UPDATE REPORT
GEOTECHNICAL INVESTIGATION

Proposed Two-Story Expansion and Renovation
Rancho Springs Medical Center Renovation
25500 Medical Center Drive, Murrieta, California

PREPARED FOR

RANCHO SPRINGS
MEDICAL CENTER
A campus of Southwest Healthcare System

UHS of Delaware, Inc.
C/O The Barrie Company
9434 Chesapeake Drive, Suite 1208
San Diego, CA 92123

PREPARED BY

NOVA

NOVA Services, Inc.
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NOVA Project No. 3019061

December 16, 2019

NOVA
GEOTECHNICAL ■ MATERIALS ■ SPECIAL INSPECTIONS
SBE ■ DVBE

UHS of Delaware, Inc.
C/O Elizabeth Barrie
The Barrie Company
9434 Chesapeake Drive, Suite 1208
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December 16, 2019
NOVA Project No. 3019061

Attention: Mrs. Elizabeth Barrie

Subject: Update Report
Geotechnical Investigation
Proposed Rancho Springs Medical Center Two-Story Expansion and Renovation
25500 Medical Center Drive, Murrieta, California

Dear Mrs. Barrie:

NOVA Services, Inc. (NOVA) is pleased to present herewith its geotechnical investigation for the above-referenced project. The work reported therein was completed by NOVA for UHS of Delaware, Inc., in accordance with the scope of work identified in NOVA's proposal dated July 16, 2019, as authorized on July 26, 2019. This report has been updated and includes 2019 California Building Code (CBC) Seismic Design Parameters after ASCE 7-16. This report updates and replaces the previously submitted report dated 30 September 2019.

NOVA appreciates the opportunity to be of continued service to The Barrie Company and UHS of Delaware, Inc. Should you have any questions, please do not hesitate to contact the undersigned at (949) 388-7710.

Sincerely,

NOVA Services, Inc.

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UPDATE REPORT
GEOTECHNICAL INVESTIGATION

Proposed Two-Story Expansion and Renovation
UHS Rancho Springs Medical Center
Murrieta, California

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1.0 INTRODUCTION

1.1 Terms of Reference

This report presents the findings of a geotechnical investigation of the site of a proposed two-story hospital building expansion and renovation to be constructed within the Rancho Springs Medical Center campus. This phase of development of the hospital will also include installation of a stormwater management facility.

The work reported herein was completed by NOVA Services, Inc. (NOVA) for UHS of Delaware, Inc. and The Barrie Company in accordance with the scope of work identified in NOVA's proposal dated July 16, 2019, as authorized on July 26, 2019.

Figure 1-1 depicts the vicinity of the Rancho Springs Medical Center campus.

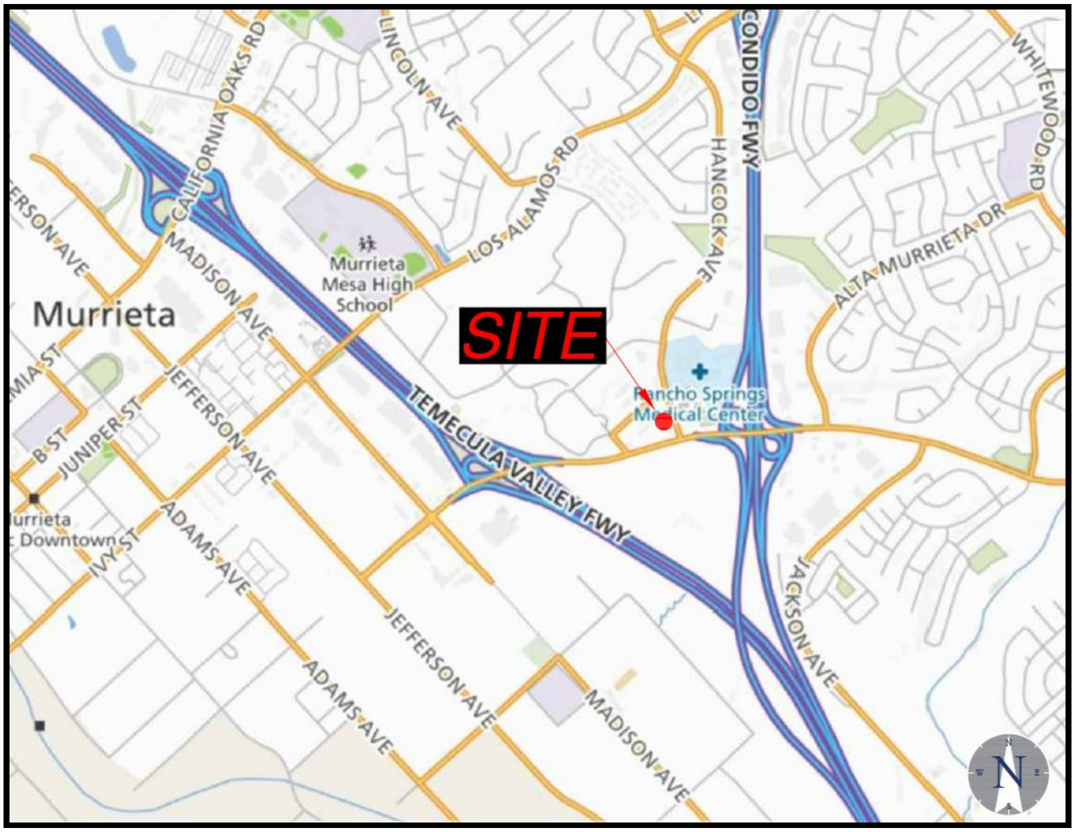

Figure 1-1. Vicinity Map

1.2 Objectives, Scope and Limitations of This Work

1.2.1 Objectives

The objectives of the work reported herein are twofold: (i) to characterize the subsurface conditions at the site in a manner sufficient to develop recommendations for geotechnical-related design and construction;

and, (ii) to conduct percolation testing to support development of recommendations for siting and design of permanent stormwater infiltration Best Management Practices ('BMPs').

1.2.2 Scope

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

- Task 1, Review. Reviewed background data, including geotechnical reports, fault investigation reports and maps, topographic maps, geologic data, aerial photographs and preliminary development plans for the project. Coordinated with the Structural Engineer to obtain current structural information.

- Task 2, Field Exploration. Completed a subsurface exploration that included the subtasks listed below.

    - *Subtask 2-1, Reconnaissance.* Conducted a site reconnaissance, including layout of the engineering borings and soundings. Underground Service Alert was notified for utility mark-out services.

    - *Subtask 2-2, Engineering Borings.* Drilled, logged and sampled seven (7) engineering borings to depths of about 15 to 50 feet below existing ground surface (bgs). The borings were drilled and sampled using ASTM methodologies.

    - *Subtask 2-3, Percolation Testing.* Drilled five (5) percolation test borings, following which percolation testing was completed in each boring.

    - *Subtask 2-4, Closure*. The engineering borings and percolation test borings were each closed following completion. Closure consisted of backfilling the borings with cuttings from the drilling, as required by the City of Murrieta. Thereafter, the area around each boring was cleaned and restored to its approximate condition prior to drilling.

- Task 3, Laboratory Testing. Laboratory testing of both bulk and relatively undisturbed samples was completed using ASTM testing methods.

- Task 4, Engineering Evaluations. Utilizing the findings of the preceding tasks, conducted engineering evaluations that address the geotechnical-related aspects of the planned construction.

- Task 5, Reporting. Preparation of this report providing NOVA's findings and preliminary geotechnical recommendations completes the scope of work described in NOVA's proposal.

1.2.3 Limitations

The construction recommendations in this report are not final. These recommendations are developed by NOVA using judgment and opinion and based upon the limited information available from the borings and soundings. NOVA can finalize its recommendations only by observing actual subsurface conditions revealed during construction. At the time of preparation of this report, neither construction nor proposed plans had been developed for the site. NOVA cannot assume responsibility or liability for the report's recommendations if NOVA does not perform construction observation.

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

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

1.3 Report Organization

The remainder of this report is organized as described below.

- Section 2 reviews the presently available project information.
- Section 3 describes the subsurface investigation and related laboratory testing.
- Section 4 describes the geologic setting and site-specific subsurface conditions.
- Section 5 reviews geologic, soil and siting-related hazards that commonly affect civil development in this region considering each for its potential to affect this site.
- Section 6 provides a description and an evaluation for developing seismic design parameters after ASCE 7-16 and 2019 California Building Code.
- Section 7 provides recommendations for earthwork and foundation-related design.
- Section 8 provides recommendations for development of stormwater infiltration BMPs.
- Section 9 provides recommendations for development of pavements.
- Section 10 lists the principal references utilized in preparation of this report.

Tables and figures that amplify discussion in the text of the report are embedded at the point at which they are referenced. Plates that provide larger scale views of certain figures 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 present logs of the borings. Appendix C provides records of the geotechnical laboratory testing.

The report is supported by four appendices.

- Appendix A presents guidance regarding use of this report.
- Appendix B provides logs of the engineering borings.
- Appendix C provides records of geotechnical laboratory testing.
- Appendix D provides documentation related to stormwater infiltration.

2.0 PROJECT INFORMATION

2.1 Location

The Rancho Springs Valley Medical Center is located at the address of 25500 Medical Center Drive, in the city of Murrieta, California. Plans for the proposed renovation are conceptual at this time, based upon referenced RFP documents (UHS RFP), NOVA understands that planned renovation will include the development of a new two-story expansion located in front of and immediately adjacent to an existing two-story emergency room building at the south side of the Rancho Springs Medical Center campus.

The medical campus and proposed project areas are bounded by vacant land to the north, Interstate 215 to the east, Murrieta Hot Springs Road to the south and Hancock Avenue to the west. Access to the medical campus is provided via the cul-de-sac of Medical Center Drive which terminates within the central portion of the campus.

Figure 2-1 provides a recent aerial view that depicts the location and approximate limits of the approximate project area at the site.

