional excavating equipment.

2.2.5 Proximity to Existing Structure

La Valencia will bound existing structures on its north side. This proximity is depicted graphically on Figure 2-3. Figure 2-4 (following page) depicts this existing structure in a recent photograph.

The proximity of the existing structure will limit options for external bracing of temporary shoring. Section 7 addresses design of internal bracing (rakers) for temporary shoring.

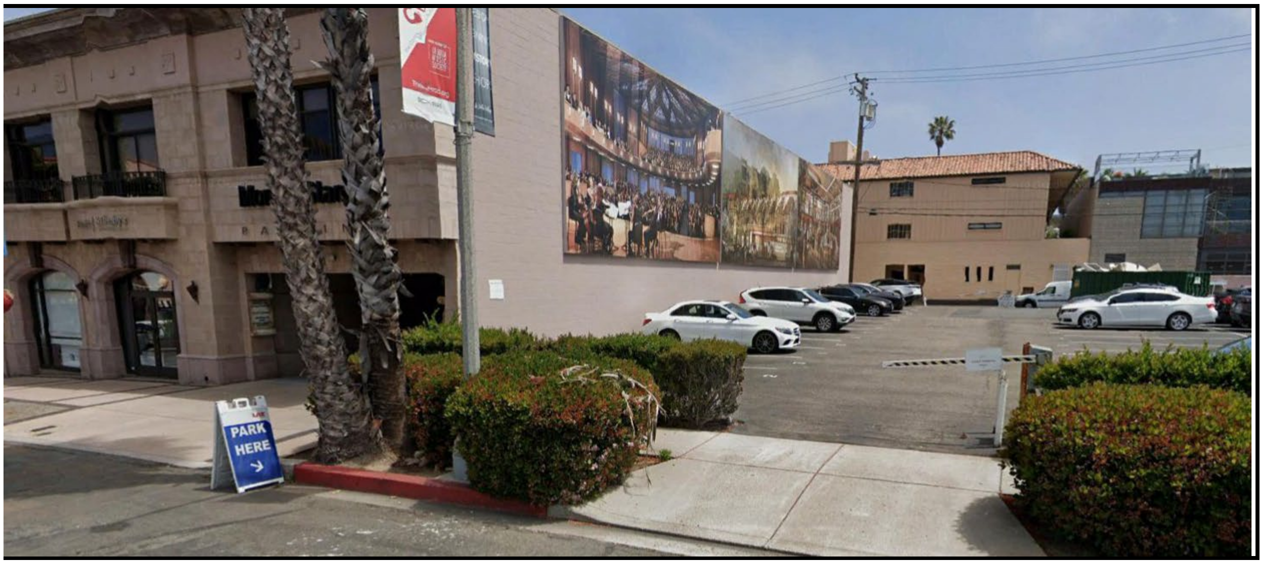

Figure 2-4. Structure of Potential Concern to the North

3.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

3.1 Overview

The subsurface exploration was completed on June 17 and 18, 2020. The work included the drilling of four (4) engineering borings ('B-1A' through 'B-3') and two percolation test borings ('P-1' and 'P-2'). Figure 3-1 provides a plan view of the site that indicates the location of the work. Plate 1, provided following the text of this report, reproduces this graphic in larger scale.

The remainder of this section describes the engineering borings, the percolation testing, and the related laboratory testing.

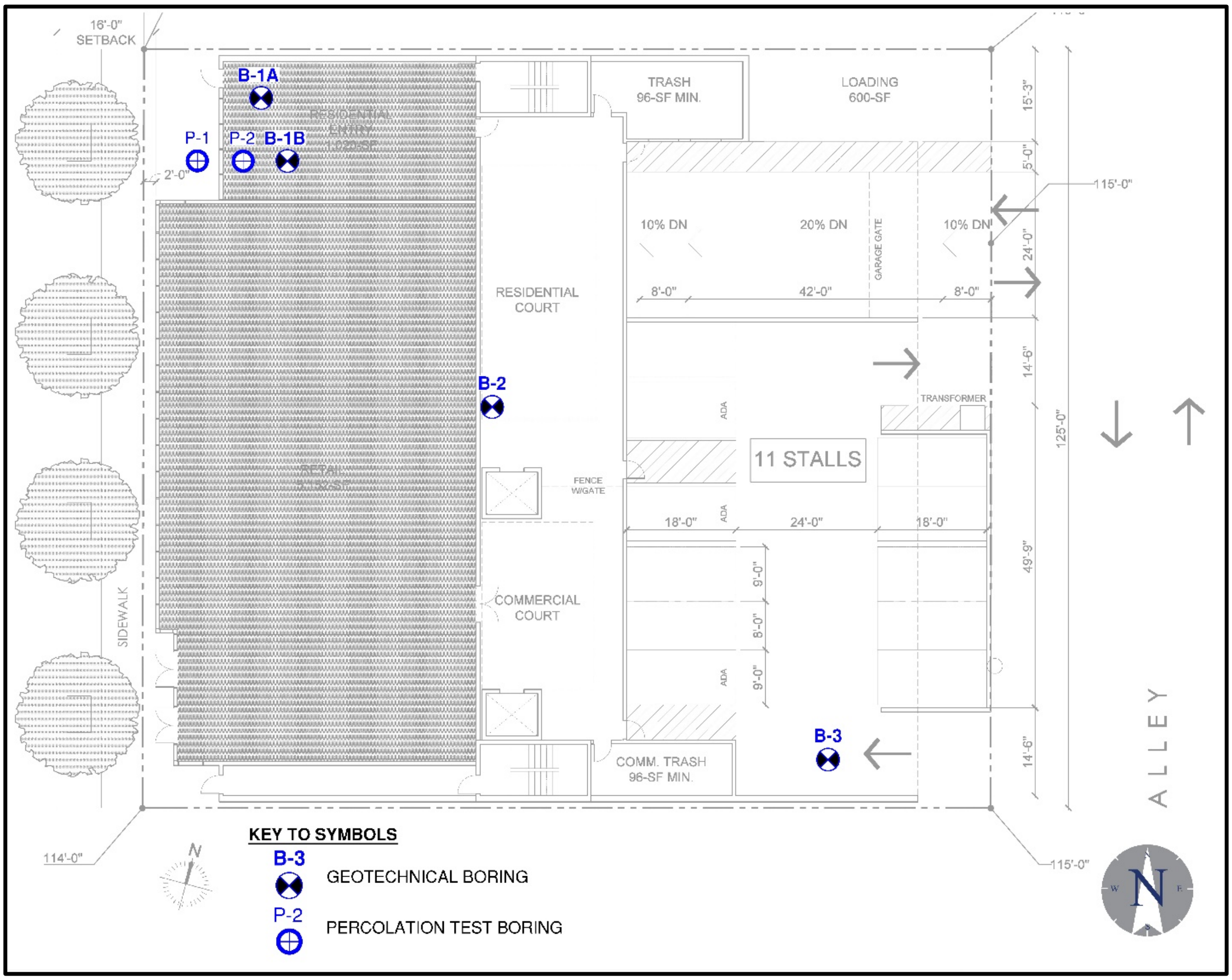

Figure 3-1. Location of Subsurface Explorations

3.2 Engineering Borings by NOVA

3.2.1 General

A NOVA geologist directed drilling and sampling of four (4) engineering borings (‘B-1A’ through ‘B-3’) to depths of up to 41 feet below ground surface (bgs) on June 17-18, 2020. Samples recovered from the borings were delivered to NOVA’s materials laboratory for analysis.

Boring B-1A was advanced to 22 feet below ground surface, when auger refusal was encountered. The drill rig was moved and Boring B-1B was advanced to 40.5 feet. In the interest of time, the top 22 feet of B-1B was not sampled and logged, as it was assumed to be represented by the sampling/logging of B-1A.

The engineering borings were advanced by a truck-mounted drilling rig utilizing hollow-stem auger drilling techniques. Boring locations were determined by the geologist based on the proposed building configuration. Table 3-1 provides an abstract of the engineering borings. Figure 3-2 (following page) depicts drilling operations on June 18.

Table 3-1. Abstract of the Engineering Borings

Boring Reference	Approx. Ground Surface Elev. (feet, msl)[1]	Total Depth Below Ground Surface (feet)	Elevation at Completion (feet, msl)[1]	Approx. Depth to Formation (feet)[2]	Approx. Depth to Groundwater (feet)
B-1A	+112	22	+90	15	Not encountered
B-1B	+112	40.5	+71.5	?[3]	Not encountered
B-2	+112	41	+71	7.5	Not encountered
B-3	+113	40.5	+73	7	Not encountered

Note 1: elevations are approximate and should be reviewed
Note 2: the referenced geologic units are Quaternary Old Paralic Deposits (Qop6) and Cretaceous Point Loma Formation (Kp)
Note 3: Upper 22 feet of B-1B was not logged, but assumed at the time to be similar to B-1A.

3.2.2 Logging and Sampling

The geologist directed sampling and maintained a log of the subsurface materials that were encountered. Both disturbed and relatively undisturbed samples were recovered from the borings. Sampling of soils is described below.

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

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

3. Bulk samples were recovered from soils in the near subsurface.

Figure 3-2. Drilling Operations, June 18, 2020

3.2.3 Closure

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

3.3 Percolation Testing

3.3.1 General

NOVA directed the excavation and construction of two (2) percolation test wells within 50 feet of the proposed infiltration dry wells following the recommendations for percolation testing presented in the City of San Diego BMP Design Manual (hereinafter, 'the BMP Manual'). The percolation test locations are shown on Figure 3-1.

3.3.2 Drilling

The borings for the test wells were each drilled with an 8-inch hollow-stem auger to depths of 30 feet bgs. Field measurements were taken to confirm that the borings were excavated to approximately 8 inches in diameter. The test wells were accompanied by an exploratory engineering boring to approximately 40.5 feet bgs, to evaluate and log the soils underlying the proposed BMP. The borings were logged by a NOVA geologist, who observed and recorded exposed soil cuttings and the boring conditions.

3.3.3 Conversion to Percolation Well

Once the borings were drilled to the desired depths, the borings were converted to percolation test wells 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. Figure 3-3 depicts the completed construction and percolation testing at well P-2.

Figure 3-3. Percolation Testing at Boring P-2

3.3.4 Percolation Testing

The percolation test wells were pre-soaked by filling the holes with water to the ground surface level and testing commenced within a 26-hour window. On the day of testing, two 25-minute trials were conducted in each well.

In both wells the pre-soak water percolated at least 6 inches into the soil unit within 25 minutes for each trial. Based on the results of the trials, water levels were recorded every 10 minutes for one hour (6 tests). At the beginning of each test interval, the water level was raised to approximately the same level as the previous tests, in order to maintain a near-constant head during all test periods.

Table 3-2 abstracts the percolation test conditions, percolation rates and calculated infiltration rates. Note that percolation rates are not the same as infiltration rates. Infiltration rates are discussed further and presented in Section 8.

Table 3-2. Abstract of the Percolation Testing

Boring Reference	Approximate Elevation (feet, msl) [1]	Total Depth (feet)	Approximate Percolation Test Elevation (feet, msl) [1]	Percolation Rate (min/in) [2]	Subsurface Unit Tested[3]	Infiltration Rate (in/hr) [2]	Infiltration Rate (in/hr, FS=2)[4]
P-1	+110	30	+80	0.83	Kp	0.82	0.41
P-2	+110	30	+80	1.41	Kp	0.42	0.21

Note 1: Elevations are approximate and should be reviewed.
Note 2: Percolation rate is not infiltration rate. Infiltration rates are discussed in detail in Section 8.
Note 3: The referenced geologic subsurface unit tested is Point Loma Formation (Kp).
Note 4: ‘FS’ indicates ‘Factor of Safety’. Discussed further in Section 8.

3.4 Geotechnical Laboratory Testing

3.4.1 General

Soil samples recovered from the engineering borings were transferred to NOVA’s geotechnical laboratory where a geotechnical engineer reviewed the soil samples and the field logs. Representative soil samples were selected and tested in NOVA’s materials laboratory to check visual classifications and to determine pertinent engineering properties.

Records of the laboratory testing are provided in Appendix C.

3.4.2 Compaction Characteristics

Two maximum dry density and moisture content tests after ASTM D1557 Method A (the ‘Modified Proctor’) were undertaken to project the moisture-density behavior of the excavated soils. Testing of a sample from B-1A from 0-4 feet shows a maximum dry density of 126 pounds per cubic foot (lb/ft^3) at an optimum moisture content of 11.2 %. Testing from the same boring from 5-10 feet shows a maximum dry density of 132.8 lb/ft^3 at an optimum moisture content of 8.2 %

3.4.3 Direct Shear

Two samples were tested in direct shear after ASTM D3080. This testing is summarized on Table 3-3.

Table 3-3. Summary of the Direct Shear Testing

Boring	Depth (feet)	Soil Description	Friction Angle	Cohesion (psf)
B-1A	5-10	Remolded silty/clayey sand fill (SM-SC)	29	192
B-2	10-11.5	Remolded silty sandstone (SM)	35	110

3.4.4 Soil Gradation

The visual classifications were further evaluated by performing grain size tests. Table 3-4 provides a summary of this testing.

Table 3-4. Abstract of the Soil Gradation Testing

Sample Ref		Percent Passing the #200 Sieve	Classification after ASTM D2488
Boring	**Depth (feet)**		
B-1A	5-6.5	33	SM-SC
B-1A	10-11.5	32	SM-SC
B-1A	15-16.5	28	SM-SC
B-1A	20-21	59	CL
B-1B	25-26	29	SC-SM

Note: ‘Passing #200’ percent by weight finer than 0.074 mm, after ASTM D 6913.

3.4.5 Plasticity and Expansion Potential

Atterberg limits testing after ASTM D4318 of a sample at B-1A from 20 to 21 feet indicated a liquid limit (LL) of LL = 32 and a plasticity index (PI) of PI = 14. This sample was also tested to determine expansion index (EI), after ASTM D4829. The sample indicated EI = 37, characteristic of a soil with Low expansion potential. This consideration is discussed in more detail in Section 5.3.

3.4.6 Chemical Testing

Resistivity, sulfate content, and chloride contents were determined to estimate the potential corrosivity of the soils. These chemical tests were performed on a representative sample of the formational sands that occur at the base of the proposed excavations by Clarkson Laboratory and Supply, Inc.

The testing shows the soil to be non-corrosive. The findings of this testing are discussed in more detail in Section 6.3. Table 3-5 abstracts the testing.

Table 3-5. Abstract of Chemical Testing

Sample Ref		pH	Resistivity (Ω-cm)	Sulfates		Chlorides	
Boring	**Depth (feet)**			**ppm**	**%**	**ppm**	**%**
B-3	25-30	9.2	2100	42	0.004	21	0.002

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 Coastal Plain in the site area is controlled by both alluvial and marine influences. This plain is underlain by near-shore marine and non-marine sedimentary rocks deposited at various intervals from the late-Mesozoic era through the Quaternary period. The Coastal Plain increases in elevation from west to east across marine terrace surfaces uplifted during Pleistocene time. Sedimentary rocks consist of conglomerate, sandstone, siltstone, and claystone that were deposited during the Cretaceous, Tertiary, and Quaternary periods.

The site is in an area that has been urbanized for decades. Fill placed to bring the site to its present groundform covers the site to depths of about 7 to 15 feet. Beneath the fill, the site is underlain by Pleistocene-aged old paralic deposits (Qop) and Cretaceous-aged sandstones of the Point Loma Formation (Kp). The old paralic deposits were not encountered in B-1A or B-1B.

The old paralic deposits extend to a depth of about 10 to 12 feet, below which occurs the Point Loma Formation (Kp). The Point Loma Formation was encountered as interbedded fine-grained, olive-gray sandstone and siltstone. These sedimentary rocks will provide foundation support for the planned structure and will be competent as a foundation material - of relatively high-strength and low compressibility.

Figure 4-1 (following page) reproduces geologic mapping of the site area.