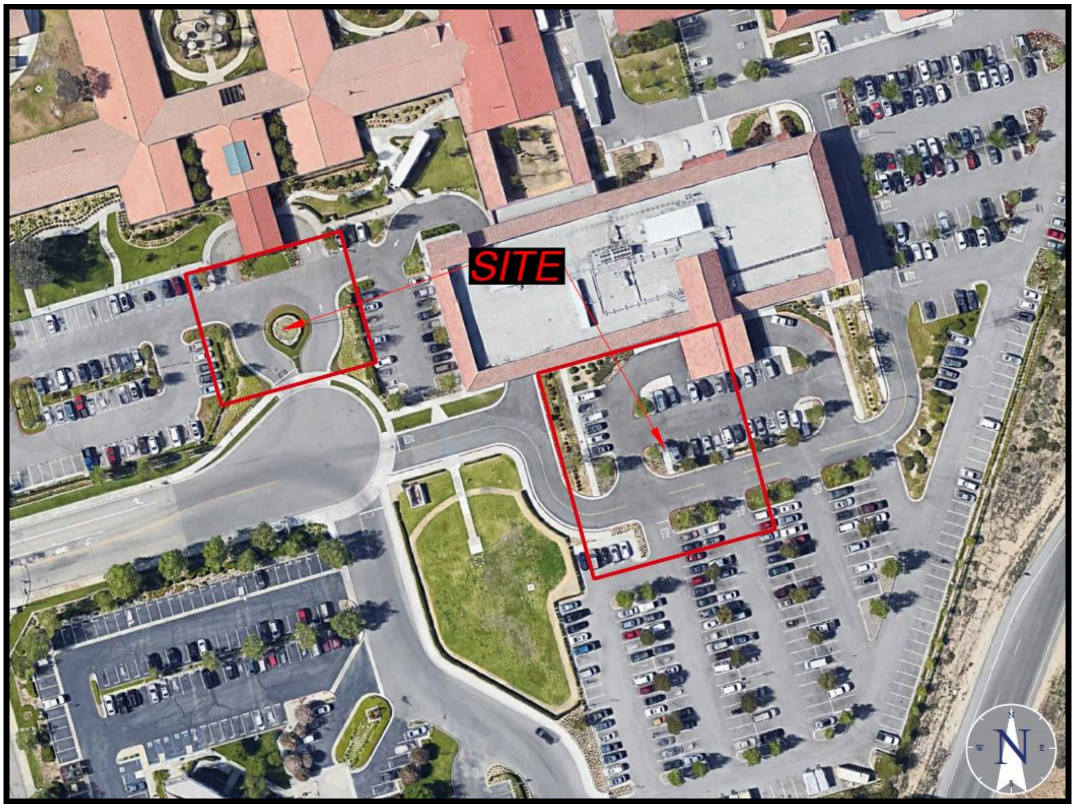

Figure 2-1. Location and Limits of the Site Improvements
(Source: adapted from Google Earth 2019)

2.2 Current and Historic Site Use

2.2.1 Current

As is evident by review of Figure 2-1, the proposed two-story expansion site location currently includes driveways and parking areas for the Rancho Springs emergency room. The parking areas include a few isolated landscaping islands supporting trees and arid environment ground cover. The average ground surface elevation in the vicinity of the planned two-story building expansion is about +1,150 and the area of the proposed stormwater infiltration system to the west is +1,147 feet mean sea level (msl), respectively.

2.2.2 Historic

NOVA reviewed historic aerial photography and topographic mapping dating to 1938 as a basis for understanding historical uses of the site. This review indicates that prior to development of Rancho Springs Hospital, the site area had minimal development. It appears that a historic drainage channel transected the campus as indicated in Figure 2-2. This drainage channel has been since graded out during the development of the hospital.

Figure 2-2. 1978 Historical Aerial Photograph of the Site Area

2.2.3 Previous Reporting

Previous geotechnical reporting for the development for some of the existing improvements and structures at Rancho Springs Valley Medical Center campus were reviewed. References to these reports are presented below.

- Leighton 2006. *Geotechnical Update Report, Rancho Springs Medical Office,* Leighton and Associates, Project No. 601207-001, January 3, 2006.

- Leighton 2007. *As Graded Report of Building Pad Remedial Grading and Post Grading, Rancho Springs Medical Office Building and Associated Improvements,* Leighton and Associates, Project No. 601207-003, November 20, 2007.

2.2.4 Schematic Planning

NOVA's understanding of current planning for the new two-story expansion and stormwater infiltration facility is based upon discussions with the design team, as well as review of the schematic design drawings that are listed below:

- KH 2019. *Rancho Springs Medical Center – Phase 2 Rough Grading,* Kimley Horn, 2019.

2.2.5 Architectural

Architect HOK describes the development of a two story, 18,095 square foot facility. NOVA anticipates the structure will be steel-framed centered near the southern midpoint of the existing hospital building, extending southward into the existing parking lot. The proposed expansion will be considered as an extension of the existing hospital emergency department at Rancho Springs Medical Center.

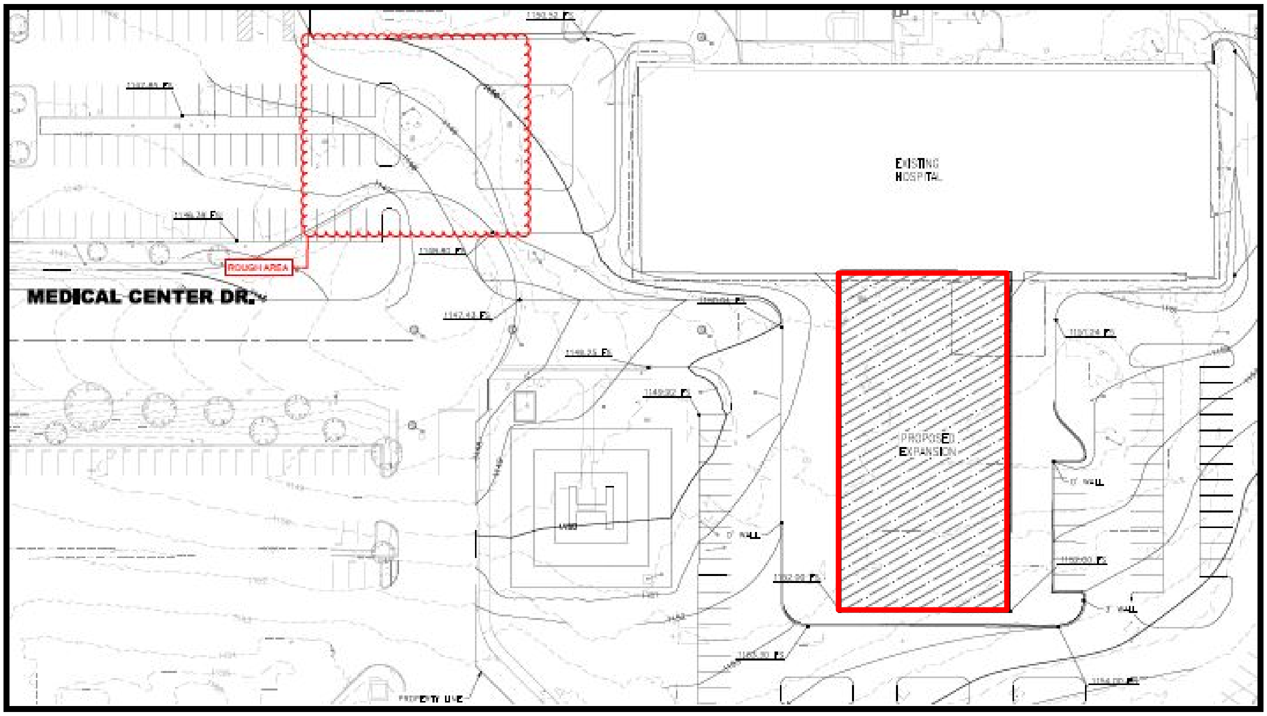

Figure 2-3. Proposed Two-Story Expansion and Infiltration Area
(Source: *Rancho Springs Medical Center, Phase 2 Rough Grading*, Kimley Horn 2019)

2.2.6 Structural

Limited information is available regarding structural concepts for the two-story expansion. Based upon experience with similar structures, NOVA expects that the new facility will be developed on shallow foundations, utilizing isolated and continuous foundations to support columns and walls. The interior floor slab will be a ground-supported mat. As noted above, it is anticipated that the structure will be steel framed.

Because design is still schematic, structural loads are unknown. However, Table 2-1 provides NOVA's estimate of the range of foundation reactions for this relatively light structure.

Table 2-1. Expected Column and Wall Loads (DL +LL)

Typical Exterior Col. Loads (kips)	Typical Interior Col. Loads (kips)	Typical Wall Loads (kips per lineal foot)
50 - 100	80 - 140	2- 4

2.2.7 Civil

The layout for the new facility is not yet finalized. Current planning considers options that generally center the planned expansion within the area of the existing asphalt-surfaced drive and parking area south of the existing emergency facility.

No below grade structures are depicted on the planning that has been reviewed by NOVA. Grading plans are not yet developed for the new facility. The current ground level is about one foot below the adjacent street level, such that it is expected that development of the site will likely involve placing two to three feet of fill to adapt the new facility to the existing site and adjacent roadways.

Planning for stormwater management is not yet finalized and remains conceptual. The site development option in Figure 2-5 depicts the use of a storm water management area located west of the existing emergency structure and north of the Medical Center Drive cul-de-sac.

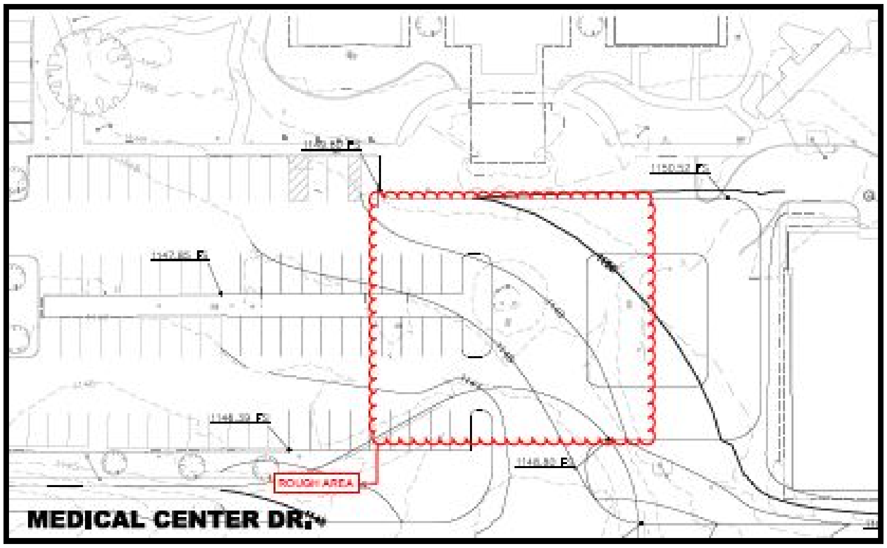

Figure 2-4. Proposed Stormwater Management Area
(Source: *Phase 2 Rough Grading* Kimley Horn 2019)

3.0 FIELD EXPLORATION AND LABORATORY TESTING

3.1 Overview

The field exploration of the site was conducted on August 19, 2019. NOVA completed seven engineering borings (‘B-1’ through ‘B-7’) and five percolation tests (‘P-1’ through ‘P-5’). The borings were drilled to a maximum depth of 50 feet below existing ground surface (bgs). Laboratory testing was completed on samples recovered from the borings. On November 2, 2019 a seismic traverse was performed to assess the one-dimensional average shear-wave velocity of the underlying site soils to a minimum depth of 100 feet bgs in order to classify the site in accordance with ASCE 7-16 Table 20.3-1.

Figure 3-1 provides a plan view of the site indicating the locations of the engineering and percolation test borings as well as the seismic traverse location (shown in blue). Plate 1, provided immediately following the text of this report, provides this graphic in larger detail.

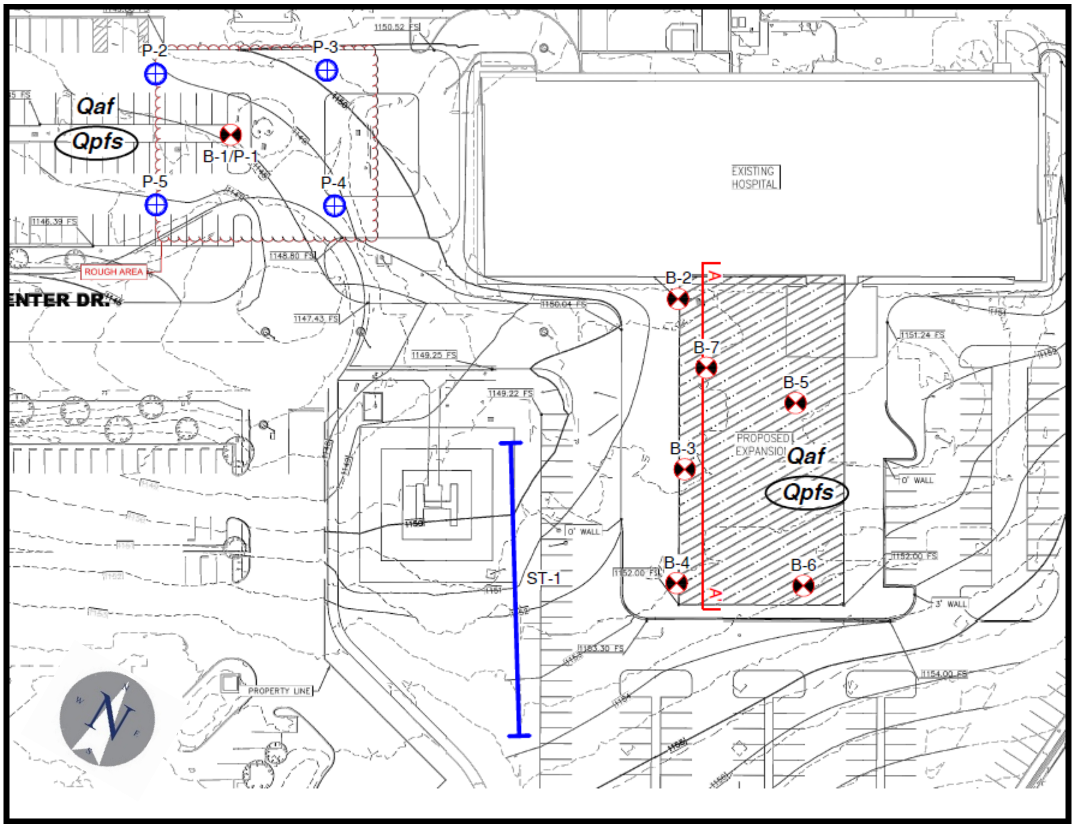

Figure 3-1. Engineering Borings, Percolation Test Borings, and Seismic Traverse Locations

3.2 Engineering Borings

3.2.1 Drilling

The geotechnical borings were advanced with a truck-mounted drill rig utilizing hollow stem drilling equipment. The borings were drilled at locations determined in the field by a NOVA geologist, then completed under the geologist's surveillance. Figure 3-2 below presents a photograph of the drilling operation.

Figure 3-2. Drilling Geotechnical Test Boring B-1

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

Table 3-1. Abstract of the Engineering Borings

Ref	Approx. Elev. (feet, msl)	Depth (feet)*	Approx. Ground Water Elevation (feet, msl)
B-1	+ 1,147	15.0	Not Encountered
B-2	± 1,149	26.5	Not Encountered
B-3	± 1,149	26.5	Not Encountered
B-4	± 1,151	21.5	Not Encountered
B-5	+1,151	21.5	Not Encountered
B-6	+ 1,151	21.5	Not Encountered
B-7	+ 1,150	50.0	Not Encountered

3.2.2 Sampling

Both disturbed and relatively undisturbed samples were recovered from the borings. Soil sampling was as described below.

1. The Modified California sampler ('ring sampler', after ASTM D3550) 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 D1586) was driven in the same manner as the ring sampler, recording blow counts in the same fashion. SPT blow counts for the final 12-inches of penetration comprise the SPT 'N' value, an index of soil consistency.

3. Bulk samples were recovered from the subsurface soils, providing composite samples for index testing.

Figure 3-3. Sample from B-7 at 30' bgs

3.2.3 Closure

Upon completion, each boring was backfilled with a mix of bentonite and soil cuttings and patched to match the existing surfacing.

Records of the engineering borings are presented in Appendix B.

3.3 Percolation Testing

3.3.1 General

NOVA directed the excavation and construction of five (5) percolation test borings, following the recommendations for percolation testing presented in the Riverside County Santa Margarita River

Watershed Region Design Handbook for Low Impact Development Best Management Practice (BMP) June 2018. The locations of these borings are shown in Figure 3-1.

3.3.2 Drilling

Borings were drilled with a truck mounted 8-inch hollow stem auger to the level of the base of expected stormwater infiltration BMPs. Field measurements were taken to confirm that the borings were excavated to approximately 8 inches in diameter.

The borings were logged by a NOVA geologist, who observed and recorded exposed soil cuttings and the boring conditions. Records of the feasibility documents for percolation testing are provided in Appendix D.

3.3.3 Conversion to Percolation Wells

Once the test borings were drilled to the design depth, the percolation test borings were converted to percolation 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 existing grade to minimize the potential of soil caving.

3.3.4 Percolation Testing

The percolation test borings were pre-soaked by filling the holes with water to the ground surface elevation. Testing was conducted the following day, within a 24-hour window.

Water levels were recorded every 30 minutes for 6 hours (minimum of 12 readings), or until the water percolation stabilized after each reading. At the start of each half-hour test interval, the water level was raised to approximately the same height of previous tests, in order to maintain a near constant head during the 6-hour test. Water level (depth) measurements were obtained from the top of the pipe. Table 3-2 (following page) abstracts the indications of the percolation testing.

Table 3-2. Abstract of the Percolation Testing

Boring	Approx. Elevation (feet, msl) [2]	Total Depth (feet)	Approximate Percolation Test Elev. (feet, msl)	Percolation Rate (in/hour)	Subsurface Unit Tested[1]
P-1[3]	± 1,147	15.0	± 1,132	113.8[3]	Qpfs
P-2	± 1,148	10.0	± 1,138	12.0	Qpfs
P-3	± 1,148	10.0	± 1,138	15.6	Qpfs
P-4	± 1,147	11.0	± 1,136	28.8	Qpfs
P-5	± 1,147	10.0	± 1,137	18.6	Qpfs

Notes:
1. 'Qpfs' indicates 'Pauba Formation', occurring as a dense sandstone
2. Percolation test elevations are estimated.
3. P-1 appears to be within an existing utility trench. Test stopped after 2 hours for erroneous rates.

3.3.5 Closure

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

3.4 Shear Wave Velocity Analysis

3.4.1 General

A seismic shear wave survey was performed on November 2, 2019 by a Professional Geophysicist (PGP). The purpose of the survey was to assess the one-dimensional average shear-wave velocity of the underlying site soils to a minimum depth of 100 feet bgs in order to classify the site in accordance with ASCE 7-16 Table 20.3-1. Multi-channel analysis of surface waves (MASW) and microtremor array measurement (MAM) methods were used for the analysis. Combining results of both methods maximizes the depth and resolution of the data.

Figure 3-4. Seismic Survey Line, View towards the South.

The seismic survey of the subject site included one seismic shear wave survey traverse, approximately 180 feet in length. The approximate location is shown on Figure 3-5 and Plate 1. A 24-channel Geometrics StrataVisor NZXP model signal-enhancement refraction seismograph was used in conjunction with 24 4.5-Hz geophones spaced at regular intervals.

For the MASW survey, two seismic records were obtained by multiple hammer strikes of a 16-pound sledge hammer on steel plates positioned 25 feet from the end of each terminus of the seismic line. Vibrations were recorded using a one second record length at a sampling rate of 0.5 milliseconds.

The MAM survey records vibrations from background and ambient noise. The ground vibrations were recorded using a 32-second record length at 2-milisecond sampling rate with 30 separate records obtained for quality control purposes.

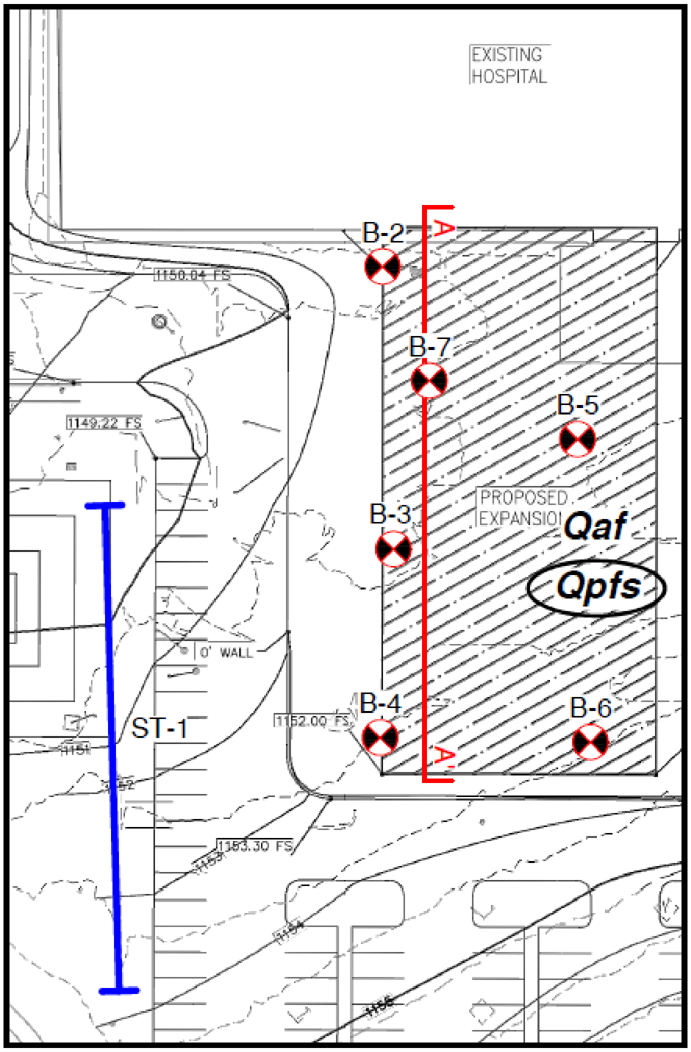

Figure 3-5. Approximate Seismic Traverse Location, ST-1

After the field data was collected, the geophysicist combined the MASW and MAM survey results using specialized software specific to this purpose. The weighted average for velocity in the upper 100 feet of the site (V_{100}) was computed from ASCE 7-16 Equation 20.4-1. The seismic model indicates that the average shear-wave velocity (weighted average) in the upper 100 feet is 1046.4 ft/sec. This average velocity classifies the underlying soils as Site Class D.

3.5 Laboratory Testing

3.5.1 General

Following completion of the fieldwork, representative samples of the subsurface soils recovered from the engineering borings were transferred to NOVA's geotechnical laboratory for testing.

An experienced geotechnical engineer classified each soil sample on the basis of texture and plasticity in accordance with the Unified Soil Classification System (USCS). The group symbols for each soil type are indicated on the boring logs. The geotechnical engineer grouped the various soil types into the major zones noted on the boring logs. The stratification lines designating the interfaces between earth materials on the boring logs and profiles are approximate; *in-situ*, the transitions may be gradual.