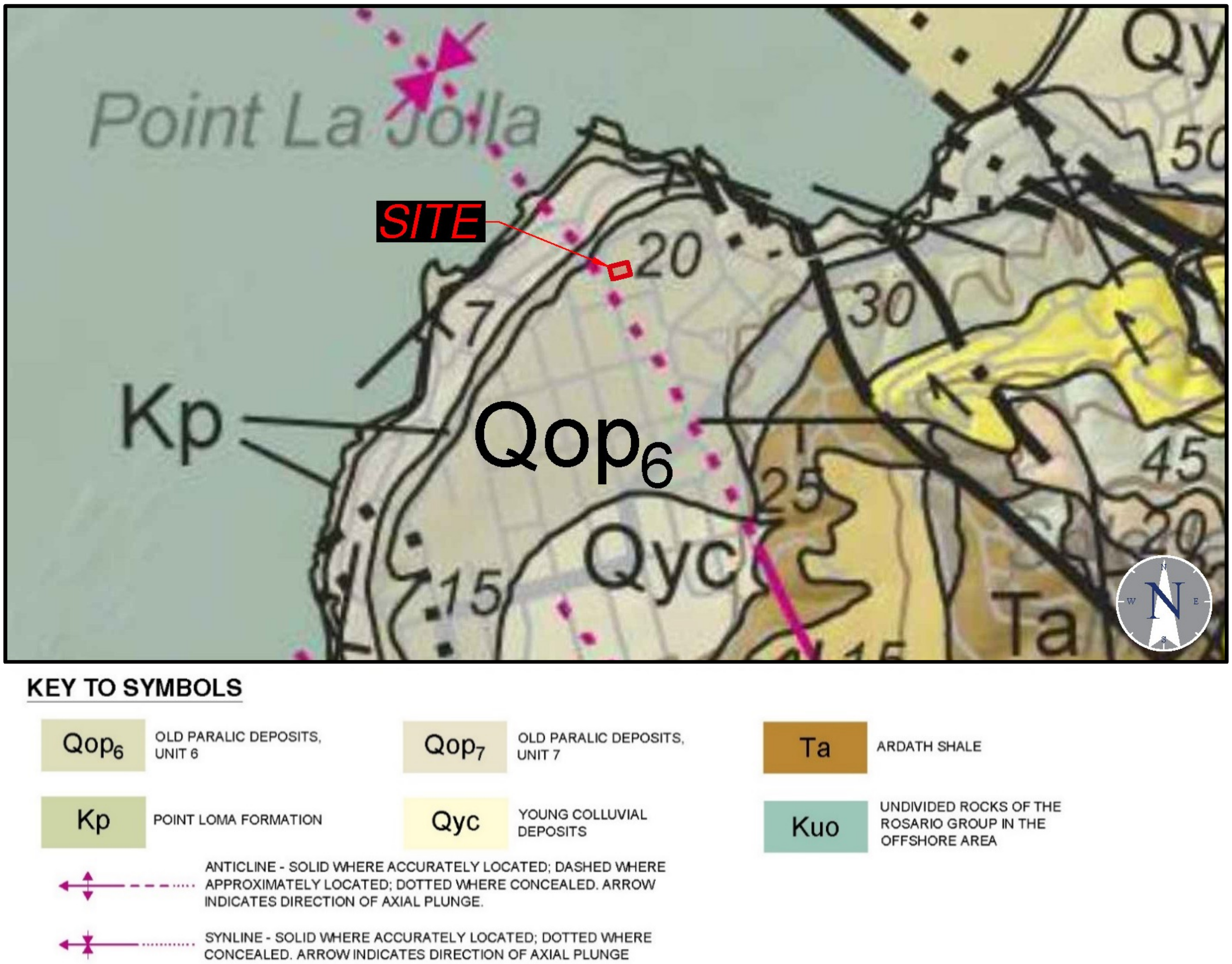

Figure 4-1. Geologic Mapping of the Site Vicinity

4.2 Surface, Subsurface and Groundwater

4.2.1 Surface Features

The site comprises about 1-acre. The site is currently an at-grade parking lot, surfaced with asphalt. It is generally level, with surface elevations ranging from +113 feet mean sea level (msl) at the eastern property line, to about +111 feet msl at the west. Figure 4-2 (following page) depicts surface conditions.

The site vicinity is well developed with commercial properties. As may be seen by review of Figure 2-4, a two commercial building adjoins the site along its northern boundary. This building has two-levels of below-grade parking. Current planning intends to connect the first level of below-grade parking at La Valencia with the first-level of below-grade parking at the existing building.

Figure 4-2. Surface Conditions, Looking Northwest from the Alley

4.2.2 Subsurface

For the purposes of this report, the sequence of soils and rock encountered by the borings may be generalized to occur as described below.

1. <u>Unit 1, Fill</u>. The site is covered by artificial fill (Qaf) that ranges from 7 to 15 feet in thickness, comprised of poorly graded ('well sorted') fine to medium sands of medium dense consistency. The fill was likely emplaced in two separate grading events. The fill encountered in Borings 2 and 3 was likely emplaced prior to 1953 (first available aerial photos of the site) to bring the project to its present grade. During construction of the subterranean parking lot to the north (between 1966 and 1980, according to photos), the northern portion of this site was likely cut, to accommodate a layback for construction of the foundations and walls of the structure. This is supported by the deep fill thickness within B-1A, compared to the depth of fill encountered in B-2 and B-3.

   All onsite fill is 'undocumented,' and at risk for wide variations in quality. Figure 4-3 depicts this fill.

Figure 4-3. Unit 1 Fill

2. Unit 2, Old Paralic Deposits. Beneath the fill, the site is underlain by Quaternary-aged old paralic deposits (Qop). The unit is characteristically of very dense consistency, composed of layers of brown to orange-brown cemented silty and clayey sandstone with some gravel. Figure 4-4 depicts this unit.

Figure 4-4. Unit 2 Old Paralic Deposits

3. Unit 3, Point Loma. Beneath the old paralic deposits, the site is underlain by Cretaceous-aged Point Loma Formation (Kp). The unit is composed of layers of sandstone and siltstone of very dense consistency. Standard Penetration Test blow counts ('N', blows/foot, after ASTM D1586) are characteristically N ≥ 100. Figure 4-5 depicts this unit.

Figure 4-5. Unit 3 Point Loma Formation

4.2.3 Groundwater

No groundwater was encountered above a depth of 41 feet below ground surface.

4.2.4 Surface Water

No surface water was evident on the site at the time of NOVA's work. There is no visual evidence of recent occurrences of surface water (e.g., staining, seeps, springs, eroded gullies, rilling erosion, etc.).

4.3 Mechanical Characteristics of the Unit 3 Point Loma Formation

The shallow foundations for the parking garage will bear on the Unit 3 Point Loma Formation. As may be seen by review of the boring logs included in Appendix B, these sandstones provide practical 'refusal' to the drive sampling device (the California Modified Sampler, after ASTM D 3550), with driving resistance greater than 100 blows/foot. Limited numbers of SPT tests (Standard Penetration Test, after ASTM D 1586) also showed refusal in this unit, with SPT blow counts ('N') of N >100 blows/foot.

Based upon the indications of the driving resistance to the SPT and California Modified sampling devices, the Mohr-Coulomb strength of the unit may be characterized by cohesion (c) and angle of friction (ϕ) as c = 300 psf and ϕ = 35°. The small strain stiffness (E) of the unit will be at least that of a dense sand/gravel mix - on the order of E > 400 tons/square foot.

4.4 Expected Bearing and Settlement

4.4.1 General

Allowable bearing capacity of shallow foundations- square or continuous footings - is lesser of:

1. <u>Criterion 1, 'Limit' Criterion</u>. Applied stress that will result in shear failure (the ultimate bearing capacity, q_{ult}) of the ground divided by a Factor of Safety (F).

or

2. <u>Criterion 2, 'Serviceability' Criterion</u>. Applied stress that results in a specified, tolerable amount of settlement of the structure.

It is exceedingly rare that the limit criterion (i.e., the shearing capacity of the soils) controls determination of allowable bearing capacity. In almost all cases, the tolerable differential and total settlement of the structure supported by the footing controls the allowable settlement. The tolerable differential and total settlement of the structure is a question best resolved by the structural engineer in consideration of the tolerable differential deformations, the time of construction, and certain serviceability constraints.

Figure 4-6 depicts the relationship between the factors that control allowable bearing capacity. The intent of the graphic is to demonstrate that allowable bearing capacity is rarely a single value and is rarely controlled by limit criterion. Settlement controls determination of allowable bearing capacity in almost all cases.

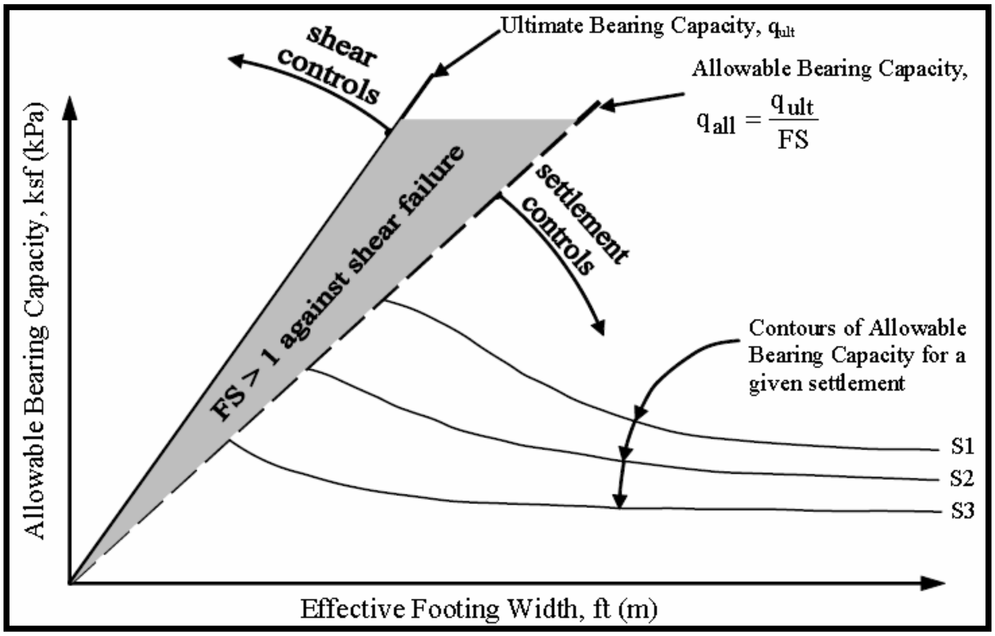

Figure 4-6. Relationships of Allowable Bearing Capacity

4.4.2 Limit Allowable Bearing Capacity

The ultimate bearing capacity of the soil (q_u) is conventionally determined by the expression:

$$q_u = c\,(N_c) + q\,(N_q) + 0.5\,(\gamma)\,(B_f)(N_\gamma)$$

where,

- c = soil cohesion
- q = surcharge at the footing base (weight of soil above the footing base
- B_f = width of the footing (in least lateral dimension)
- γ = soil unit weight
- N_c , N_q , N_γ = empirical 'bearing capacity factors' related to soil strength

The soil strength parameters of this unit may be characterized as c = 300 psf and ϕ = 35°. For a variety of footings the q_u exceeds 100,000 psf, such that the limit criterion is not of concern (i.e., FS = $q_u/q_a \geq 6$).

4.4.3 Expected Settlement

A safe bearing with respect to failure (i.e., with respect q_u) does not ensure adequate foundation performance. The safe bearing is settlement limited in virtually all cases of foundation bearing analysis.

For the purposes of this report, NOVA has completed analyses of the expected elastic settlement of foundations that would be associated with a variety of allowable bearing pressures (q_a), up to q_a ~ 8,000 psf. Figure 4-7 (on the following page) abstracts a family of curves of footing bearing stress and settlement for a variety of square (L/B = 1) footings.

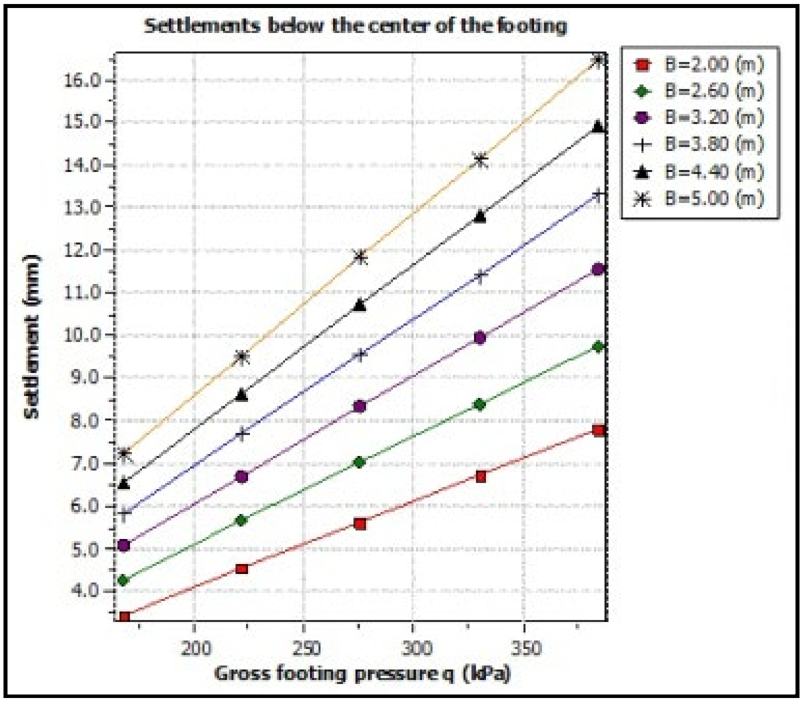

Figure 4-7. Settlement Performance of Shallow Foundations Set in Unit 3
(note the units in this graphic are metric, 384 kPa ~ 8,000 psf)

5.0 REVIEW OF GEOLOGIC, SOIL, AND SITING HAZARDS

5.1 Overview

This section provides review of geologic, soil, and siting-related hazards common to civil works in this region, considering each for its potential to affect the structure during construction and its expected useful life.

The primary hazard identified by this review are moderate-to-severe ground motions in response to large-magnitude earthquakes during its useful life. This hazard is common to all civil works in this area. Despite the expectation of a severe seismic event, there is no risk of liquefaction or secondary soil seismic phenomena (liquefaction, lateral spreading, lurching, etc.). The following subsections describe the review of soil and geologic hazards.

5.2 Geologic Hazards

5.2.1 Strong Ground Motion

The site is located in a seismically active area, as is the majority of southern California. The potential for strong ground motion is considered significant during the design life of the proposed structure. The San Diego/La Jolla tectonic setting includes north and northwest striking fault zones, the most prominent and active of which is the Newport-Inglewood Rose Canyon Fault Zone (NIRCFZ).

Fault segments within the NIRCFZ can generate an earthquake with a moment magnitude (MW) of up to MW = 6.9. The web-based tool provided by the American Society of Civil Engineers (ASCE) was used to estimate a corresponding site-adjusted peak ground acceleration (PGA_M) of PGA_M~ 0.75 g after ASCE 7-16.

5.2.2 Faulting

The site is not located within a currently designated Alquist-Priolo Earthquake Zone. No known active faults are mapped on the site. As is noted above, the site is proximate to the RCFZ less than a mile to the east, and the Coronado Bank fault zone approximately 13 miles to the west.

The closest mapped fault to the site is within the San Diego section of the RCFZ. It is a potentially active fault about 0.3 miles east of the site. The nearest active fault within the zone is approximately 0.9 miles to the east. Figure 5-1 (following page) maps the occurrence of major fault segments in the La Jolla area.

Because no known active faults are mapped on the site, the risk for damage due to fault rupture is considered very low.