Representative soil samples were selected and tested in NOVA's materials laboratory to check visual classifications and to determine pertinent engineering properties. The laboratory work included visual classifications of all soil samples as well as strength and index testing on selected soil samples. Testing was performed in general accordance with ASTM standards.

Records of the geotechnical laboratory testing are presented in Appendix C.

3.5.2 Gradation

The visual classifications were supplemented by soil gradation analyses after ASTM D 6913. The results of these analyses were used to support soil classification after ASTM D2488. Table 3-4 summarizes the results of this testing.

Table 3-3. Summary of the Soil Gradation Testing

Sample Reference		**Percent Finer Than the U.S. No 200 Sieve**	**Classification after ASTM D2488**
Boring	**Depth (feet)**		
2	10-16.5	58	ML
5	2.5-4	58	ML
7	20-21.5	36	SM

Note 1: The U.S. # 200 sieve is 0.074 mm,
Note 2. Gradation testing after ASTM D6913.

3.5.3 Moisture and Density

Laboratory compaction testing was completed after ASTM D1557 on a composite sample of soil from the upper five feet of B-7. This testing indicated an optimum dry unit weight ($\gamma_{dry\ opt}$) of 131.2 lb/ft^3 at a moisture content of 8.4%.

3.5.4 *In Situ* Moisture and Density

In-situ moisture content and dry unit weight testing were performed within NOVA's laboratory. The following Table 3-4 summarizes the results of this testing.

Table 3-4. In-Situ Moisture and Density

Sample Reference		Percent Moisture	Density (pcf)
Boring	Depth (feet)		
5	5	9.9	117.1
7	15	14.7	116.5
7	25	15.9	117.2

Note 1: The U.S. # 200 sieve is 0.074 mm,
Note 2. Gradation testing after ASTM D6913.

3.5.5 Corrosivity Testing

Resistivity, sulfate content and chloride contents were determined to estimate the potential corrosivity of on-site soils. These chemical tests were performed on a representative sample of the near-surface soils by Clarkson Laboratory and Supply, Inc. Table 3-5 summarizes the results of this testing.

Table 3-5. Summary of Corrosivity Testing of the Near Surface Soil

Parameter	Units	Boring B-7, 0-5 feet depth
pH	standard unit	8.3
Resistivity	Ohm-cm	1300
Water Soluble Chloride	ppm	75
Water Soluble Sulfate	ppm	220

4.0 SITE CONDITIONS

4.1 Geologic Setting

4.1.1 Regional

The site is located within the northern portion of the Peninsular Range Geomorphic Province. This province, which stretches from the Los Angeles basin to the tip of Baja California, is characterized by a series of northwest trending mountain ranges separated by subparallel fault zones, and a coastal plain of subdued landforms. The mountain ranges are underlain primarily by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the southern California batholith. The active Elsinore fault zone, considered part of the greater San Andreas fault system, divides the Santa Ana Mountains block to the west from the Perris block to the east. In the center of this mapped area, the Murrieta Hot Springs Fault, a late Quaternary fault, not considered active, is a generally east striking major fault splay.

4.1.2 Site Specific

The property is underlain by the sandstone member of the Pauba Formation (Qpfs) of Pleistocene Age. This unit is generally composed of alluvial stream deposits with interbeds and mixtures of brownish siltstones, sandstones, and conglomerates that are moderately cemented. The Pauba Formation includes two informal members: an upper sandstone member (Qpfs) consisting of brown, moderately well-indurated, cross-bedded sandstone with sparse cobble to boulder conglomerate interbeds; and a lower fanglomerate member (Qpf) consisting of grayish brown, well-indurated, poorly sorted fanglomerate and mudstone. According to Kennedy and Morton, 2003, only the upper sandstone member is exposed near the site. Figure 4-1 reproduces mapping that depicts the area geology.

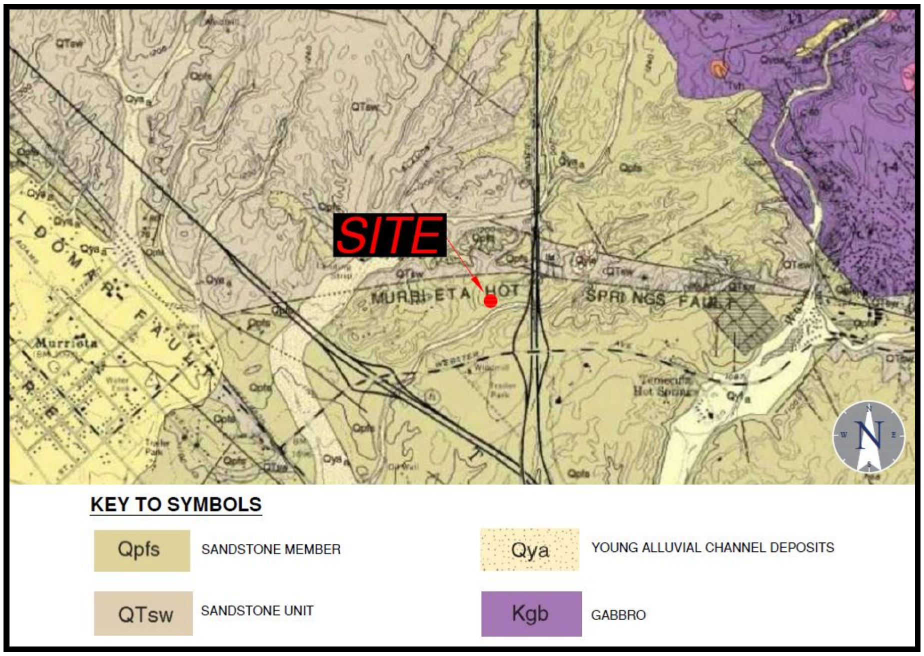

Figure 4-1. Geologic Map of the Site Area
(source: adapted from CGS 2007)

4.1.3 Faulting

There are no known active faults underlying the property. The nearest mapped active fault zone is the Elsinore-Temecula fault zone, which lies about 1.1 miles to the west of the project location. This vertical strike-slip fault has the potential to generate an earthquake with a maximum magnitude of 7.7 (USGS, Unified Hazard Tool) with peak ground accelerations (PGA) of 0.77g. Immediately north of the site lies the Murrieta Hot Springs fault. This well constrained late Quaternary fault is mapped as a discontinuous linear fault zone striking east-west between the Willard and Wildomar Faults along the southwestern side of the valley. This fault is considered to be potentially active and thus, not classified as an Alquist-Priolo (AP) Earthquake Fault Zone. An active fault is defined by the State of California as having surface displacement within the past 11,700 years or during the Holocene geologic time period. Figure 4-2 shows the AP fault zone hazard map of the site vicinity.

4.1.4 Seismic Hazard Mapping

Seismic hazard mapping developed by the California Geological Survey indicates the site is not located in an area at risk for liquefaction in the event of a severe seismic event. Liquefaction refers to the loss of soil strength and related subsidence that occurs when saturated (i.e., below the water table), predominately sandy soils are subject to earthquake shaking.

Figure 4-2 reproduces the AP Zone liquefaction hazard mapping of the site area. Section 5 of this report provides detailed evaluation of this risk. Figure 4-3 reproduces the faults mapped in the region of the site area. Active faults are indicated in orange and late Quaternary (potentially active) faults are in green.

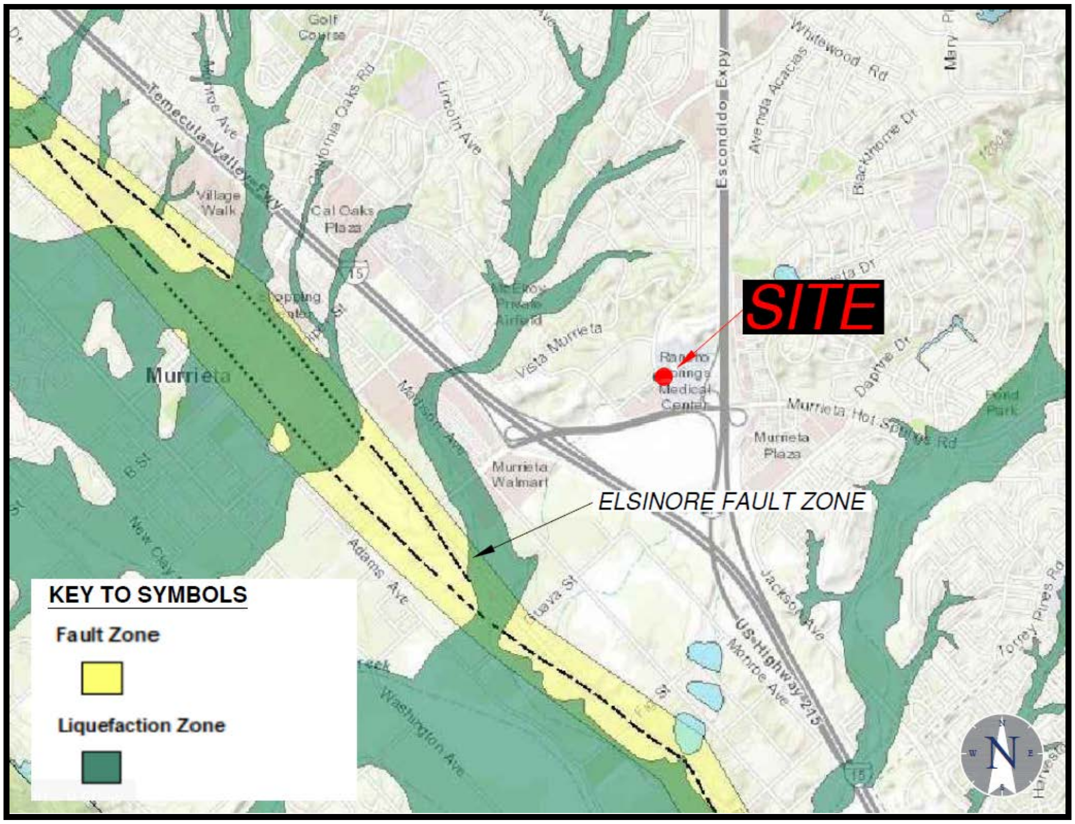

Figure 4-2. Geologic Hazard Mapping of the Site Area
(Source: *California Geological Survey AP Zone, Murrieta Quadrangle*, Jan. 11, 2018)