5.2.3 Seismic Safety

The occurrence of a large magnitude seismic event can be associated with a variety of related phenomena, including fault rupture (discussed above), liquefaction, landsliding, etc. These risks vary with varying geologic setting. Figure 5-2 (following page) reproduces mapping by the City of San Diego of seismic risk of the site area. As may be seen by review of this graphic, the site is located in an area considered to be '*… of favorable geologic structure, low risk.*'

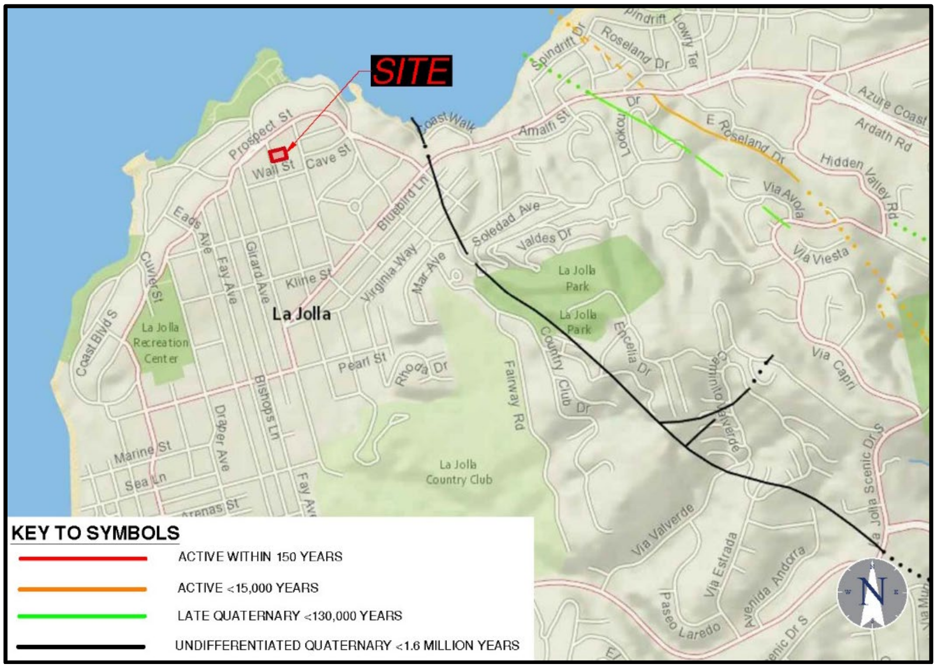

Figure 5-1. Faulting in the Site Area

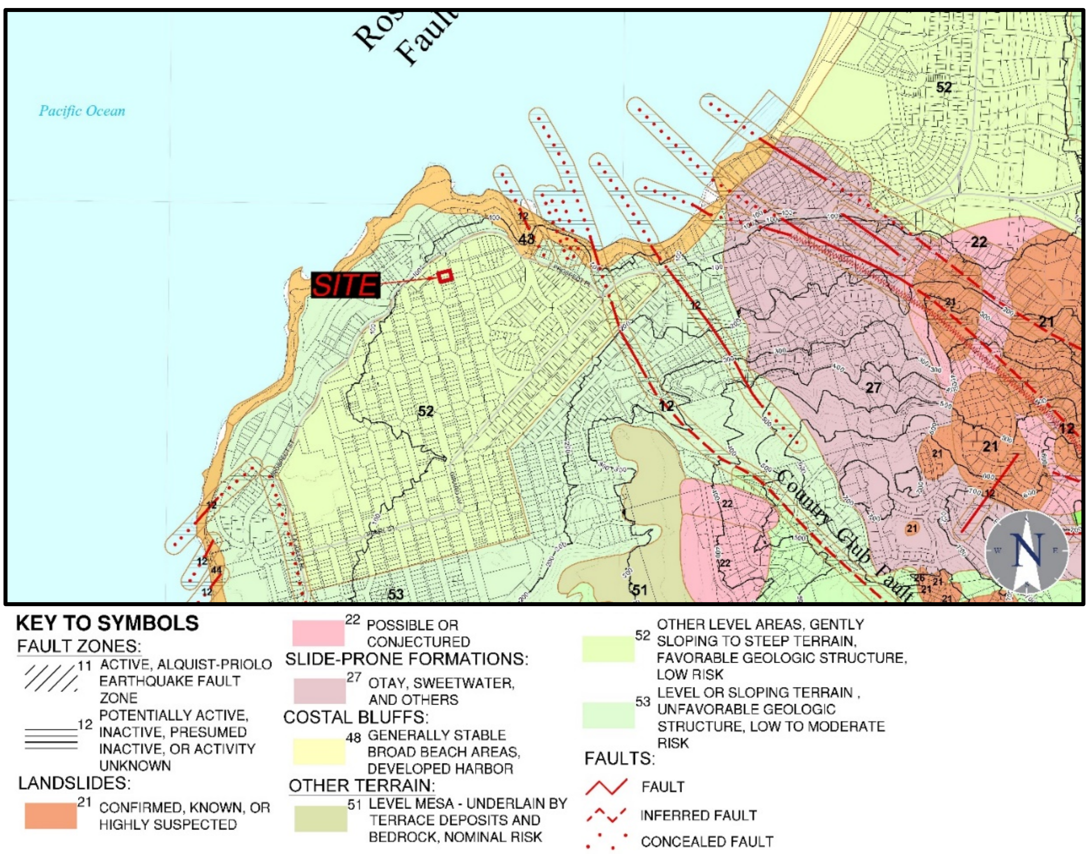

Figure 5-2. Seismic Safety Mapping of the Site Area

5.2.4 Landslide

As used herein, 'landslide' is intended to describe downslope displacement of a mass of rock, soil, and/or debris by sliding, flowing, or falling. Such mass earth movements are greater than about 10 feet thick and larger than 300 feet across. Landslides typically include cohesive block glides and disrupted slumps that are formed by translation or rotation of the slope materials along one or more slip surfaces ('failure planes'). These mass displacements can also include similarly larger-scale, but more narrowly confined modes of mass wasting such as rock topples, 'mud flows,' and 'debris flows'.

The causes of classic landslides start with a preexisting condition - characteristically, a plane of weak soil or rock - inherent within the rock or soil mass. Thereafter, movement may be precipitated by earthquakes, moisture accumulation from wet weather or irrigation, and changes to the structure or loading conditions on a slope (e.g., by new construction, erosion, cutting, filling, etc.). Rainfall is the most common trigger for landslide events. In the San Diego area, landsliding has been also been precipitated by a larger-scale earthwork, by destabilizing slopes by the cutting and/or filling on existing adverse geologic structure.

Clues to landslide hazards for an area can also be obtained by review of mapping that depicts both historic landslides and landslide-prone geology/topography. Published mapping indicates that the site is in an area judged to be 'marginally susceptible' to landsliding.

The site is set in a relatively flat area, such that NOVA considers that no landslide hazard exists for the site and the surrounding area in their current condition.

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 slope 'creep.'

No new slopes are planned as part of the proposed development. There are no embankments in the area. There is no risk of embankment instability.

5.3.2 Seismic

<u>Liquefaction</u>
"Liquefaction" refers to the loss of soil strength during a seismic event. The phenomenon is observed in geologically young. looser sandy and silty soils set in a shallow water table environment. Earthquake ground motions increase soil water pressures, decreasing grain-to-grain contact among the soil particles, causing the soil mass to lose strength. Liquefaction resistance increases with increasing soil density, plasticity (associated with clay-sized particles), geologic age, cementation, and stress history.

The cemented, 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 phenomenon is sometimes referenced as 'dynamic compaction' or seismic compaction.' The Unit 1 fill and Unit 2 paralic deposits will be removed by construction. The sandstones of Unit 3 are sufficiently dense and cemented such that these soils will not be prone to seismically induced settlement.

5.3.3 Expansive Soils

Expansive soils are characteristically clayey, able to undergo significant volume changes (shrinking or swelling) due to variations in soil moisture content (drying or wetting). These volume changes can be damaging to structures. Nationally, the value of property damage caused by expansive soils is exceeded only by that caused by termites.

Based on logging, sampling and lab testing conducted as part of this investigation, the potential for problems associated with expansivity is low.

5.3.4 Hydro-Collapsible Soils

Hydro-collapsible soils are common in geologically younger deposits 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, or wind-blown sediment). These soils are characterized by low *in situ* density, low moisture contents, sometimes high porosity, and relatively high unwetted strength.

The geomorphology that creates conditions of hydro-collapsible soils is not present at this site. Moreover, any such soils would have been removed during the original site grading.

Collapsible soils do not constitute a hazard to site development.

5.3.5 Corrosive Soils

Chemical testing of the near-surface soils indicates the soils contain low concentrations of soluble sulfates and chlorides. The testing indicates the soils should not be corrosive to embedded concrete. Resistivity testing indicates that onsite soils may be moderately corrosive to buried metals. Section 6 addresses this consideration in more detail.

5.4 Siting Hazards

5.4.1 Flood

The site is not located within a FEMA-designated flood zone, designated as Flood "Zone X" (FEMA, Map 06073C1582H, effective 12/20/2019). Zone X describes "*Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.*"

Figure 5-3 (following page) reproduces the portion of FEMA Map that includes the site.

Figure 5-3. Portion of the Flood Mapping That Includes the Site
(source: FEMA 2019, found at https://msc.fema.gov/portal/)

5.4.2 Inundation

Surface Water
The site is not located near a dam, water storage tank, levee or related structure, the failure of which could cause widespread inundation.

Tsunami
Tsunami ('tidal wave') describes a series of fast-moving, long-period ocean waves caused by earthquakes or volcanic eruptions. The elevation of the site from the ocean precludes this threat.

Seiche
Seiches are standing wind or seismic-driven waves that develop in an enclosed or partially enclosed bodies of water such as larger lakes, reservoirs, or embayments. 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, geologic, and siting 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. This circumstance is common to all civil works in this area of California. While strong ground motion could affect the site, there is no risk of liquefaction or related seismic phenomena (fault rupture, liquefaction, seismic settlement, ground lurching, etc.).

Section 6.2 provides structural design parameters that address the seismic hazard.

6.1.2 Site Suitability

Based upon the indications of the field and laboratory data developed for this investigation, it is the judgment of NOVA that the site is suitable for development of the planned structure on shallow foundations, provided the geotechnical recommendations described herein are followed.

Development of the structure 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

NOVA should review the grading plan, foundation plan, and geotechnical-related specifications as they become available to confirm that the recommendations presented in this report have been incorporated into the plans prepared for the project. All earthwork related to site and foundation preparation should be completed under the observation of NOVA. Section 9 addresses this consideration in more detail.

6.2 Seismic Design Parameters

6.2.1 Site Class

Though soil borings extend to about 40 feet below existing ground surface, the subsurface in this area is well understood to great depths. The Point Loma Formation is reported to be over 300 meters in total thickness (Kennedy, 1975). SPT blow counts average N> 100 blows/foot within the borings conducted for this investigation.

A series of dense sandstones, siltstones, and related sedimentary rock are known to extend to at least 200 feet below existing ground surface, such that the subsurface may be classified as Site Class C per ASCE 7-16 (Table 20.3-1).

6.2.2 Seismic Design Parameters

Table 6-1 presents the seismic design parameters for Site Class C.

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

Parameter	Value
Site Class	C
Site Latitude (decimal degrees)	32.84782
Site Longitude (decimal degrees)	-117.27315
Risk Factored Peak Ground Acceleration (PGA_M)	0.75
Mapped Short Period Spectral Acceleration, S_S	1.364
Mapped One-Second Period Spectral Acceleration, S_1	0.478
Short Period Spectral Acceleration Adjusted For Site Class, S_{MS}	1.637
One-Second Period Spectral Acceleration Adjusted For Site, S_{M1}	0.717
Design Short Period Spectral Acceleration, S_{DS}	1.091
Design One-Second Period Spectral Acceleration, S_{D1}	0.478

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. Concentrations of water-soluble sulfates are indexed to the potential for sulfate attack to embedded concrete.

A representative sample of the of the formational sands that occur at the base of the proposed excavations was tested for these parameters. Appendix C provides a complete record of this testing. The results of the testing are abstracted in Table 6-2.

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

Sample Ref		pH	Resistivity (Ω-cm)	Sulfates		Chlorides	
Boring	Depth (feet)			ppm	%	ppm	%
B-3	25-30	9.2	2100	42	0.004	21	0.002

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 criteria established by Caltrans, the on-site soils would not be considered ‘corrosive’ to buried metals. Appendix C provides a record of the chemical testing that includes estimates of the life expectancy of unprotected 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 moderately corrosive to embedded ferrous metals. Ferrous metals include mild steel, carbon steel, stainless steel, cast iron, and wrought iron.

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 Sulfates

As shown in Table 6-2, the soil sample tested indicated water-soluble sulfate (SO_4) content of 42 parts per million (‘ppm,’ about 0.004% by weight). With $SO_4 < 0.10$ percent by weight, the American Concrete Institute (ACI) 318-08 considers a soil to have no potential (S0) for sulfate attack.

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

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

Exposure Category	Class	Water-Soluble Sulfate (SO_4) In Soil	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

6.4 Earthwork

6.4.1 General

As is noted in Section 2, no structural related design information is available at this time. However, based on the known condition of the site and the design concept that is currently considered, earthwork will be considerable in excavations for two garage levels, plus earthwork for foundations and utilities. As is discussed in Section 2, the Contractor could be required to remove on the order of 15,000 yd^3 of soil (about 20,300 tons hauled in perhaps 1,500 dump trucks).

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

6.4.2 Site Preparation

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. Any existing utilities which are to be abandoned should either be (i) excavated and the trenches backfilled, or (ii) the lines completely filled with sand-cement slurry.

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

6.4.3 Excavation Characteristics

The Unit 1 fill, Unit 2 paralic deposits and Unit 3 Point Loma sandstones will be readily excavated by earthwork equipment usual for construction of this nature.

6.4.4 Select Fill

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

- at least 40% by weight finer than ¼ inches in size,
- maximum particle size of 4 inches, and
- expansion index (EI) of less than 30 (i.e., EI < 30, after ASTM D 4829).

The Unit 1 fill now in place will conform to the above criteria. Select Fill may also be generated from the excavations in the underlying sandy portions of Unit 2 paralic deposits, though this unit may contain some gravel and cobbles that will require screening to meet the Select Fill criteria.

Select Fill may also be imported. Any imported soils will need to be sampled and tested by NOVA to confirm suitability as Select Fill prior to use.

Compaction

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

The sandy Select Fill should be densified by purpose-designed vibratory compaction equipment, placed in loose lifts no thicker than the ability of the compaction equipment to thoroughly densify the lift. For most self-propelled compaction equipment adaptable to this site, this criterion will limit loose lifts to on the order of 8 inches or less. Lift thickness for hand-operated equipment (tampers, walked behind compactors, etc.) will be limited to on the order of 4 inches or less.

6.4.5 Foundation and Subgrade Preparation

The Unit 1 fill will be removed in its entirety by planned excavations for the below-grade construction, and expose competent formational soils. Care should be taken to not undermine or destabilize off-site structures or pavements. The bottom of all excavations should be approved by NOVA.

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 placement.

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, overlying slabs, and pavements.

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% relative compaction after ASTM D 1557 (the 'modified Proctor') at soil moisture +2% 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% relative compaction with respect to the Modified Proctor.

6.5 Shallow Foundations

6.5.1 General

Excavation for the below-grade levels of the structure will expose competent soils of the Unit 3 Point Loma Formation. As is discussed in more detail in Section 4.5, this unit is well adapted to provide support for shallow foundations at relatively higher bearing loads.

The following subsections provide design criteria for foundations supported on the Unit 3 sandstones.

6.5.2 Footings

Shallow foundations - either isolated or continuous footings - may be established on the Unit 3 Point Loma formation and designed in accordance with the parameters listed below.

Minimum Dimensions and Reinforcing
Continuous footings should be at least 20 inches wide and have a minimum embedment of 24 inches below lowest adjacent grade.

Isolated square or rectangular footings should be a minimum of 30 inches wide, embedded at least 24 inches below finish pad grade.

All foundation elements, including any grade beams, should be reinforced top and bottom. The actual reinforcement should be designed by the Structural Engineer.

Contact Stress
Continuous and isolated footings constructed as described above may be designed using an allowable (net) contact stress of 8,000 pounds per square foot (psf) to combined dead and sustained live loads (DL + LL). The allowable bearing pressure may be increased by ⅓ when considering transient loads, including seismic and wind.

The bearing surface of footings adjacent to utility trenches should be either embedded or set back such that the utility trench is outside of an imaginary 1.5H: to 1V plane projected upward from the base of the utility trench.

Lateral Resistance.
Resistance to lateral loads will be provided by a combination of (i) interface friction between Unit 3 sandstones and the foundation base, and (ii) passive pressure acting against the vertical portion of the footings. A frictional coefficient of 0.35 may be used. Passive pressure may be calculated at 350 psf per foot of depth. No reduction is necessary when combining frictional and passive resistance.