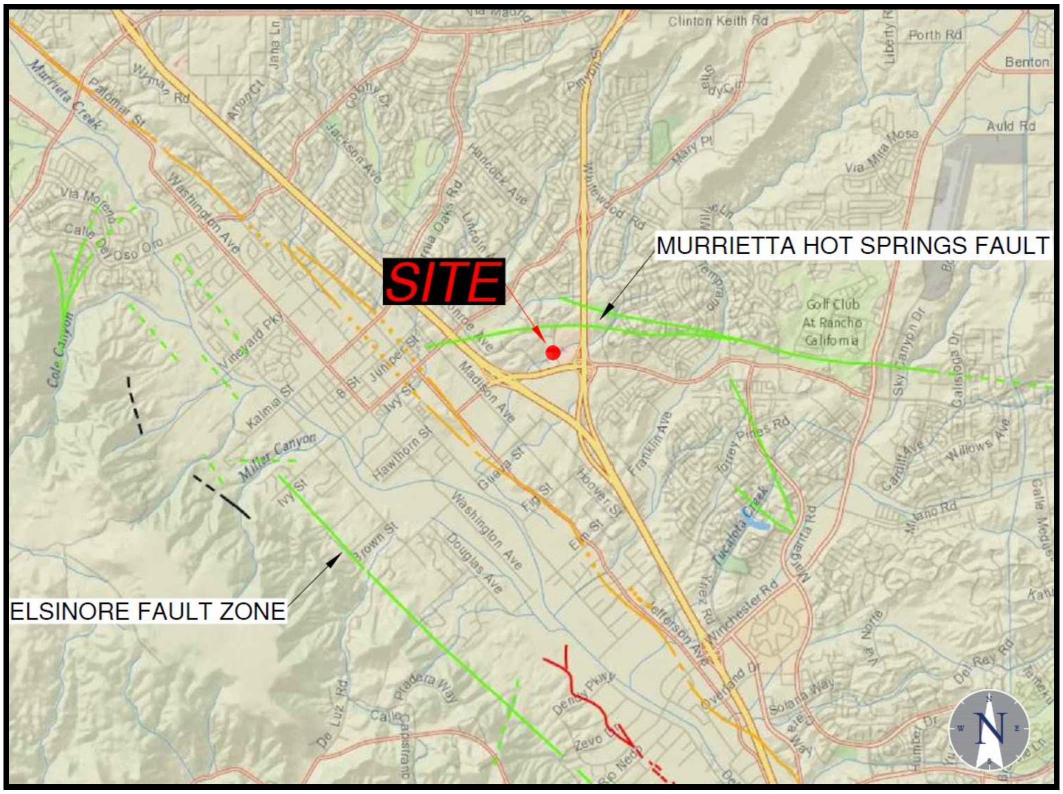

Figure 4-3. Regional Fault Map Site Area

(Source: U.S. Quaternary Faults, 2014, USGS Geologic Hazards Science Center, Elsinore [Temecula])

4.2 Site Conditions

4.2.1 Surface

As discussed in Section 2, the site is currently used for drive ways, parking areas, and landscaping improvements. The ground surface across the site is relatively level, with surface drainage flowing from the eastern edge of the campus at an elevation of approximately +1154 feet MSL westward toward Handcock Avenue with an elevation of approximately +1138 feet MSL.

4.2.2 Subsurface

For the purposes of this report, the sequence of soils that underlie the site may be described as follows.

- Unit 1, Fill (Qaf). The upper approximately 3 feet to 13 feet of the subsurface is fill comprised of silty and clayey sand and sandy silt. The fill was found to be in a damp to moist and in a loose to dense condition.
- Unit 2, Pauba Formation (Qpfs). Light to dark brown sandstone of the Pauba Formation was encountered below the overlying fill materials. The Pauba Formation was found to be medium dense to very dense.

4.2.3 Groundwater

Groundwater was not encountered during NOVA's subsurface investigation to a depth of 50 feet bgs. In reviewing historical groundwater levels in the site vicinity, it was found that within a well located about 0.25 miles from the site (State Well Number 335545N1171801W001), groundwater was at a depth of 124 feet bgs (elevation +1043 feet MSL) as measured in 1968 (California Department of Water Resources website). Monitoring wells for the Shell Service station #121641 (T0606553648), approximately 0.75 miles west of the site, show groundwater at an elevation of +1084 feet MSL (approximately 66 feet below finished grade of proposed building) as measured in 2009 (GeoTracker website).

4.2.4 Surface Water

No surface water was evident on the site at the time of NOVA's work. There was no evidence of springs, seeps, surface erosion, or staining that would indicate historic or current problems with surface water.

5.0 REVIEW OF GEOLOGIC, SOIL AND SITING HAZARDS

5.1 General

This section provides a review of soil, geologic and siting-related hazards common to this region of California, considering each for its potential to affect the planned facility. The primary hazards identified by this review are abstracted below.

1. Strong Ground Motion. 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. The expectation of strong ground motion is common to all civil works in this area of California.

2. Liquefaction. Strong ground motion associated with a large magnitude earthquake will affect the site; however, the subsurface consists of a relatively thin layer of fill underlain by dense/stiff soil of the Pauba Formation (Qpfs). Liquefaction concerns are considered negligible.

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

5.2 Geologic Hazards

5.2.1 Strong Ground Motion

The site is not located within a currently designated Alquist-Priolo Earthquake Zone (CGS, 2018). No known active faults are mapped on the site.

The nearest known active faults are two major strands of the Temecula segment of the Elsinore Fault Zone. These two strands are located approximately 1.1 and 1.2 miles to the west of the subject site at their closest points. The Elsinore Fault system has the potential to be a source of strong ground motion, generating an earthquake of Richter magnitude (M) of about M = 7.7, with a risk-based peak ground acceleration (PGA) of $PGA_G = 0.77g$.

5.2.2 Fault Rupture

As noted above there are no known active faults mapped at the subject property and the property is not located within an Alquist-Priolo earthquake fault zone. NOVA's site reconnaissance did not present any indications of active faulting. In consideration of these findings, NOVA does not consider the potential for onsite surface rupture from a seismic event a significant hazard.

5.2.3 Landslide

As used herein, 'landslide' describes downslope displacement of a mass of rock, soil, and/or debris by sliding, flowing, or falling. Such mass earth movements are generally greater than 10 feet thick and larger than 100 feet across. Landslides typically include cohesive block glides and disrupted slumps that are formed by translation or rotation of the slope materials along one or more slip surfaces.

The causes of classic landslides start with a preexisting condition- characteristically a plane of weak soil or rock inherent within the rock or soil mass. Thereafter, movement may be precipitated by earthquakes, wet weather, and changes to the structure or loading conditions on a slope (e.g., by erosion, cutting, filling, release of water from broken pipes, etc.). The site is set in nearly flat area such that NOVA considers the landslide hazard to be negligible for the site.

5.3 Soil Hazards

5.3.1 Liquefaction

General

"Liquefaction" refers to the loss of soil strength during a seismic event. The phenomenon is observed in geologically 'young' soils that include a shallow water table and coarse grained (i.e., 'sandy') soils of loose to medium dense consistency. 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.

As is discussed in Section 4.1, the site is not mapped in an area that is at risk for liquefaction, and based on bedrock density and low groundwater levels, the potential for liquefaction-induced settlement is low.

5.3.2 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.

In consideration of the largely sandy soils that comprise the subsurface at this site, as supported by the index testing provided in Section 3, the potential for problems associated with soil expansivity is low. Surface reconnaissance and the subsurface investigation did not reveal the presence of potentially expansive soils that could affect development. Based on visual observation soils are not considered to be expansive.

5.3.3 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 permanent slopes are planned as part of the proposed development. There is no risk of embankment instability for permanent construction. Section 7 provides guidance for management of the stability of temporary embankments and excavations during construction.

5.3.4 Collapsible Soils

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

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

Based upon the indications of the blow counts collected during our subsurface investigation, the site soils are not at risk for hydro-collapse.

5.3.5 Corrosive Soils

Chemical testing of the near surface soils indicates the soils contain low concentrations of soluble sulfates and chlorides. The tested soils will be corrosive to embedded metals, but not to embedded concrete. Section 7 addresses this consideration in more detail.

5.4 Siting Hazards

5.4.1 Effect on Adjacent Properties

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

5.4.2 Flood

The site is located within a flood zone designated as Flood "Zone X" (FEMA, Map 06065C2720G, effective 08/28/08). 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.*" This is an area of minimal flood hazard. Figure 5-1 reproduces flood mapping of the site area.

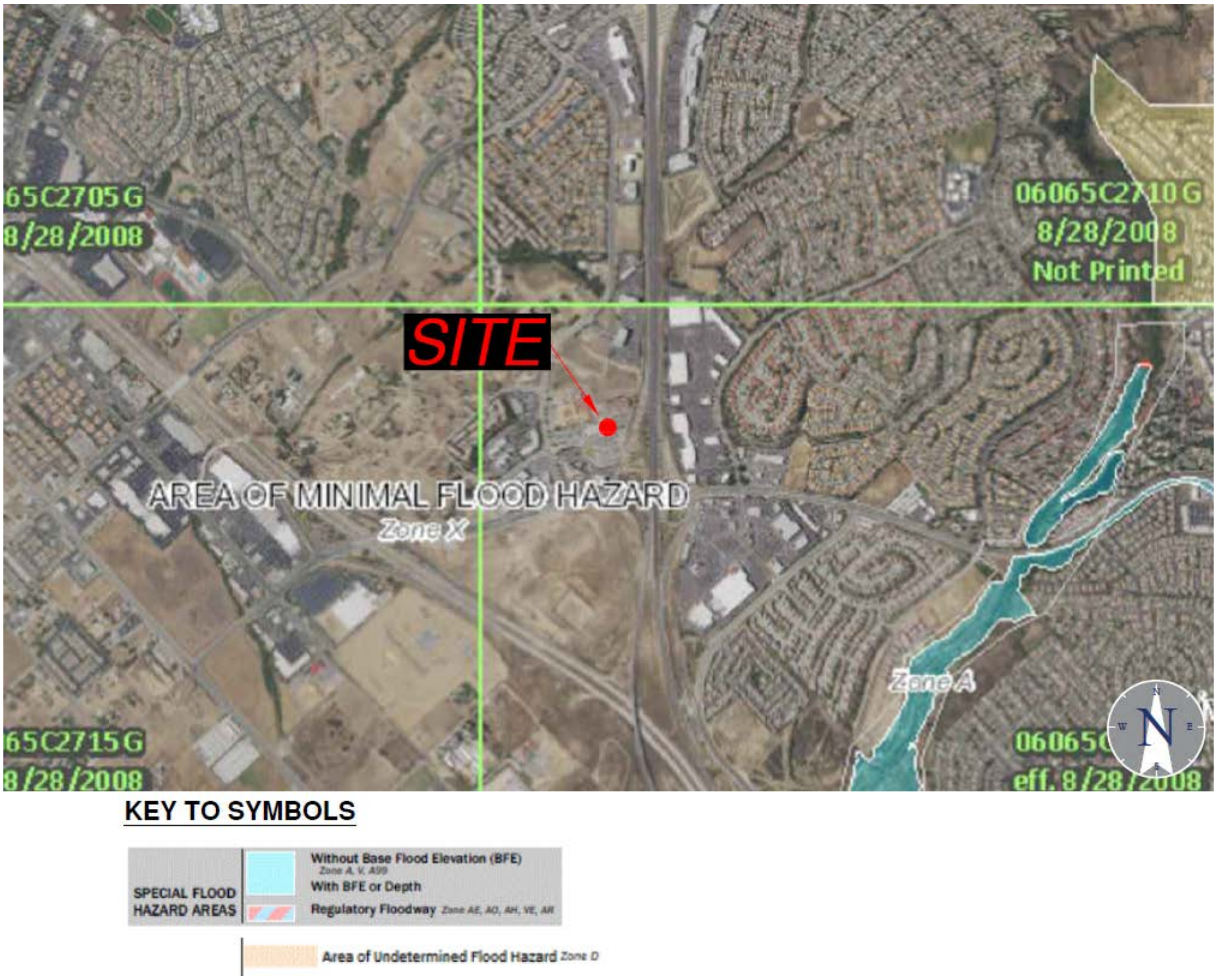

Figure 5-1. Flood Mapping of the Site Area
(Source: FEMA, Map 06065C2720G, effective 08/28/2008)

6.0 SEISMIC DESIGN PARAMETERS

6.1 Background

It has been known for some time[1] that seismic design parameters determined as described in ASCE 7-10 Chapter 11 have the potential to underestimate potential accelerations for structures founded on Site Classes D, E, and F. The recent code update of Sections 11 and 21 in ASCE 7-16 is intended to mitigate these shortcomings by requiring a Site-Specific Hazard Analysis (SSHA). The SSHA is used for quantitative estimations of ground motion characteristics at a specific site by incorporating several site-specific variables, principally distance from fault, site shear wave velocity, and fault geometry.