Settlement.
Shallow foundations designed as recommended above will settle on the order of 0.5 inches or less. Foundation settlement will be entirely elastic, with about 70% of this settlement occurring during the construction period. Angular distortion (Δ/L) due to differential settlement between adjacent unevenly loaded areas will be on the order of ½-inch over a horizontal distance of 40 feet.

6.5.3 Ground Supported Slab

A conventionally reinforced on-grade concrete slab may be supported on the Unit 3 Point Loma Formation. Founded as such, the concrete slab may be designed using a modulus of subgrade reaction of 200 pounds per cubic inch (200 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.

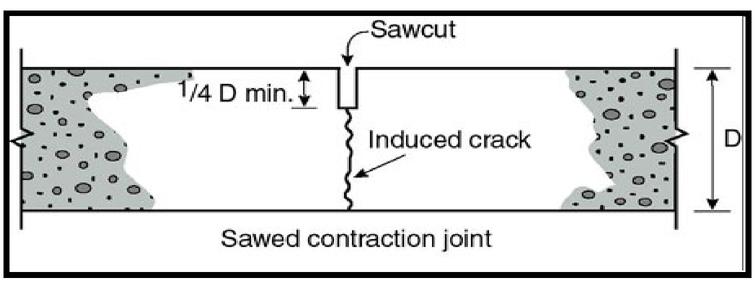

Figure 6-1. Sawed Contraction Joint

6.6 Capillary Break and Underslab Vapor Retarder

6.6.1 Capillary Break

If employed beneath the ground supported slab, the requirements for a capillary break (‘sand layer’) should be determined in accordance with ACI Publication 302 “*Guide for Concrete Floor and Slab Construction*.” A capillary break may consist of a 4-inch thick layer of compacted, well-graded sand should be placed below the floor slab. This porous fill should be clean coarse sand or sound, durable gravel with not more than 5% coarser than the 1-inch sieve or more than 10% 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 floors, some floor sealers, or sensitive equipment in direct contact with the floor. It is not the responsibility of the geotechnical consultant to provide recommendations for vapor retarders to address this concern. This responsibility usually falls to the Architect. Decisions regarding the appropriate vapor retarder are principally driven by the nature of the building space above the slab, floor coverings, anticipated penetrations, concerns for mold or soil gas, and a variety of other environmental, aesthetic, and materials factors known only to the Architect.

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 area 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 Walls

6.7.1 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. Table 6-5 provides recommendations for lateral soil wall loading to below-grade walls with level backfill for varying conditions of wall yield.

Table 6-5. Lateral Earth Pressures to Below Grade Walls

Condition	Equivalent Fluid Pressure (psf/foot) for Approved Backfill Notes A, B
Active	35
At Rest	55
Passive	350

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

It is expected that the garage walls will be unyielding, designed to resist 'at rest' soil loads. 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. To account for such potential loads, a surcharge pressure of 75 psf can be applied uniformly over the wall to a depth of about 10 feet.

6.7.2 Seismic Increment to Non-Yielding Garage Walls

The lateral seismic thrust acting on a non-yielding garage walls may be estimated by the dynamic (seismic) thrust, $\Delta P_{E.}$ Dynamic thrust is approximated as:

$\Delta P_E = k_h H^2 \gamma$ where,
k_h , pseudostatic horizontal earthquake coefficient, equal to $S_{DS}/2.5$
H is the height of the wall in feet from the footing to the point of fixity
γ is equal to the unit weight of the backfill material, in pcf (about 120 pcf)

The resultant dynamic thrust acts at a distance of 0.6H above the base of the wall.

6.7.3 Drainage

Design for permanent walls should include drainage to limit accumulation of water behind the wall. Figure 6-2 (following page) provides guidance for such design. Note that the guidance provided on Figure 6-2 is conceptual. A variety of options are available to drain permanent below-grade walls.

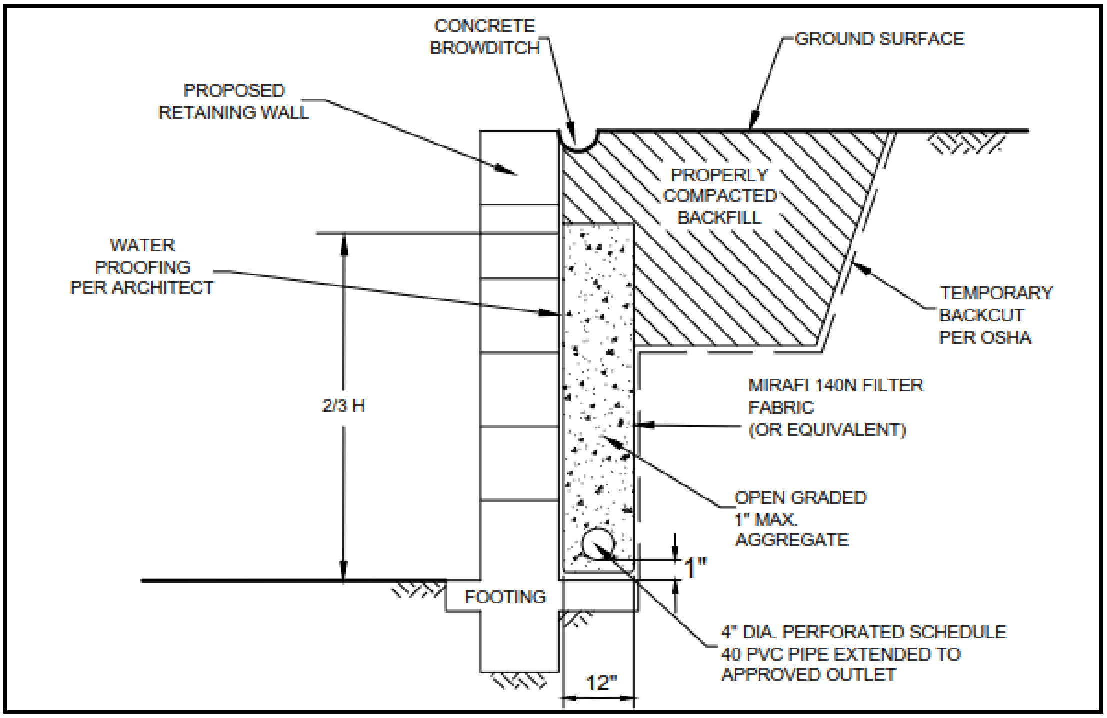

Figure 6-2. Conceptual Design for Wall Drainage

6.8 Elevator Pits

As is depicted on Figure 2-3, an elevator may extend to the parking garage and may require a pit that extend below the lowest garage slab level. An elevator pit slab and related retaining wall footings will derive suitable support from the Unit 3 Point Loma Formation around it. Design for the elevator pit walls should consider the circumstances and conditions described below.

1. Wall Yield. NOVA expects that proper function of the elevator pit should not allow yielding of the elevator pit walls. As such, walls should be designed to resist 'at rest' lateral soil pressures and seismic pressures provided above, also allowing for any structural surcharge.

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

6.9 Flatwork

In areas to support at-grade flatwork, the upper one foot of the Unit 1 soil should be removed and be replaced as compacted fill. The bottom of removals should be scarified to 12 inches, moisture-conditioned, and compacted to at least 90% relative compaction. The bottom of all excavations should be approved by NOVA prior to replacing any of the excavated soils. Removed soils that meet the criteria of "Select Fill" can be replaced as properly compacted Select Fill per Section 6.4.4.

Exterior concrete slabs for pedestrian traffic or landscape should be at least 4 inches thick. Weakened plane joints should be located at intervals of about 6 feet. Control of the water/cement ratio can limit shrinkage cracking due to excess water or poor concrete finishing or curing. Typical reinforcement for exterior slabs may be reinforced with No. 3 bars on 18-inches centers, each way.

6.10 Temporary Slopes

6.10.1 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 the Unit 2 paralics and Unit 3 Point Loma Formation may assume California Occupational Safety and Health Administration (Cal/OSHA) Soil Type B for planning purposes. The Unit 1 fill may be assumed to be Type C.

6.10.2 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 paralic deposits and Unit 3 Point Loma Formation 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. 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.

7.0 TEMPORARY SHORING

7.1 General

7.1.1 Responsibilities

It is the responsibility of the Contractor to provide an excavation that is safe, with deflections appropriate to the site setting. Design of temporary shoring should be performed by a qualified Shoring Engineer. The Shoring Engineer should be solely responsible for the design, utilizing the indications of subsurface conditions provided in the geotechnical reporting.

7.1.2 Scope of this Section

As is discussed in Section 2, planning for the building is very preliminary. Presently, design contemplates a two-level below-grade garage that would require excavation to about 22 feet depth below the surrounding ground.

Development of the garage will require temporary shoring employing one-level of external or internal bracing.

Regardless of the scope/mode of temporary shoring, it is likely that a ‘soldier beam and lagging’ system will be preferred on a basis of expected cost and performance. This system dominates temporary wall construction in the San Diego area. The following subsections provide guidance for the Owner, the Design Team and the Shoring Engineer in development of design for temporary shoring utilizing.

Note that installation of the soldier beams will likely require drilling (rather than driving) near the existing structure at the north end of the site in order to limit ground vibrations.

7.2 Design Conditions for A Temporary Wall

7.2.1 General

The Owner and the Design Team should consider that design for a braced/retained excavation may address two broad conditions of wall loading as described below.

1. Condition 1, ‘At Rest.’ Design for the retaining wall should consider the use of ‘at-rest’ soil pressures at locations where wall deflections may affect potentially damaging settlement to utilities or structures. Of common potential concern in this regard is the structure bounding the excavation on the north.

2. Condition 2, ‘Active.’ Design for the walls that are not located near sensitive structures or utilities should consider design to resist ‘active’ earth pressures. Based on review of the site area, it appears that this condition may be more appropriate for the walls for the planned development.

7.2.2 Condition 1, ‘At Rest’

Design to resist the Condition 1 ‘at rest’ (i.e., ‘K_o’) earth pressures employs a rectangular wall pressure distribution that is more conservative than the Condition 2 loading. Figure 7-1 provides this pressure distribution, reproducing published guidance of relevance to this design circumstance.

Design for the 'at rest' wall pressure diagram depicted in Figure 7-1 using the parameters would yield:

P (psf) = 0.45 (K_o) (γ) (H) where,
$K_o = 1 - \sin\phi$ $\phi = 32°$, and $K_o = (1 - 0.53) = 0.47$
$\gamma = 125$ lb/ft^3
H = wall height

$P = 0.45 \times 0.47 \times 120 \times H = 25H$

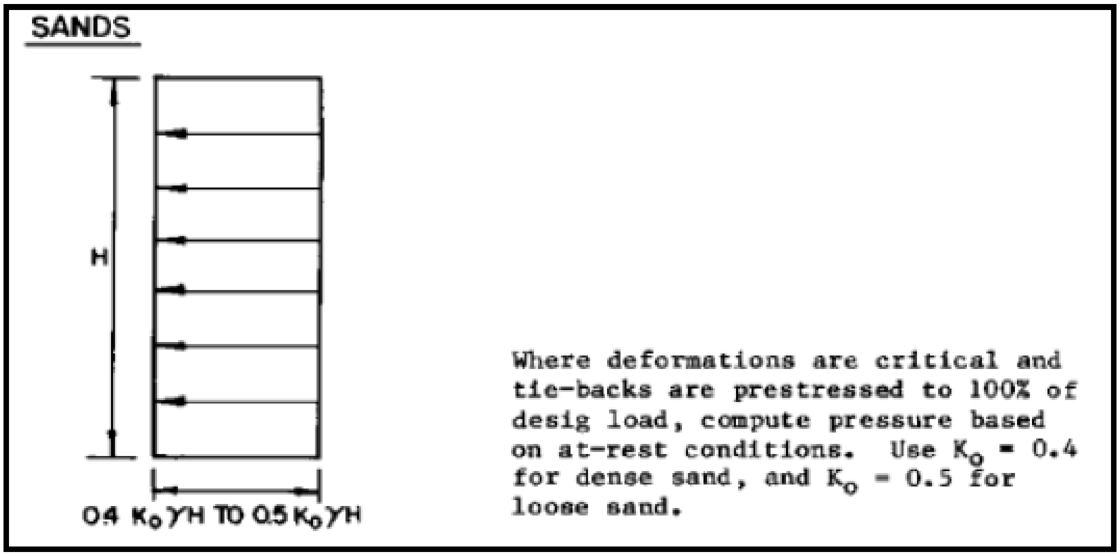

Figure 7-1. 'At Rest' Wall Pressure Distribution in Sands
(source: NAVFAC 1986)

The Shoring Engineer should also consider additional lateral pressure due to the surcharging effects of adjacent structures or traffic loads should be considered by the Shoring Engineer, as appropriate. These loads will act as a surcharge to the temporary wall.

7.2.3 Condition 2, 'Active'

Based on review of aerial photography of the area, it is the judgment of NOVA that the site is favorable for design for less conservative wall pressures than those driven by Condition 1. That is, there is no indication that the project bounds an area where wall deflections will immediately threaten structures or utilities.

With the above perspective, NOVA recommends that wall design using active earth pressures should be computed as described by the trapezoidal active earth pressure distribution of Figure 7-2(a) (following page). The magnitude of the maximum trapezoidal pressure may be calculated as:

P (psf) = 0.65 (K_a) (γ) (H) where,

$K_a = (1 - \sin \phi) / (1 + \sin \phi)$, $\phi = 32°$, $K_a = 0.30$
$\gamma = 120$ lb/ft^3
H = wall height

For a variety of assumptions regarding γ, ϕ, and K_a, the maximum magnitude of lateral pressure for a tied back soldier beam and lagging wall system will normalize to be in the range 20H to 23H, where 'H' is the height of the wall in feet.

NOVA recommends employing the trapezoidal distribution of Figure 7-2(a), using 22H for determination of the maximum wall pressure.

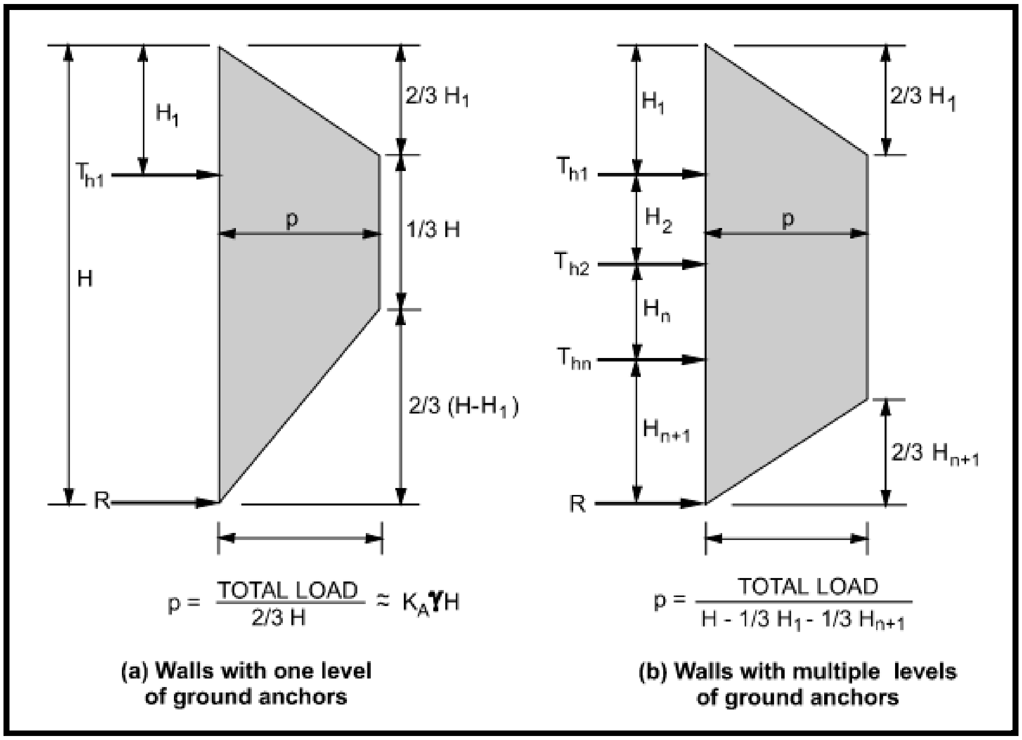

Figure 7-2. 'Active' Lateral Earth Pressures
(source: FHWA 1999)

It should be noted that the pressure distribution of Figure 7-2(a) are empirical, derived from experience. The recommendation for this pressure distribution follows guidance provided by FHWA 1999.

It should be understood that other empirical pressure distributions may be preferred by others. However, it is NOVA's experience that the pressure distribution of Figure 7-2 works well to predict wall loads/anchor loads in this area.

7.2.4 Passive Resistance to Soldier Piles

It is expected that soldier beams will be set in pre-drilled holes and backfilled with lean concrete or a sand-cement slurry with a compressive strength of at least 700 psf. Passive resistance to embedment of a temporary wall in Unit 2 & Unit 3 formational soils may be calculated using an 'equivalent fluid wall pressure' distribution, where the maximum equivalent fluid pressure (P) may be calculated as:

P (psf) = $(K_p)\ (\gamma)\ (D)$ where,

$K_p = (1 + \sin \phi) / (1 - \sin \phi)$ $\phi = 35°$, $K_p = 3.6$

$\gamma = 130\ lb/ft^3$

D = depth of wall embedment

$P = 3.6 \times 130 \times D = 470\ D$

7.3 Tieback Anchors

7.3.1 Failure Wedge

Design should assume that the failure wedge adjacent to the shoring is defined by a plane drawn at 29° from the vertical from the toe of the wall. Figure 7-3 depicts this wedge graphically.

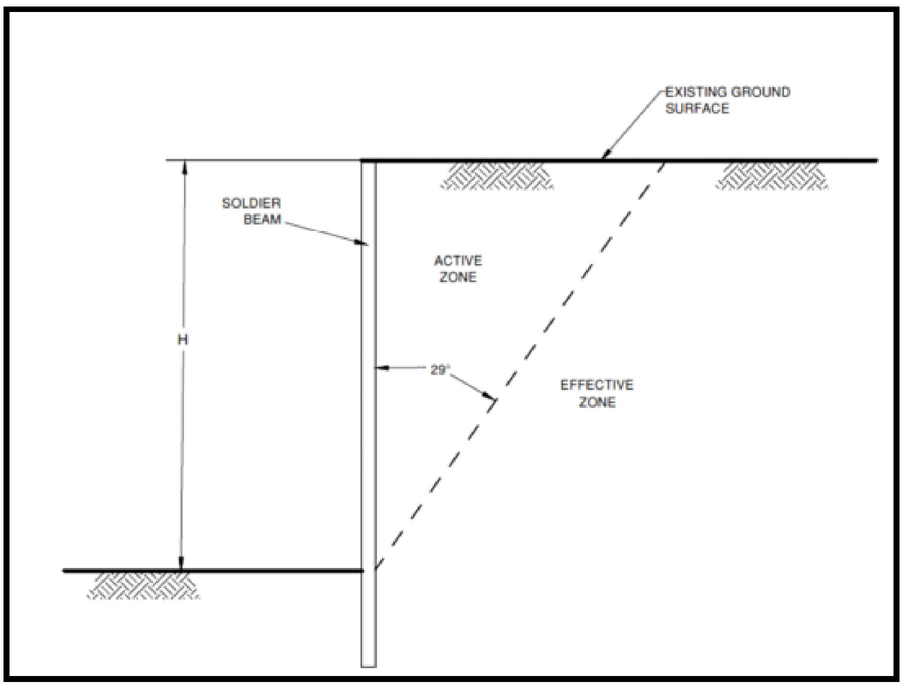

Figure 7-3. Recommended Effective Zone for Tieback Anchors

Tieback anchors should extend at least 20 feet beyond the failure wedge (i.e., the "bonded" zone) depicted in Figure 7-3. The intent of this provision is to provide global stability for the shored wall. The bonded length should commence at least 5 feet beyond the failure wedge.

7.3.2 Anchor Installation

The anchors may be installed at angles of 15° to 35° below the horizontal. The anchors should be filled with concrete placed by pumping from the tip of the anchor to the failure wedge (i.e., over the bonded zone). The portion of the anchor tendons outside of the bonded length should be sleeved in plastic (i.e., over the unbonded zone). If anchor tendons are sleeved, the entire length of the anchor may be concreted.

7.3.3 Bond Stress

The Shoring Engineer should be solely responsible for determination of allowable bond stresses to pressure-concreted ('post-grouted') anchors. NOVA expects that an allowable bond stress of 5,000 psf will be achievable. Only the resistance developed beyond the failure wedge should be used in resisting lateral loads. If the anchors are spaced at least 6 feet on center, no reduction in the capacity of the anchors need be considered due to group action. In no event should the anchors extend less than the minimum length beyond the potential failure wedge as given above.

As a tie-back anchor system is intended for temporary use, provisions should be made in the design to de-tension and abandon the tie-backs when the basement walls are able to support the lateral loads.

7.3.4 Performance Testing

Wall design should provide for (i) performance testing, (ii) proof testing, and (iii) creep testing of wall anchors. In this regard, it is recommended that guidance provided in FHWA 1999 be utilized. Guidance for proof testing for all anchors provides for loading to a single cycle and load hold at the test load. The guidance provides that loading be applied pre-provided in load increments of 0.25DL, 0.50DL, 1.00DL, 1.20DL and 1.33DL (the 'test load').

All of the production anchors should be tested to at least 130% of the design load; the total deflection during the tests should not exceed 1.5 inches. The rate of creep under the 130% test should not exceed 0.1-inch over a 15-minute period for the anchor to be approved for the design loading.

7.4 Rakers

The north face of the excavation near the existing two-story building may require internal bracing. Similarly, the excavation along the alley on the eastside of the site may have limited room for external bracing.

If rakers (inclined struts) are employed for internal bracing of the excavation, these units will gain lateral resistance from either (i) temporary foundations or (ii) the central part of the basement level slab. In the latter case, the excavation would first be carried in full depth at its center, so that the basement level slab could be placed. Thereafter, the slab could provide resistance to rakers loads.

If temporary foundations are utilized to support the rakers inclined at 40° or steeper, mass concrete heel blocks, embedded a minimum of 3 feet below surrounding grade will provide ultimate passive resistance of 700 psf over the face of the heel block. Alternatively, a steel section may be embedded in a predrilled hole to provide lateral resistance similar to that described above for soldier piles.

7.5 Miscellaneous Wall Design Considerations

End bearing for soldier piles will be negligible and should not be considered. As noted previously, it is expected that soldier beams will be set in pre-drilled holes and backfilled with lean concrete or a sand-cement slurry with a compressive strength of at least 700 psf. The soil-pile bond will be on the order of 400 psf or greater. The coefficient of friction (μ) between the wall and surrounding soils is μ = 0.35.

7.6 Expected Wall Movements

7.6.1 Reliance on Construction Quality

Actual wall movement and related ground settlement are related to a variety of factors, most significantly (i) subsurface conditions, including effective dewatering; and, (ii) workmanship in wall construction.

The dense sandy formational soil is favorable for sound wall construction. NOVA will coordinate with the Shoring Engineer to ensure that good workmanship prevails throughout wall construction. The combination of workmanship and favorable subsurface conditions will result in good wall performance. Additionally, ground and wall movement monitoring described in the following subsections should be sufficient to detect any unusual wall movement before the condition becomes problematic.

The following subsections discuss expected settlements for the Condition 1 and Condition 2.

7.6.2 Condition 1

Design for Condition 1 wall will limit wall movement, though the 'at rest' wall condition will not eliminate all wall movement. Because of the reliance on a wide variety of parameters, including workmanship, it is difficult to rigorously predict wall performance during the design stage. Expectations in this regard are primarily empirical. Figure 7-4 provides a published summary of project experiences.

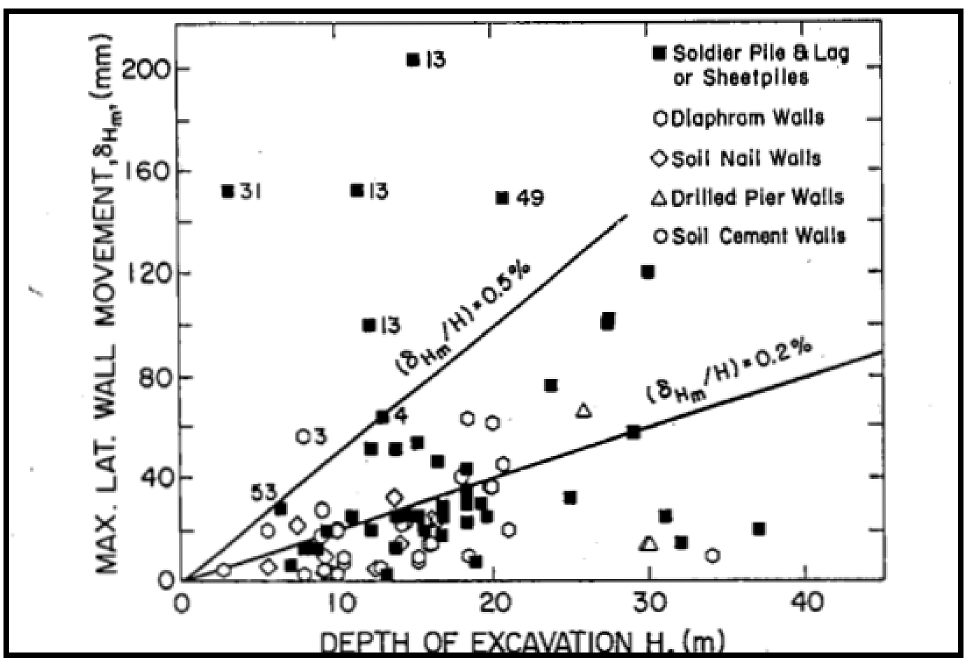

Figure 7-4. Observed Maximum Lateral Movements for *In Situ* Walls in Stiff Clays, Residual Soils and Sands (source: Clough and O'Rourke 1990)

By review of Figure 7-4, it can be seen that a well-constructed soldier pile wall- that is, a wall similar to that anticipated to be constructed for this project- might expect to have a horizontal movement (δ_{Hm}) of about 0.2% of the wall height (H). Assuming H ~20 feet, design might anticipate δ_{Hm} ~ 0.5-inch.

If deflection of the top of the wall is limited to about 0.5-inch, it is NOVA's expectation that the resultant ground movement *immediately* behind/adjacent to the wall will be a similar magnitude, perhaps as great as 0.5-inch. This vertical movement will taper from that point, such that settlement at a distance of about 10 feet from the wall will be about 0.4-inch.

7.6.3 Condition 2

Walls designed for Condition 2 will likely limit deflection of the top of the wall to 1-inch or less. This wall movement will limit ground settlement immediately behind the shoring system to a similar amount, or less. This movement should be imperceptible beyond a distance of about 20 feet from the wall.

7.7 Construction

Walls will be constructed by first setting the soldier beams. Thereafter, the pace of the excavation will be limited by the establishment of lagging. Excavation should not be advanced deeper than about 3 feet below the bottom of the lagging at any time. These gaps of up to 3 feet should only be allowed to stand for short periods of time in order to decrease the potential for sloughing/caving. Backfilling should be conducted when necessary between the back of the lagging and excavation sidewalls to reduce any sloughing in this zone.

The Unit 2 and Unit 3 soils are favorable for sound wall construction. The Geotechnical Engineer-of-Record (GEOR) should coordinate with the Shoring Engineer to ensure that good workmanship prevails throughout wall construction. The combination of workmanship and favorable subsurface conditions will result in good wall performance. Additionally, ground and wall movement monitoring should be employed to detect any unusual wall movement before the condition becomes problematic.

1. As is discussed above, a 20-foot tall wall might expect to have a horizontal movement (δ_{Hm}) of about 0.2% of the wall height (H), δ_{Hm} ~ 0.5-inch to δ_{Hm} ~ 1-inch.

2. <u>Vibration Monitoring</u>. Construction will be completed by near structures that bound the site to the north. Every effort should be made to limit ground vibrations in this area. In particular, soldier piles in this area should be drilled, not driven. Despite even the best efforts, because the human body can sense vibrations at a level much lower level than that necessary to effect damage to even sensitive buildings, nearby property owners may claim damage or the threat of damage during the period of excavation/shoring/ foundation construction. Construction should include planning for periodic vibration monitoring. Such monitoring may avert or provide defense for any such claims.

3. <u>Pre-Construction Survey</u>. The condition of the pavements, structures and utilities near the excavation should be documented by a careful walk-over by an experienced structural and/or geotechnical engineer. These observations will include condition of the ground floor slab, walls and roof, observing these elements for any signs of distress/movement. Photo documentation is an important part of any pre-construction

survey. Observations of this survey may suggest the need for monitoring of cracks or other areas of evident distress in this building.

4. Building Monitoring. The movement of the building that borders the site to the north should be monitored by survey during construction. If a building shows evidence of movement, the floor level measurements, structure inspections, etc. may need to be reproduced.

5. Soldier Beam Monitoring. Prior to construction select soldier beams should be marked and surveyed, establishing a basis for a long-term plot of soldier pile movement with time.

6. Ground Monitoring. The ground surrounding the excavation, to a distance (where accessible) of at least 20 feet from the walls, should be periodically surveyed for evidence of settlement. Such monitoring will require a preconstruction ground survey.

7. Post-Construction Building Condition Survey. The pre-construction building survey should be reproduced at the end of construction, establishing the condition of structures of concern at that time. Claims by nearby property owners of movement-related building damage are common.

7.7.1 Contingency Plan

The above-described scope of monitoring should be sufficient to identify areas of evident concern for ground movement, as well as provide a defense against claims for such damage or nuisance.

The preconstruction survey may identify a particular need for careful, frequent monitoring. For example, if the survey shows the structure already shows a particular area of concern that could be profoundly degraded by wall movement, planning for monitoring during construction should reflect this condition.

Based on the indications of the preconstruction survey, a Contingency Plan may be appropriate. This plan would be developed by collaboration among the Shoring Engineer, Geotechnical Engineer and Contractor to identify actions that would be taken in the event that wall deformations rise to levels of concern.

8.0 STORMWATER INFILTRATION

8.1 Overview

Current planning for permanent stormwater Best Management Practices (BMP's) structures, includes development of a dry well at the northwest corner of the structure, extending the well to a depth of about 30 feet.

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 (hereinafter, 'the BMP Manual').

The feasibility of stormwater infiltration is principally dependent on structural, geotechnical and hydrogeologic conditions at the project site. In consideration of the infiltration rates observed during testing (see table 8-1), NOVA concludes that the site is feasible for the development of partially infiltrating permanent stormwater BMPs, such that factors listed in Section 8.3.1 will not adversely impact the future and neighboring structures.

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

8.2 Infiltration Rates

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 8-1 provides a summary of the infiltration rates determined by the percolation testing.

Table 8-1. Infiltration Rates Determined by Percolation Testing

Boring	Approximate Ground Elevation	Geologic Unit	Depth of Test	Approximate Test Elevation (feet, msl)	Infiltration Rate (inches/hour)	Design Infiltration Rate (in/hour, FS=2*)
P-1	+110	Kp	30	+80	0.82	0.41
P-2	+110	Kp	30	+80	0.42	0.21

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

As may be seen by review of Table 8-1, a factor of safety (FS) is applied to the calculated infiltration rate (I) determined by the percolation testing. This factor of safety, at least FS = 2 in local practice, considers the nature and variability of subsurface materials, as well as the natural tendency of infiltration structures to become less efficient with time. The calculated infiltration rates at locations P-1 and P-2 after applying F = 2 are 0.41 and 0.21 inches per hour, respectively.

Partial BMPs may be considered with infiltration rates greater than 0.05-inches per hour and less than 0.5-inches per hour.

8.3 Review of Geotechnical Feasibility Criteria

8.3.1 Overview

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

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

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

8.3.2 Soil and Geologic Conditions

The soil borings and percolation tests borings completed for this assessment disclose the sequence of soil units described below. The location of these units is presented on Plate 1.

1. Unit 1, Fill. The site is covered by artificial fill (Qaf) that ranges from 7 to 15 feet in thickness and comprised of poorly graded ('well sorted') fine to medium sands of medium dense consistency. The fill is 'undocumented,' and at risk for wide variations in quality.

2. Unit 2, Old Paralic Deposits. Beneath the fill, the site is underlain by Quaternary-aged old paralic deposits (Qop). This unit is characteristically of very dense consistency, composed of layers of cemented poorly graded clayey and silty sandstone.