The SSHA includes the following principal elements of analyses and evaluation:

- field determination of the site class of the subject site,
- Probabilistic Seismic Hazard Analysis (PSHA);
- Deterministic Seismic Hazard Analysis (DSHA); and,
- determining design acceleration parameters using the resulting acceleration spectra.

The PSHA allows the uncertainties in size, location, and rate of recurrence of earthquakes and the variation of ground motion characteristics to be considered in the seismic evaluation. A DSHA involves development of an evaluation of ground motion hazard at a site based on a scenario in which an earthquake of a specified size occurring at a specified location occurs. This procedure provides a framework for evaluated worst-case ground motions (Kramer 1996).

NOVA has completed a SSHA for the subject site in accordance with CBC 2019 and ASCE 7-16. This report provides a summary of NOVA's procedure and results of the SSHA for the site

6.2 Procedure

6.2.1 Site Classification

A seismic shear wave survey of the subject site was completed on November 2, 2019. The objective of this survey was to determine the site class based on shear wave velocities of the upper 30 meters of subsurface, referred to as either V_{s30} or V_{100}.

The measured shear wave velocities were found to average 1,046.4 feet/second. Results of this analysis are shown in Figure 6-1. Using Table 20.3-1 from ASCE 7-16, the site is determined to be Site Class D.

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[1] For example, see Kircher, C. A., *New Site-Specific Ground Motion Requirements of ASCE 7-16*, Proceedings, SEAOC Convention, 2017

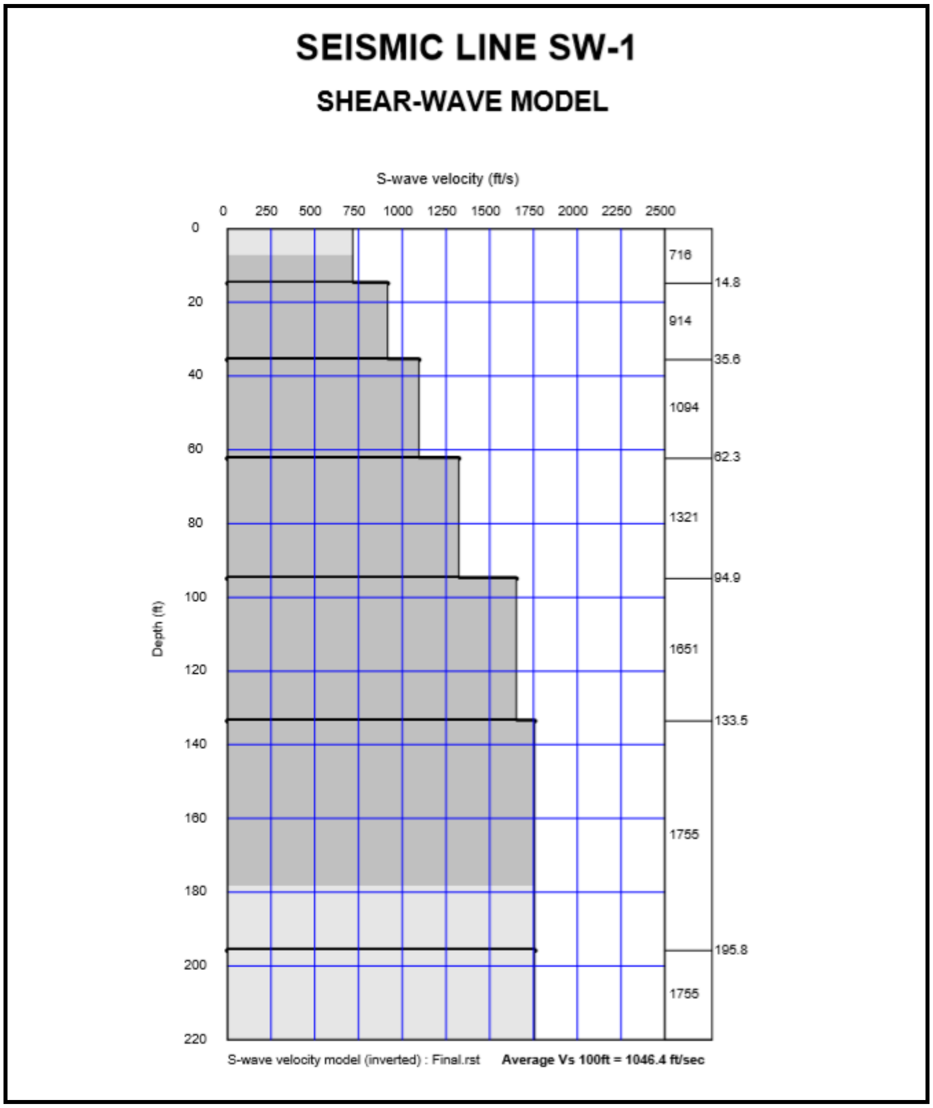

Figure 6-1. Shear Wave Velocities for the Subject Site

6.2.2 Probabilistic Hazard Analysis

The Probabilistic Seismic Hazard Analysis (PSHA) was completed using tools provided by USGS for this purpose. Site-specific parameters including site location by latitude and longitude, site class, and probability of an earthquake with 2% exceedance in 50 years were input into the Unified Hazard Tool. Peak Ground Acceleration was selected for Spectral Period and the default Time Horizon of 2,475 years was used. The earthquake fault dataset selected for the calculations performed by the tool was Dynamic: Conterminous U.S. 2014 (update) (v4.2.0) Edition. Calculations provided spectral acceleration values for periods between 0 and 5 seconds.

The computed values were then input into the USGS Risk-Targeted Ground Motion (RTGM) Calculator and recorded at periods shown in Table 1. Maximum Direction Scale Factors were then determined using those specified in Section 21.2 for varying periods. Maximum Direction RTGM was then calculated as the product of RTGM and the Maximum Direction Scale Factor as shown in Table 6-1.

Table 6-1. Probabilistic Seismic Hazard Analysis Values

Period (s)	Risk Targeted GM (g)	Max Dir Scale Factor	Max Direction RTGM (g)
0	0.69	1.1	0.76
0.1	1.18	1.1	1.30
0.2	1.58	1.1	1.74
0.3	1.74	1.125	1.96
0.5	1.66	1.175	1.95
0.75	1.36	1.2375	1.68
1	1.13	1.3	1.47
2	0.62	1.35	0.84
3	0.41	1.4	0.57
4	0.29	1.45	0.42
5	0.22	1.5	0.33

Figure 6-2 provides the probabilistic site response based on the method described above.

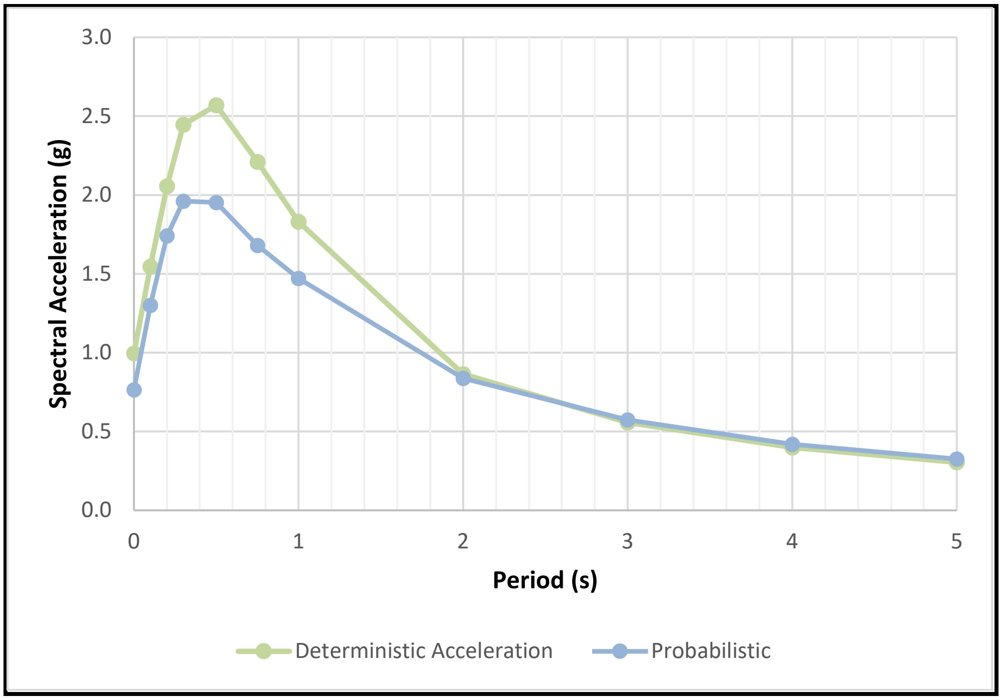

Figure 6-2. Probabilistic and Deterministic Seismic Accelerations

6.2.3 Deterministic Hazard Analysis

For the Deterministic Seismic Hazard Analysis (DSHA) the nearest active fault to the site was located using the USGS KML fault database overlain on Google Earth. Other active faults in the region were evaluated to ensure the correct controlling fault was used in the analysis. The nearest active fault is the Temecula section of the Elsinore Fault Zone at an approximate location of 1.69 km from the site.

The PEER NGA-West2 Excel file with 5 models calculating horizontal ground motion was used in the DSHA. The file provides the weighted average of peak values and the response spectra of the NGA-West2 horizontal ground motion prediction equations. NOVA used four of the five available models in the evaluation. The following four were weighted at 25% contribution: Abrahamson et al., Boor et al., Campbell and Bozorgnia, and Chiou and Youngs.

Site-specific parameters inputted into this spreadsheet were retrieved from the USGS Fault Section Data Database (USGS 2013) including Lower Seismic Depth and Dip Angle, and Earthquake Magnitude. Shear wave velocity at the upper 30 m (V_{S30}) determined from the site seismic shear-wave survey was also input into the model. The deterministic spectral response acceleration at each period was calculated as an 84th percentile 5% damped spectral response acceleration. These values were multiplied by the same Maximum Direction Scale Factors applied in the PSHA to produce the Maximum Direction Deterministic Spectral Accelerations. For simplicity of data presentation, the same periods were selected as those of the PSHA.

The values for the deterministic accelerations are shown in Table 6-2. Figure 6-2 depicts the Deterministic and Probabilistic curves graphically. Per Section 21.2.3, the MCE_R is taken as the lesser of the spectral response accelerations from the PSHA and the DSHA; and therefore, the PSHA accelerations control for this site specific analysis.

Table 6-2. Deterministic Seismic Hazard Analysis Values

Period (s)	84th Percentile 5% Dampening	Max Dir Scale Factor	Max Direction Deterministic Spectral Acceleration (g)
0	0.91	1.1	1.00
0.1	1.41	1.1	1.55
0.2	1.87	1.1	2.06
0.3	2.17	1.125	2.45
0.5	2.19	1.175	2.57
0.75	1.79	1.2375	2.21
1	1.41	1.3	1.83
2	0.64	1.35	0.86
3	0.40	1.4	0.56
4	0.27	1.45	0.40
5	0.20	1.5	0.30

6.2.4 Design Response Spectrum

Per ASCE 7-16 Section 21.3, the spectral response calculated above, shall not be less than 80% of those determined in accordance with Section 11.4.6. Figure 6-3 presents the 80% design response spectrum and the results of the controlling PSHA curves, which confirms site-specific accelerations exceed the 80% design response at all periods.

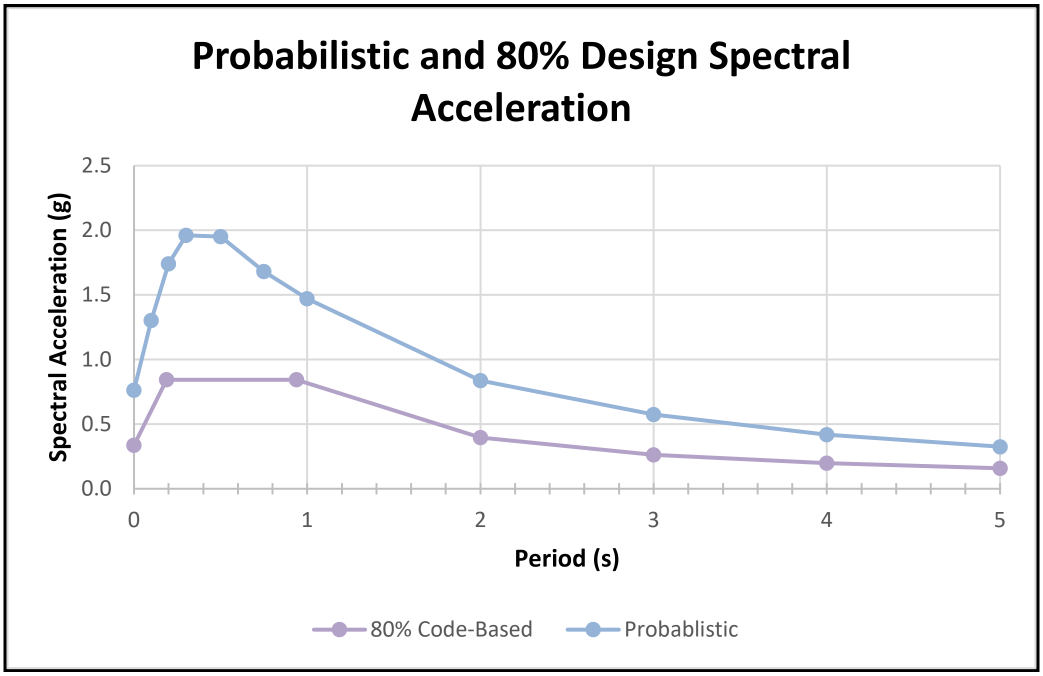

Figure 6-3. Probabilistic Spectral Accelerations and 80% Design Spectral Accelerations

6.2.5 Design Acceleration Parameters

Following Section 21.4 of ASCE 7-16, S_{DS} was taken as 90% of the maximum spectral acceleration (S_a) from the PSHA over the periods 0.2s to 5s. S_{D1} was taken as the maximum product value of period and spectral acceleration for the period, calculated over the periods 1s through 5s. The parameters S_{MS} and S_{M1} were calculated as 1.5 times S_{DS} and S_{D1}, respectively.

The values calculated were confirmed not to be less than 80% of the values determined in accordance with Section 11.4.3 of ASCE 7-16 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}. The calculated values of S_{DS}, S_{D1}, S_{MS}, and S_{M1} are shown in Table 6-3 (following page).

Table 6-3. Calculated and Code Based Design Acceleration Parameters

Parameter	Calculated	OSHPD F_a=1.0 F_v =1.7*
S_{MS}	2.65	1.58
S_{M1}	2.58	1.00
S_{DS}	1.76	1.05
S_{D1}	1.72	0.67
S_S	1.58	1.58
S_1	0.59	0.59

*F_a value taken from Table 11.4-1 (confirmed Site Class D)
F_v value taken from Table 11.4-2

6.2.6 Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration

The probabilistic peak ground acceleration was determined according to Section 21.5.1 using the Risk Targeting Ground Motion Tool for the Unified Hazard Ground Motion at a period of 0s. This calculator presents the geometric mean peak ground acceleration with a 2% probability of exceedance within a 50-year period. The resulting acceleration is 0.73g

The deterministic peak ground acceleration was determined according to Section 21.5.2 and calculated as the largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region. PGA was calculated as the point in the DSHA where the period is equal to 0s, resulting in spectral acceleration of 0.99 g.

The site-specific peak ground acceleration (PGA_M) was taken as the lesser of the probabilistic and deterministic peak ground accelerations. In accordance with code, it was confirmed that PGA_M was not taken as less than 80% of PGA_M determined from Eq. 11.8-1.

Table 6-4. Calculated and Code Based MCE_G Peak Ground Acceleration

Parameter	Calculated	OSHPD	80% OSHPD
MCE_G PGA	0.73	0.77	0.62

6.2.7 Exceptions to Site-Specific Hazard Analysis

Per Section 11.4.8 Exception 2, a SSHA is not required for structures in which the Structural Engineer will be using the Equivalent Lateral Force (ELF) procedure, which is common for buildings with short fundamental periods. If the ELF procedure is used, the seismic parameters may be calculated by using the F_a and F_v coefficients in Tables 11.4-1 and -2 (parameters shown under the OSHPD heading within Tables 6-3 and 6-4 of this report).

7.0 EARTHWORK AND FOUNDATIONS

7.1 Overview

7.1.1 Review of Site Hazards

Section 5 provides a review of soil and geologic hazards common to development of civil works in the project area. The primary hazards identified by that review are abstracted below.

1. Strong Ground Motion. 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. The expectation of strong ground motion is common to all civil works in this area of California. Section 6 addresses seismic design parameters

2. Liquefaction. Strong ground motion associated with a large magnitude earthquake will effect some liquefaction and related ground settlement. However, ground movements will be small- about 1 inch or less- and will not threaten the integrity of the planned structure. With this consideration, the site is suitable for development of the facility on shallow foundations. Section 7.5 addresses design parameters for shallow foundations.

7.1.2 Site Suitability

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

7.1.3 Review and Surveillance

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

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

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

7.2 Corrosivity and Sulfates

7.2.1 Corrosivity

Electrical resistivity, chloride content, sulfate contents and pH level are all indicators of a soil's tendency to corrode/attack metals and concrete. Chemical testing was performed on representative samples of soils from the site. The results of the testing are tabulated on the following Table 7-2.

Table 7-1. Summary of Corrosivity Testing of the Unit 1 Soil

Parameter	Units	Boring B-7, 0-5 feet
pH	standard unit	8.3
Resistivity	Ohm-cm	1300
Water Soluble Chloride	ppm	75
Water Soluble Sulfate	ppm	220

7.2.2 Metals

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

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

Based on the Caltrans criteria, the on-site soils would not be considered corrosive to buried metals. Records of this testing are provided in Appendix C. These records include estimates of the life expectancy of buried metal culverts of varying gauge.

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

Table 7-2. 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 summarized on Table 7-2 suggests that design should consider that the soils may be corrosive to embedded metals. Typical recommendations for mitigation of such corrosion potential in embedded ferrous metals include:

- a high quality protective coating such as an 18 mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar;

- electrical isolation from above grade ferrous metals and other dissimilar metals by means of dielectric fittings in utilities and exposed metal structures breaking grade; and,

- steel and wire reinforcement within concrete having contact with the site soils should have at least 2 inches of concrete cover.

If extremely sensitive ferrous metals are expected 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

7.2.3 Sulfate Attack

As shown on Table 7-2, the soil sample tested indicated water-soluble sulfate (SO_4) content of the soils are 0.02 percent by weight. With $SO_4 < 0.10$ percent by weight, the American Concrete Institute (ACI) publication ACI 318-08 considers a soil to have no potential (S0) for sulfate attack. Table 7-4 reproduces the sulfate Exposure Categories considered by ACI.

Table 7-3. Exposure Categories and Requirements for Water-Soluble Sulfates

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

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

7.2.4 Limitations

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

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

7.3 Earthwork

7.3.1 General

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

7.3.2 Compaction

All fill and backfill should be compacted to a minimum of 90% relative compaction after ASTM D1557 (the 'modified Proctor') following moisture conditioning to 2% above the optimum moisture content. Fill placed in loose lifts no thicker than the ability of the compaction equipment to thoroughly densify the lift. For most construction equipment, this limit loose lifts to on the order of 10-inches or less.

7.3.3 Select Fill

Any engineered fill should be Select Fill; i.e., soil with at least 40 percent of the material less than ¼-inches in size, a maximum particle size of 1 inch, with an expansion index ('EI', after ASTM D 4829) of EI < 20. Select Fill should not include fibrous organic, perishable, spongy, deleterious, environmentally affected, or otherwise unsuitable material. The sandy Unit 1 soils will be suitable for use as Select Fill. If a detention pond is developed on site, this feature may be a good source of Select Fill.

7.3.4 Site Preparation and Remedial Grading

Any abandoned utilities should be removed and properly disposed off-site before the start of excavation operations. The area planned for structures and pavements should be cleared of vegetative material, including the root zone. Thereafter, remedial grading to improve and proof the quality of the Unit 1 fill should be undertaken in the step-wise manner described below.

1. Step 1, Excavation/Densification. Due to loose material encountered in the borings in the near-surface, remedial removals shall extend a minimum of 5 feet below existing ground surface within the limits of planned hospital expansion structure. This material should be excavated and staged for later replacement. Removals for areas receiving pavements should extend to at least 2 feet below existing or proposed grade, whichever is deeper. A NOVA representative should observe all excavation bottoms after removals. Deeper excavation may be necessary in localized areas. Laterally, removals should extend outward at least 5 feet and 2 feet for of the proposed structure and pavements, respectively.

   Removals directly adjacent to existing structures should be performed by slot cutting such that the existing improvements and existing foundations are not completely exposed. Existing foundations should in no case be undermined. A NOVA representative should observe the grading near existing improvements during the removal operation.

   The ground surface disturbed by this excavation should be densified to at 90% relative compaction after ASTM D1557 (the 'modified Proctor') following moisture conditioning to 2% above the optimum moisture content.