3. Unit 3, Point Loma. Beneath the old paralic deposits, the site is underlain by Cretaceous-aged Point Loma Formation (Kp), which extends beyond the depth of the deepest field explorations. The unit is comprised of layers of sandstone and siltstone of very dense consistency.

8.3.3 Settlement and Volume Change

Settlement and soil volume change due to stormwater infiltration is not a concern with: (i) low expansive soils, (ii) no potential for liquefaction, and (iii) no potential for hydro collapse.

8.3.4 Slope Stability

BMPs should not be sited within 50 feet of an existing slope over 25%. Stormwater infiltration would not affect embankment stability at this or adjacent properties.

8.3.5 Utilities

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

8.3.6 Groundwater Mounding

Stormwater infiltration can result in damaging ground water mounding during wet periods. Based on the depth to groundwater, groundwater mounding is not a high risk.

8.3.7 Retaining Walls and Foundations

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

8.3.8 Other Factors

Full and partial BMPs should not be placed within existing fill materials greater than 5 feet thick. The fill on site is as deep as 15 feet bgs. This condition is unsuitable for stormwater infiltration. However, the current design is for infiltration dry wells extending to a depth of 30 feet. Planning for deep wells should not be affected by the existing fill on site.

8.4 Suitability of the Site for Stormwater Infiltration

It is NOVA's judgment that the planning for the current 30-foot dry well location will be suitable for partial infiltration; however, this judgment should be reviewed when the siting of BMPs with respect to proposed structures, utilities, and other improvements has been finalized by the Civil Engineer.

Appendix D provides completed forms related to stormwater infiltration.

9.0 CONSTRUCTION REVIEW, OBSERVATION AND TESTING

9.1 Overview

As is discussed in Section 1, the recommendations contained in this report are based upon a limited number of borings and an assumption of general continuity of subsurface conditions between borings.

The recommendations provided in both NOVA's proposal for this work and this report assume that NOVA will be retained to provide consultation and review during the design phase, to interpret this report during the construction phase, and to provide construction monitoring in the form of testing and observation.

9.2 Design Phase Review

NOVA should be retained to provide review of final grading and foundation plans. This review is provided for in NOVA's proposal for this work.

9.3 Construction Observation and Testing

9.3.1 Preconstruction Conference

A preconstruction conference among representatives of the Owner, Contractor and/or Construction Manager, and GEOR is recommended to discuss the planned construction procedures and quality control requirements.

9.3.2 Special Inspections

Special inspections should be provided per Section 1705 of the California Building Code. The soils special inspector should be a representative of NOVA as the Geotechnical Engineer-of-Record (GEOR).

NOVA should be retained to provide construction-related services abstracted below.

- Surveillance during site preparation, grading, and foundation excavation.
- Construction of temporary shoring.

A program of quality control should be developed prior to the beginning of construction. It is the responsibility of the Owner, the Contractor and/or the Construction Manager to determine any additional inspection items required by the Architect/Engineer or the governing jurisdiction.

9.3.3 Continuous Soils Special Inspection

The earthwork operations listed below should be the object of continuous soils special inspection.

- Tieback installation and testing.
- Foundation and mat subgrade preparation/compaction.

9.3.4 Periodic Soils Special Inspection

The earthwork operations listed below should be the object of periodic soils special inspection, subject to approval by the Building Official.

- Site preparation and removal of existing development features.
- Placement and compaction of utility trench backfill.

9.3.5 Testing During Inspections

The locations and frequencies of compaction test should be determined by the geotechnical engineer at the time of construction. Test locations and frequencies may be subject to modification by the geotechnical engineer based upon soil and moisture conditions encountered, the size and type of compaction equipment used by the Contractor, the general trend of compaction test results, and other factors.

Of particular concern to NOVA during earthwork operations will be good practices in moisture conditioning, loose soil placement, and soil compaction. In particular, NOVA will be vigilant with regard to the use of compaction equipment appropriate to the full lift thickness of the type of soil being compacted. Reliance on construction traffic (for example, loaders or dump trucks) to achieve compaction will not be approved.

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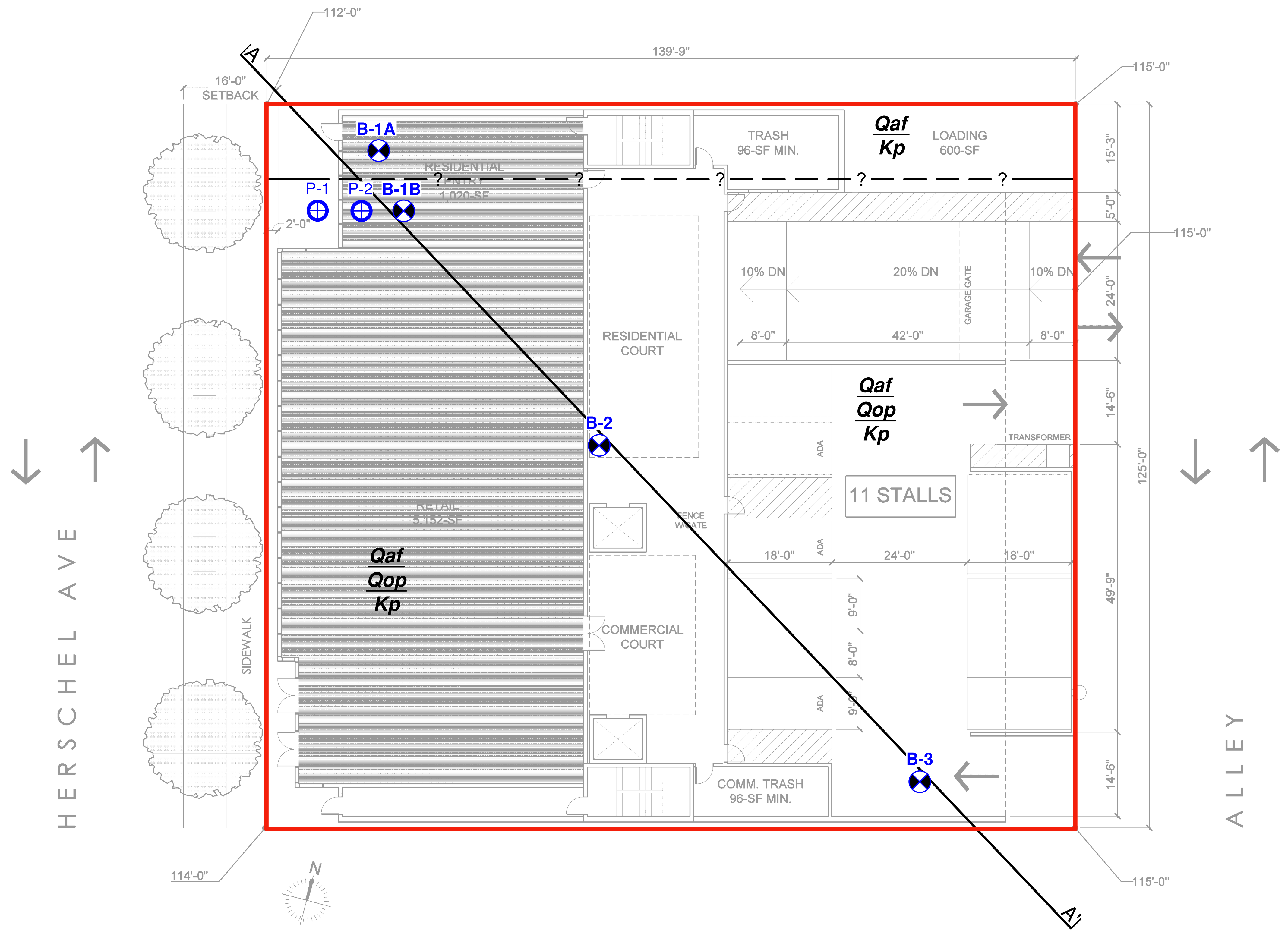
PLATES



KEY TO SYMBOLS

Qaf	FILL
Qop	OLD PARALIC DEPOSITS
Kp	POINT LOMA FORMATION
B-3	GEOTECHNICAL BORING
P-2	PERCOLATION TEST BORING
	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
	ANTICIPATED LIMITS OF EXCAVATION
A A'	GEOLOGIC CROSS-SECTION

0 20' 40'

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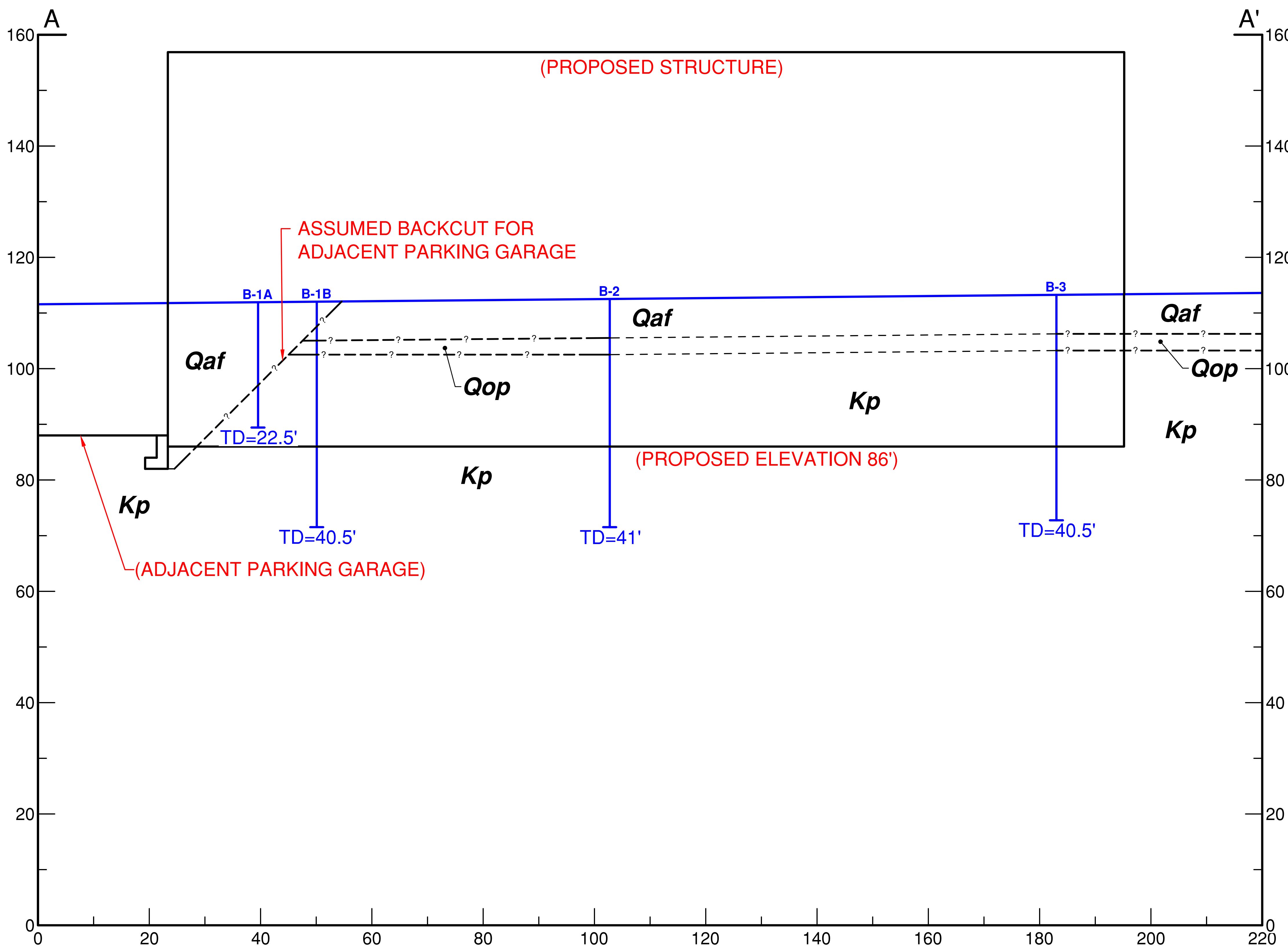
PROJECT NO.:	2020093
DATE:	JULY 2020
DRAWN BY:	DTJ
REVIEWED BY:	WM
SCALE:	1"=20'
DRAWING TITLE:	

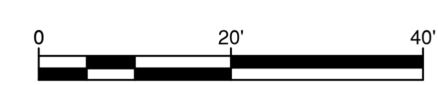
SUBSURFACE
INVESTIGATION MAP

PLATE NO. 1 OF 2



KEY TO SYMBOLS

Qaf	FILL
Qop	OLD PARALIC DEPOSITS
Kp	POINT LOMA FORMATION
(contact line)	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
B-3	GEOTECHNICAL BORING



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PROJECT NO.:	2020093
DATE:	JULY 2020
DRAWN BY:	DTJ
REVIEWED BY:	WM
SCALE:	1"=20'
DRAWING TITLE:	

CROSS SECTION AA'
MAP

PLATE NO. 2 OF 2

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

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e-mail: info@asfe.org www.asfe.org

APPENDIX B
LOGS OF ENGINEERING BORINGS

BORING LOG B-1A

DATE EXCAVATED:	JUNE 17, 2020	**EQUIPMENT:**	CME 75, 95
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 111.8 FT MSL

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0						*3 INCHES OF ASPHALT CONCRETE*		
				SP-SM	5	**FILL (Qaf):** *POORLY GRADED SAND-SILTY SAND; DARK BROWN, MOIST, LOOSE, FINE TO MEDIUM GRAINED* *BROWN, SCATTERED GRAVEL, 8" CONCRETE DEBRIS, SCATTERED ROOTS*	MD	
5				SC-SM	10	*CLAYEY SAND-SILTY SAND; ORANGE BROWN, MOIST, LOOSE, FINE TO MEDIUM GRAINED, SCATTERED SHELL FRAGMENTS, SCATTERED ROOTS* *BROWN*	SA DS MD SA	
10					10	*OLIVE BROWN MOTTLED WITH ORANGE BROWN*	SA	
15				SC-SM	50/6"	**POINT LOMA FORMATION (Kp):** *CLAYEY SANDSTONE-SILTY SANDSTONE; OLIVE BROWN, MOIST, VERY DENSE, FINE TO MEDIUM GRAINED*	SA	*SMALL COBBLE IN SHOE*
20				CL	50/2" 50/3"	*SANDY CLAYSTONE; OLIVE BROWN, MOIST, HARD*	EI SA	*37 LOW*
						BORING TERMINATED AT 22.5 FT DUE TO REFUSAL ON HARD POINT LOMA FORMATION (Kp). NO GROUNDWATER ENCOUNTERED. NO CAVING		
25								
30								

KEY TO SYMBOLS

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

LA VALENCIA MIXED-USE BUILDING
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LA JOLLA, CALIFORNIA

APPENDIX B.1

LOGGED BY:	GAN	DATE:	JUL 2020
REVIEWED BY:	MS	PROJECT NO.: 2020093	

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BORING LOG B-1B

DATE EXCAVATED:	JUNE 18, 2020	**EQUIPMENT:**	CME 75, 95
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 112.1 FT MSL

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0						*3 INCHES OF ASPHALT CONCRETE*		
						LOGGING BEGINS AT 22 FT.		
5								
10								
15								
20								
						POINT LOMA FORMATION (Kp): *SANDY SILTSTONE; OLIVE BROWN, MOIST, HARD*		
25		☒	◸	SC-SM	50/3"	*CLAYEY SANDSTONE-SILTY SANDSTONE; OLIVE BROWN, MOIST, VERY DENSE, FINE GRAINED, GRAVEL LAYER*	SA	
30								

KEY TO SYMBOLS

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

LA VALENCIA MIXED-USE BUILDING
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APPENDIX B.2

LOGGED BY:	GAN	DATE:	JUL 2020
REVIEWED BY:	MS	PROJECT NO.: 2020093	

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CONTINUED BORING LOG B-1B

DATE EXCAVATED:	JUNE 18, 2020	**EQUIPMENT:**	CME 75, 95
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 112.1 FT MSL

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
30				SC-SM	50/3"	**POINT LOMA FORMATION (Kp):** *(CONTINUED) CLAYEY SANDSTONE-SILTY SANDSTONE; OLIVE BROWN, MOIST, VERY DENSE, FINE GRAINED*		
35					50/3"			
40					50/5"	*ORANGE STAINING*		
						BORING TERMINATED AT 40.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
45								
50								
55								
60								

KEY TO SYMBOLS

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

LA VALENCIA MIXED-USE BUILDING
HERSCHEL AVENUE
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APPENDIX B.3

LOGGED BY:	GAN	DATE:	JUL 2020
REVIEWED BY:	MS	PROJECT NO.: 2020093	

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BORING LOG B-2

DATE EXCAVATED: JUNE 18, 2020
EQUIPMENT: CME 75, 95
EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING
GPS COORD.: N/A
GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED
ELEVATION: ± 112.6 FT MSL

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0						*3 INCHES OF ASPHALT CONCRETE*		
				SP-SM		**FILL (Qaf):** *POORLY GRADED-SILTY SAND; BROWN, MOIST, LOOSE TO MEDIUM DENSE, MEDIUM GRAINED*		
5						*LIGHT BROWN*		
				SC-SM		**OLD PARALIC DEPOSITS (Qop):** *CLAYEY SAND-SILTY SAND; BROWN WITH WHITE MOTTLING, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED SHELLS TRACE GRAVEL*		
10			CAL. MOD. SAMPLE	SM	50/6"	**POINT LOMA FORMATION (Kp):** *SILTY SANDSTONE; YELLOW-OLIVE BROWN, MOIST, VERY DENSE, FINE TO COARSE GRAINED* *OLIVE BROWN*	DS	
15								
20		BULK SAMPLE						
25			SPT SAMPLE	SM/ML	50/3"	*SANDY SILTSTONE/SANDY SILTSTONE; OLIVE BROWN, MOIST, VERY DENSE, FINE GRAINED*		
30								

KEY TO SYMBOLS

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

LA VALENCIA MIXED-USE BUILDING
HERSCHEL AVENUE
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APPENDIX B.4

LOGGED BY: GAN | DATE: JUL 2020
REVIEWED BY: MS | PROJECT NO.: 2020093

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CONTINUED BORING LOG B-2

DATE EXCAVATED:	JUNE 18, 2020	**EQUIPMENT:**	CME 75, 95
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 112.6 FT MSL

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
30			⧄	ML	50/3"	**POINT LOMA FORMATION (Kp):** *(CONTINUED) SANDY SILTSTONE; OLIVE BROWN, MOIST, HARD*		
35			⧄	SM	50/4"	*SILTY SANDSTONE; OLIVE BROWN, MOIST, VERY DENSE, FINE GRAINED*		
40			⧄	ML	50/4"	*SANDY SILTSTONE; OLIVE BROWN, MOIST, HARD*		
						BORING TERMINATED AT 41 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
45								
50								
55								
60								

KEY TO SYMBOLS

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

LA VALENCIA MIXED-USE BUILDING
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APPENDIX B.5

LOGGED BY:	GAN	DATE:	JUL 2020
REVIEWED BY:	MS	PROJECT NO.:	2020093

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BORING LOG B-3

DATE EXCAVATED:	JUNE 18, 2020	**EQUIPMENT:**	CME 75, 95
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 113.1 FT MSL

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0						*3 INCHES OF ASPHALT CONCRETE*		
				SM		**FILL (Qaf):** *SILTY SAND; DARK BROWN, MOIST, LOOSE, FINE TO MEDIUM GRAINED*		
			CAL. MOD.		8			
5						*ORANGE BROWN*		
				SC-SM		**OLD PARALIC DEPOSITS (Qop):** *CLAYEY SANDSTONE-SILTY SANDSTONE; ORANGE BROWN, MOIST, VERY DENSE, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL*		
10			SPT		50/6"			*ROCK FRAGMENTS IN SAMPLE*
				ML/SM		**POINT LOMA FORMATION (Kp):** *SANDY SILTSTONE/SILTY SANDSTONE; OLIVE BROWN, MOIST, VERY DENSE/HARD, FINE GRAINED*		
15								
20								
25		BULK	SPT		50/3"	*SCATTERED GRAVEL*		
							CR	
30								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
◻	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

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APPENDIX B.6

LOGGED BY:	GAN	DATE:	JUL 2020
REVIEWED BY:	MS	PROJECT NO.:	2020093

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CONTINUED BORING LOG B-3

DATE EXCAVATED: JUNE 18, 2020 **EQUIPMENT:** CME 75, 95

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

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 113.1 FT MSL

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
30				SM/ML	75	**POINT LOMA FORMATION (Kp):** *(CONTINUED) SILTY SANDSTONE/SANDY SILTSTONE; OLIVE BROWN WITH ORANGE AND DARK BROWN INTERBEDDING, MOIST, VERY DENSE/HARD, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL*		
35					50/5"	*YELLOW-OLIVE BROWN*		
40					50/4"			
						BORING TERMINATED AT 40.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
45								
50								
55								
60								

KEY TO SYMBOLS

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

LA VALENCIA MIXED-USE BUILDING

HERSCHEL AVENUE

LA JOLLA, CALIFORNIA

APPENDIX B.7

LOGGED BY:	GAN	DATE:	JUL 2020
REVIEWED BY:	MS	PROJECT NO.: 2020093	

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PERCOLATION BORING LOG P-1

DATE EXCAVATED:	JUNE 17, 2020	**EQUIPMENT:**	CME 75, 95
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 111.5 FT MSL

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0						*3 INCHES OF ASPHALT CONCRETE*		
				SP		**FILL (Qaf):** *POORLY GRADED SAND; DARK BROWN, DAMP TO MOIST, LOOSE, MEDIUM GRAINED, SCATTERED ROOTS* *BROWN* *ORANGE BROWN* *BROWN*		
5								
				SC-SM		**OLD PARALIC DEPOSITS (Qop):** *CLAYEY SANDSTONE-SILTY SANDSTONE; BROWN DAMP TO MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
10				ML/CL		**POINT LOMA FORMATION (Kp):** *SANDY SILTSTONE/SANDY CLAYSTONE; OLIVE BROWN, MOIST, STIFF*		
				ML		*SANDY SILTSTONE; OLIVE BROWN, MOIST, STIFF*		
15								
20				CL		*SILTY CLAYSTONE; OLIVE BROWN, MOIST, STIFF*		
25								
				ML		*SANDY SILTSTONE; OLIVE BROWN, MOIST, VERY STIFF TO HARD*		
30						*BORING TERMINATED AT 30 FT AND CONVERTED TO A PERCOLATION WELL.*		

KEY TO SYMBOLS

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

LA VALENCIA MIXED-USE BUILDING
HERSCHEL AVENUE
LA JOLLA, CALIFORNIA

APPENDIX B.8

LOGGED BY:	GAN	DATE:	JUL 2020
REVIEWED BY:	MS	PROJECT NO.: 2020093	

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PERCOLATION BORING LOG P-2

DATE EXCAVATED:	JUNE 17, 2020	**EQUIPMENT:**	CME 75, 95
EXCAVATION DESCRIPTION:	8-INCH DIAMETER AUGER BORING	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	GROUNDWATER NOT ENCOUNTERED	**ELEVATION:**	± 111.8 FT MSL

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0						*3 INCHES OF ASPHALT CONCRETE*		
				SP		**FILL (Qaf):** *POORLY GRADED SAND; DARK BROWN, DAMP TO MOIST, LOOSE, MEDIUM GRAINED, SCATTERED ROOTS* *BROWN* *ORANGE BROWN*		
5						*BROWN*		
				SC-SM		**OLD PARALIC DEPOSITS (Qop):** *CLAYEY SANDSTONE-SILTY SANDSTONE; BROWN DAMP TO MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED $\frac{1}{4}$" GRAVEL*		
10				ML/CL		**POINT LOMA FORMATION (Kp):** *SANDY SILTSTONE/SANDY CLAYSTONE; LIGHT OLIVE BROWN, MOIST TO WET, STIFF* *OLIVE BROWN*		
				ML		*SANDY SILTSTONE; OLIVE BROWN, MOIST, STIFF*		
15						*HARD*		*REFUSAL WITH CME 75, STEPOUT WITH CME 95*
20				CL/ML		*SANDY CLAYSTONE/SANDY SILTSTONE; OLIVE BROWN, MOIST, STIFF*		
25				ML		*SANDY SILTSTONE; OLIVE BROWN, MOIST, VERY STIFF TO HARD*		
30						*BORING TERMINATED AT 30 FT AND CONVERTED TO A PERCOLATION WELL.*		

KEY TO SYMBOLS

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

LA VALENCIA MIXED-USE BUILDING
HERSCHEL AVENUE
LA JOLLA, CALIFORNIA

APPENDIX B.9

LOGGED BY:	GAN	DATE:	JUL 2020
REVIEWED BY:	MS	PROJECT NO.: 2020093	

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APPENDIX C
RECORDS OF LABORATORY TESTING

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

- **CLASSIFICATION:** *Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soils Classification System and are presented on the exploration logs in Appendix B.*
- **ATTERBERG LIMITS (ASTM D 4318):** *Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System.*
- **EXPANSION INDEX (ASTM 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 thick 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.*
- **DIRECT SHEAR TEST (ASTM D3080):** *Direct shear tests were performed on remolded and relatively undisturbed samples in general accordance with ASTM D3080 to evaluate the shear stregth characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions.*
- **CORROSIVITY TEST (CAL. TEST METHOD 417, 422, 643):** *Soil PH, and minimum resistivity tests were performed on a representative soil sample in general accordance with test method CT 643. The sulfate and chloride content of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively.*
- **GRADATION ANALYSIS (ASTM C 136 and/or ASTM D422):** *Tests were performed on selected representative soil samples in general accordance with ASTM D422. The grain size distributions of selected samples were determined in accordance with ASTM C 136 and/or ASTM D422. The results of the tests are summarized on Appendix C.3 through Appendix C.7.*

Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

Sample Location	Sample Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1A	0 - 4	Dark Brown Silty Sand	126.0	11.2
B-1A	5 - 10	Orange Brown Clayey Sand-Silty Sand	132.8	8.2

Density of Soil in Place (ASTM D2937)

Sample Location	Sample Depth (ft)	Soil Description	Moisture (%)	Dry Density (pcf)
B-3	2.5 - 3	Dark Brown Silty Sand	7.1	110.0

Atterberg Limits (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI	USCS (% Finer than No. 40)
B-1A	20 - 21	32	18	14	CL

Expansion Index (ASTM D4829)

Sample Location	Sample Depth (ft)	Expansion Index	Expansion Potential
B-1A	20 - 21	37	Low

Direct Shear (ASTM D3080)

Sample Location	Depth (ft)	Soil Description	Friction Angle (degrees)	Apparent Cohesion (psf)
B-2	10 - 11.5	Yellowish Olive Brown Silty Sandstone	35	110
B-1A	5 - 10	Orange Brown Clayey Sand-Silty Sand	29	192

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

Sample Location	Sample Depth (ft)	pH	Resistivity (Ohm-cm)	Sulfate Content (ppm)	Sulfate Content (%)	Chloride Content (ppm)	Chloride Content (%)
B-3	25 - 30	9.2	2100	42	0.004	21	0.002

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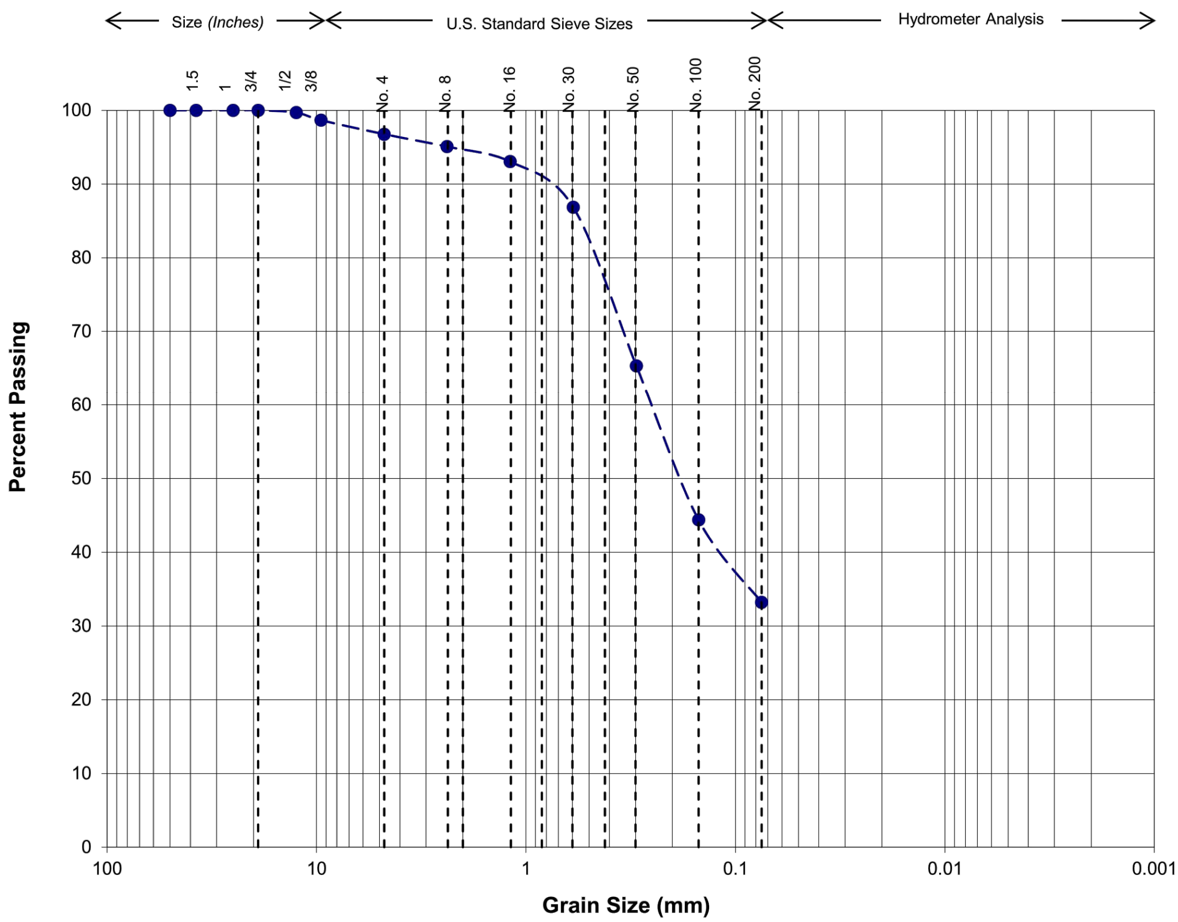
LAB TEST RESULTS

LA VALENCIA MIXED-USE BUILDING

HERSCHEL AVENUE

LA JOLLA, CALIFORNIA

BY: CLS	DATE: JUL 2020	PROJECT: 2020093	APPENDIX: C.2



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

Sample Location:	B-1A
Depth (ft):	5 - 6.5
USCS Soil Type:	SC-SM
Passing No. 200 (%):	33

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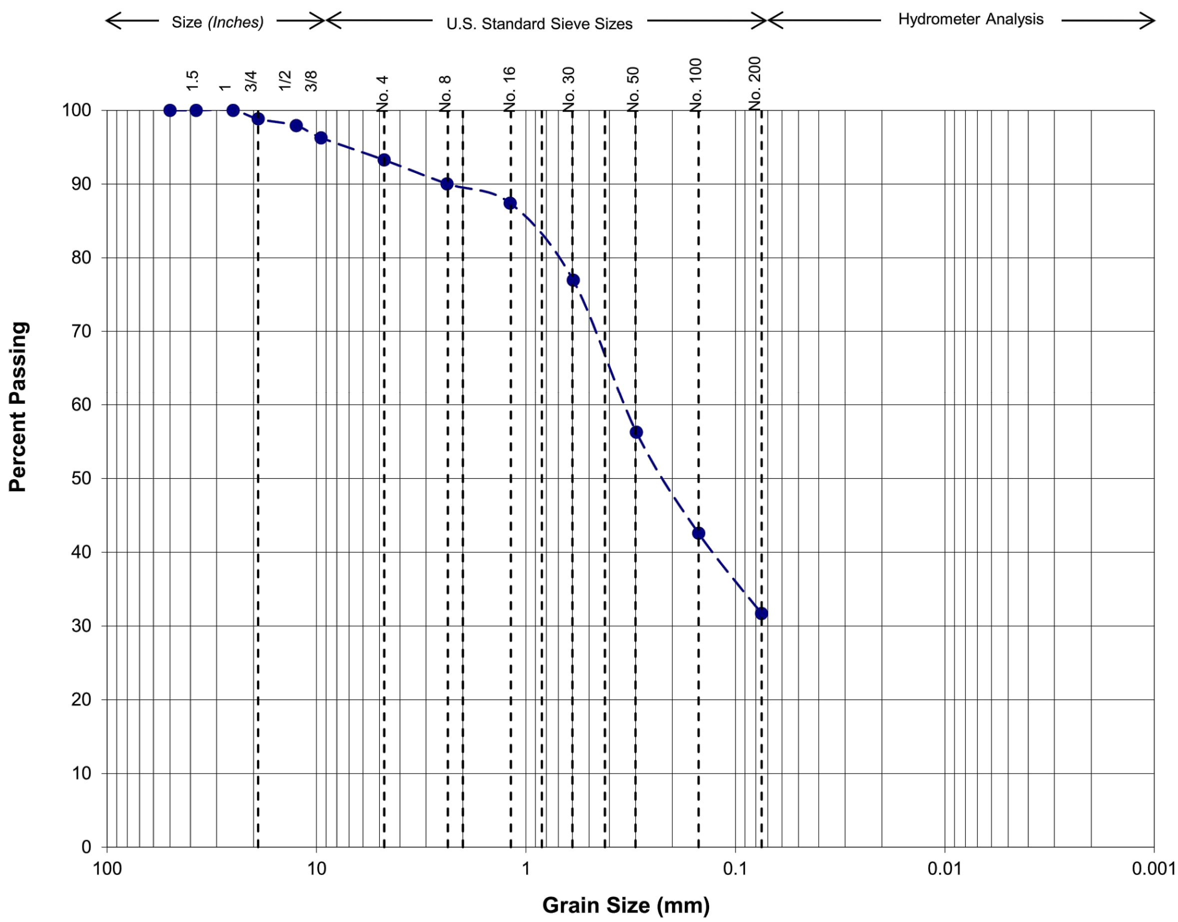
GRADATION ANALYSIS TEST RESULTS

LA VALENCIA MIXED-USE BUILDING
HERSCHEL AVENUE
LA JOLLA, CALIFORNIA

BY: CLS	DATE: JUL 2020	PROJECT: 2020093	APPENDIX: C.3



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-1A

Depth (ft): 10 - 11.5

USCS Soil Type: SC-SM

Passing No. 200 (%): 32

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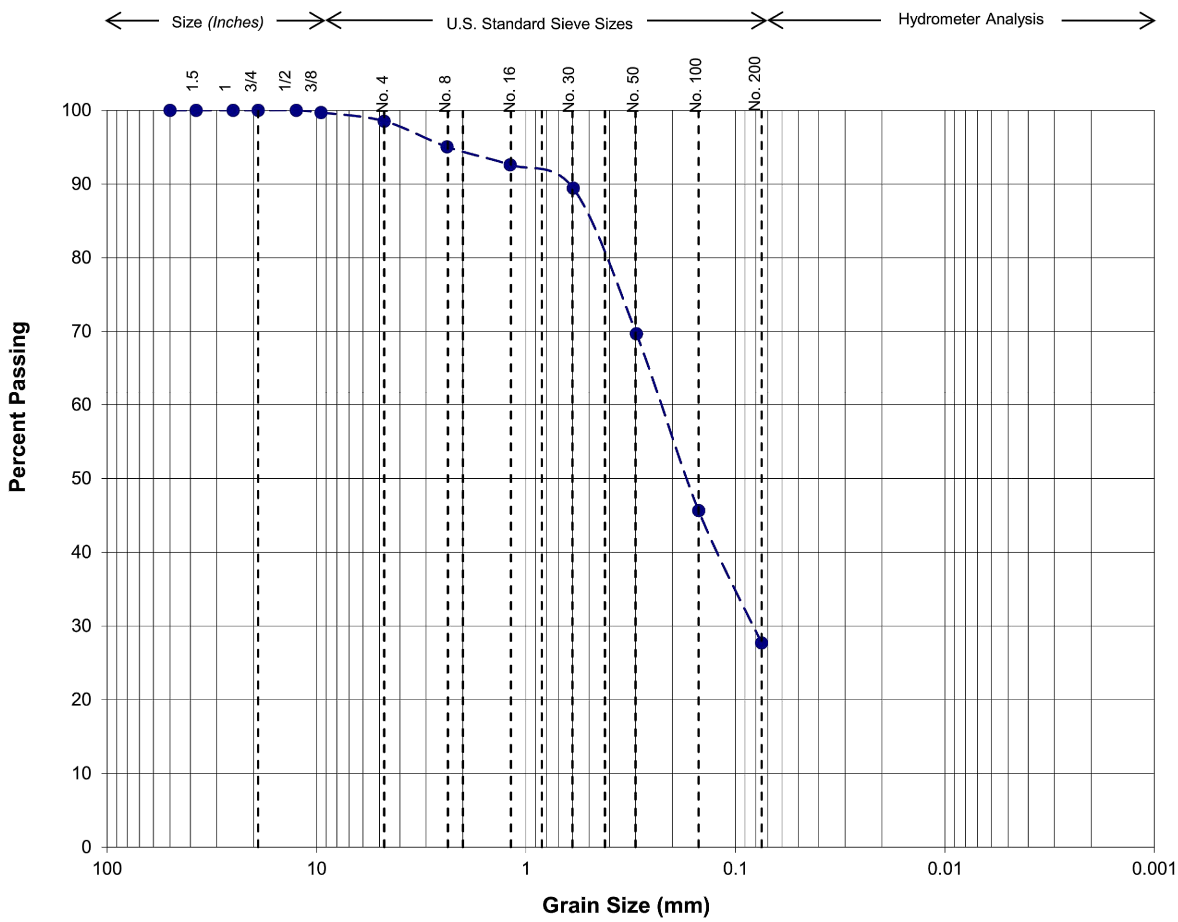
GRADATION ANALYSIS TEST RESULTS

LA VALENCIA MIXED-USE BUILDING

HERSCHEL AVENUE

LA JOLLA, CALIFORNIA

BY: CLS	DATE: JUL 2020	PROJECT: 2020093	APPENDIX: C.4



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-1A

Depth (ft): 15 - 16.5

USCS Soil Type: SC-SM

Passing No. 200 (%): 28

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GRADATION ANALYSIS TEST RESULTS

LA VALENCIA MIXED-USE BUILDING

HERSCHEL AVENUE

LA JOLLA, CALIFORNIA

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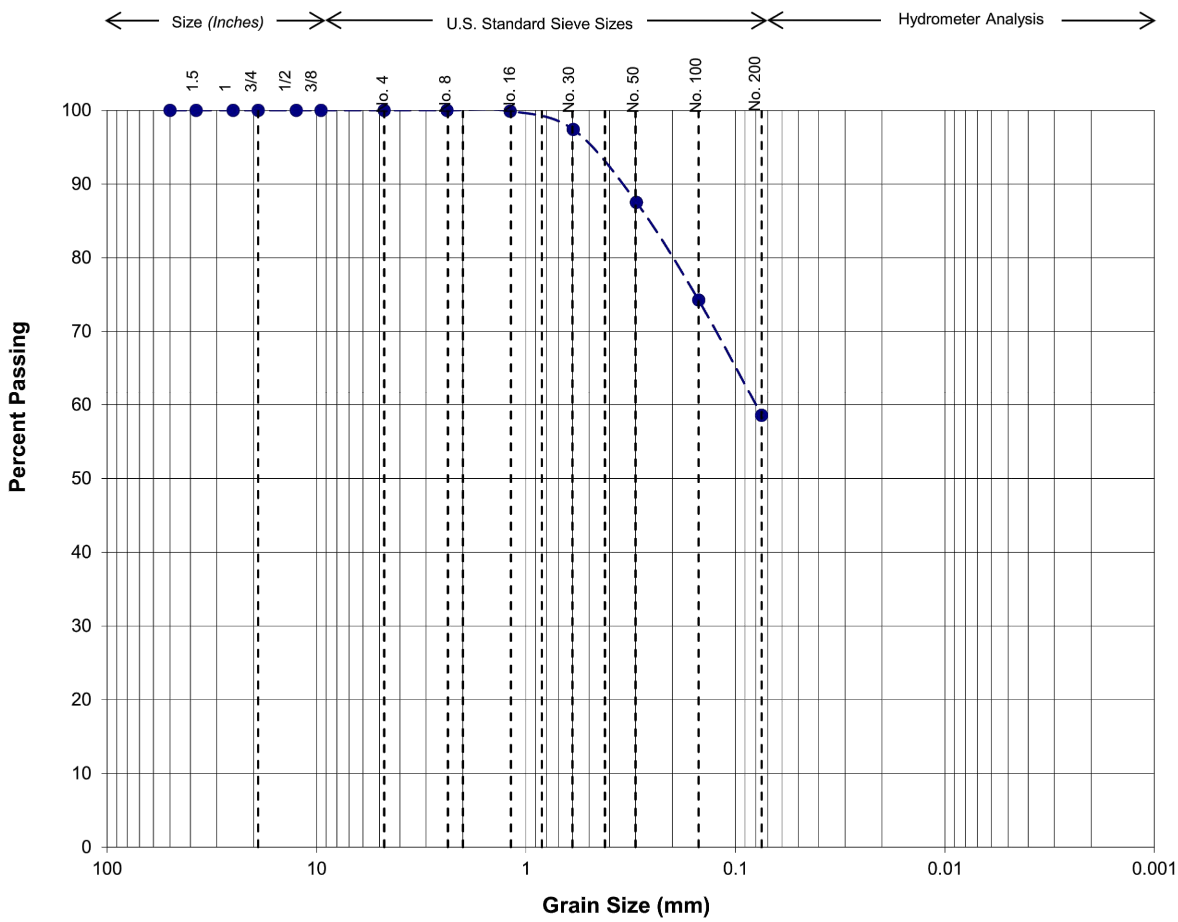

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-1A

Depth (ft): 20 - 21

USCS Soil Type: CL

Passing No. 200 (%): 59

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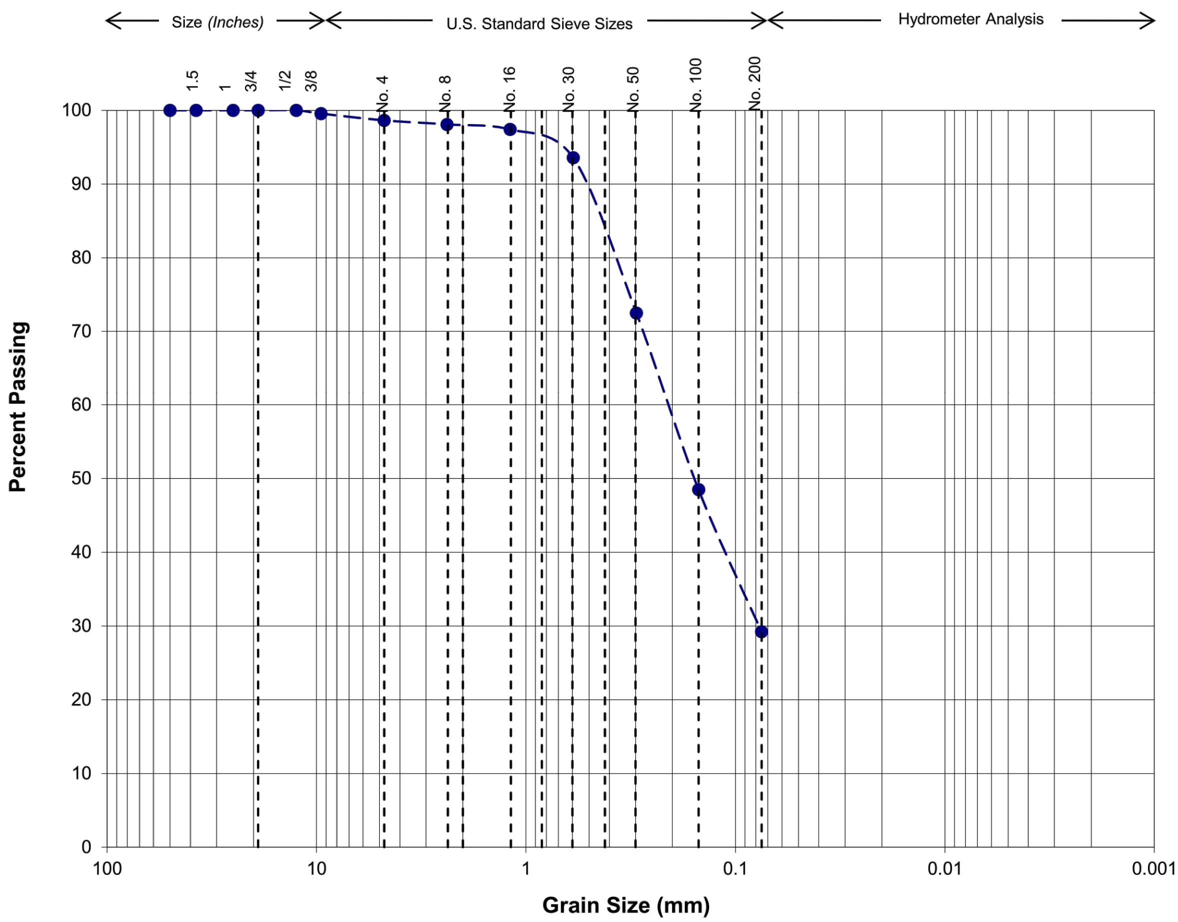
GRADATION ANALYSIS TEST RESULTS

LA VALENCIA MIXED-USE BUILDING

HERSCHEL AVENUE

LA JOLLA, CALIFORNIA

BY: CLS	DATE: JUL 2020	PROJECT: 2020093	APPENDIX: C.6



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-1B

Depth (ft): 25 - 26

USCS Soil Type: SC-SM

Passing No. 200 (%): 29

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GRADATION ANALYSIS TEST RESULTS

LA VALENCIA MIXED-USE BUILDING

HERSCHEL AVENUE

LA JOLLA, CALIFORNIA

BY: CLS	DATE: JUL 2020	PROJECT: 2020093	APPENDIX: C.7

APPENDIX D
STORMWATER INFILTRATION

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:
Location at P-1 and P-2		Planning Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[11]? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ☐ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ☒ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ☐ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ☐ Yes; Continue to Step 1C. ☐ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ☐ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ☐ Yes; continue to Step 1E. ☐ No; select an appropriate infiltration testing method.	

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

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

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[10]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ☐ Yes; continue to Step 1F. ☐ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ☐ Yes; continue to Step 1G. ☐ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ☐ Yes; answer "Yes" to Criteria 1 Result. ☐ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ☐ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ☒ No; full infiltration is not required. Skip to Part 1 Result.
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. The findings of this geotechnical investigation and infiltration assessment are detailed in NOVA 2020. A qualified representative of NOVA Services directed the drilling of two percolation test borings to depths of approximately 30 ft below ground surface (bgs) for the proposed dry well infiltration system. A continuously sampled exploratory boring near the BMP was drilled to approximately 40.5 ft bgs to evaluate the soil strata below the proposed BMP. The tests were conducted in compliance with the Borehole Percolation Tests method (D.3.3.2) of the BMP Manual. The percolation rates were converted to infiltration rates by the Porchet Method. Percolation testing indicated infiltration rates of 0.41 and 0.21 inches per hour for P-1 and P-2, respectively, utilizing a factor of safety of F=2.	

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

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

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

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

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
Locations at P-1 and P-2		Planning Phase
Criteria 3 : Infiltration Rate Screening		
3A	**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.	
3B	**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.	
Criteria 3 Result	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.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). Percolation test methods and infiltration results are detailed in a geotechnical investigation report (NOVA 2020). Percolation testing indicated infiltration rates of 0.41 and 0.21 inches per hour for P-1 and P-2, respectively, utilizing a factor of safety of F=2. Partial BMPs may be implemented in areas with tests resulting in infiltration rates greater than 0.05 inches per hour.		

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?	X Yes	☐ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	X 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?	X 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?	X 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?	X 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. See geotechnical investigation NOVA 2020.			

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.