2. Step 2, Proof-Rolling. After the completion of compaction/densification of the excavated surface, the area should be proof-rolled. A loaded dump truck or similar should be used to aid in identifying localized soft or unsuitable material. Any soft or unsuitable materials encountered during this proof-rolling should be removed, replaced with an approved backfill, and compacted.

3. Step 3, Replacement. The soil excavated by Step 2 should be replaced in conformance with the criteria identified in Section 7.4.2 and Section 7.4.3.

7.3.5 New Fill

New fill to establish site grades should be placed in conformance with the criteria identified in Section 7.4.2 and Section 7.4.3.

Shallow foundations should be constructed as soon as possible following subgrade approval. The Contractor should be responsible for maintaining the subgrade in its approved condition (i.e., at the compacted moisture content, frees of disturbance, etc.) until foundations are constructed.

7.3.6 Trenching and Backfilling for Utilities

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

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

Backfill for utility trenches is as important as the original subgrade preparation or engineered fill placed to support either a foundation or slab. Backfill for utility trenches must be placed to meet the project specifications for the engineered fill of this project. Unless otherwise specified, the backfill for the utility trenches should be placed in 4 to 6-inch loose lifts and compacted to a minimum of 90 percent relative compaction after ASTM D1557 (the ‘modified Proctor’) at soil moisture +2 percent of the optimum moisture content. Up to 4 inches of bedding material placed directly under the pipes or conduits placed in the utility trench can be compacted to 90 percent relative compaction with respect to the Modified Proctor.

7.3.7 Flatwork

Prior to casting exterior flatwork, the upper one foot of subgrade soils- either Unit 1 sands or Select Fill- should be moisture conditioned densified as recommended in Section 7.4.2. Concrete slabs for pedestrian traffic or landscaping should be at least four (4) inches thick.

7.4 Shallow Foundations

7.4.1 Isolated and Continuous Foundations

Unit 1 fill improved as described in Section 7.4 and any new fill placed as described in Section 6.4 may be used to support isolated and continuous footings, as described below.

Isolated Foundations
Isolated foundations for interior columns may be designed for an allowable contact stress of 3,000 psf for dead and commonly applied live loads (DL+LL). These foundation units should have a minimum width of 30 inches, embedded a minimum of 24 inches below surrounding grade. This bearing value may be increased by one-third for transient loads such as wind and seismic.

Continuous Foundations
Continuous foundations may be designed for an allowable contact stress of 2,500 psf for dead and commonly applied live loads (DL+LL). These footings must be a minimum of 18 inches in width and embedded a minimum of 24 inches below surrounding grade. This bearing value may be increased by one-third for transient loads such as wind and seismic.

Resistance to Lateral Loads
Lateral loads to shallow foundations may be resisted by passive earth pressure against the face of the footing, calculated as a fluid density of 200 psf per foot of depth, neglecting the upper 1 foot of soil below surrounding grade in this calculation. Additionally, a coefficient of friction of 0.30 between soil and the concrete base of the footing may be used with dead loads.

Settlement

Supported as recommended above, the structure will settle on the order of 0.2 inch. This movement will occur elastically, as dead load (DL) and permanent live loads (LL) are applied. In usual circumstance, about 50% of this settlement will occur during the construction period. Angular distortion due to differential settlement of adjacent, unevenly loaded footings should be less than 1 inch in 40 feet (i.e., Δ/L less than 1:480).

7.4.2 Ground Supported Slabs

The ground level of the planned facility may employ a conventional on-grade (ground-supported) slab designed using a modulus of subgrade reaction (k) of 150 pounds per cubic inch (i.e., k = 150 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 #4 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 must be established to a depth of ¼ the slab thickness as depicted in Figure 7-1.

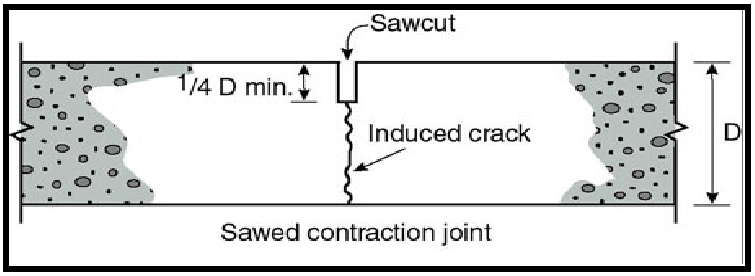

Figure 7-1. Sawed Contraction Joint

7.5 Capillary Break and Underslab Vapor Retarder

7.5.1 Capillary Break

The requirements for a capillary break (‘sand layer’) beneath the ground supported slab should be determined in accordance with ACI Publication 302 “*Guide for Concrete Floor and Slab Construction*.”

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

7.5.2 Vapor Retarder

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.

Products

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

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

- ASTM E1745-97 (2009). *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs.*
- ASTM E154-88 (2005). *Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Walls, or as Ground Cover.*
- ASTM E96-95 (2005). *Standard Test Methods for Water Vapor Transmission of Materials.*
- ASTM E1643-98 (2009). *Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs.*
- ACI 302.2R-06. *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials.*

Vapor retarders employed for ground supported slabs are commonly specified as minimum 15 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.

Installation

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

7.6 Control of Moisture Around Foundations

7.6.1 Erosion and Moisture Control During Construction

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

7.6.2 Design

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

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

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

7.7 Retaining Walls

7.7.1 Lateral Pressures

Lateral earth pressures for retaining walls are related to the type of backfill, drainage conditions, slope of the backfill surface, and the allowable rotation of the wall. Table 7-5 provides recommendations for lateral soil for retaining walls with level backfill for varying conditions of wall yield.

Table 7-4. Lateral Earth Pressures to Retaining Walls

Condition	**Equivalent Fluid Pressure (psf/foot) for Approved Backfill** Notes A, B	
	Level Backfill	**2:1 Backfill Sloping Upwards**
Active	35	55
At Rest	55	80
Passive	250	300

Note A: site-sourced Unit 1 sands or similar imported soil.

Note B: assumes wall includes appropriate drainage and no hydrostatic pressure.

If footings or other surcharge loads are located a short distance outside the wall, these influences should be added to the lateral stress considered in the design of the wall. Surcharge loading should consider wall loads that may develop from adjacent streets and sidewalks. To account for such potential loads, a surcharge pressure of 75 psf can be applied uniformly over the wall to a depth of about 12 feet.

7.7.2 Seismic Increment

The seismic load increment should be calculated as a uniform 22H psf (with H the height of the wall in feet).

7.7.3 Drainage

Design for retaining walls should include drainage to limit accumulation of water behind the wall. Figure 7-2 provides guidance for such design. Note that the guidance provided on Figure 7-2 is conceptual. A variety of options are available to drain permanent below grade walls.

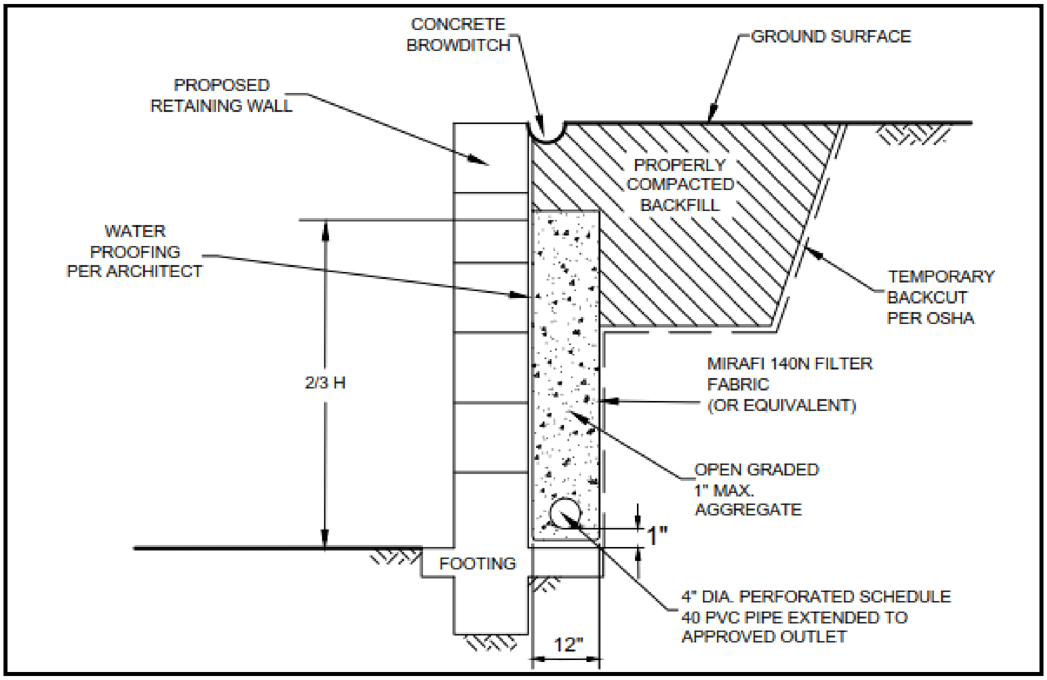

Figure 7-2. Conceptual Design for Retaining Wall Drainage

7.7.4 Elevator Pits

Elevators will likely be included within the projects final design. Elevators may require pits that extend below the lowest slab level. An elevator pit slab and related retaining wall footings will derive suitable support from the Unit 2 sandstones 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.

7.8 Temporary Slopes

Any temporary slopes should be made in conformance with OSHA requirements. All temporary excavations should comply with local safety ordinances, as well all Occupational Safety and Health Administration (OSHA) requirements, as applied to California. These requirements may be found at http://www.dir.ca.gov/title8/sb4a6.html.

8.0 STORMWATER INFILTRATION

8.1 Overview

Based upon the indications of the field exploration and laboratory testing reported herein, NOVA has evaluated the site as abstracted below after guidance contained in *Riverside County, Santa Margarita River Watershed Region Design Handbook for Low Impact Development, Best Management Practices*, Riverside County Flood Control and Water Conservation District, Revised June 2018 (hereafter, 'the BMP Manual').

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

As is well-established in the BMP Manual, the feasibility of stormwater infiltration is principally dependent on geotechnical and hydrogeologic conditions at the project site. In consideration of the measured infiltration rates at this site, NOVA concludes that the site is feasible for development of "partial infiltration" permanent stormwater infiltration BMPs.

8.2 Infiltration Rates

8.2.1 General

The percolation rate of a soil profile is not the same as its infiltration rate ('I'). Therefore, the measured/calculated field percolation rate (see Table 3-3) 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 (feet, msl)	Depth of Test (feet)	Approximate Test Elevation (feet, msl)	Infiltration Rate (inches/hour)	Design Infiltration Rate (in/hour, F=3*)
P-1*	± 1,147	15.0	± 1,132	2.64*	0.88*
P-2	± 1,148	10.0	± 1,138	0.33	0.11
P-3	± 1,148	10.0	± 1,138	0.46	0.15
P-4	± 1,147	11.0	± 1,136	0.80	0.27
P-5	± 1,147	10.0	± 1,137	0.37	0.12

Notes: (1) 'F' indicates 'Factor of Safety' (2) elevations are approximate and should be reviewed.
* P-1 was inferred to be within an existing utility trench resulting in erroneous rates.

8.2.2 Design Infiltration Rate

As may be seen by review of Table 8-1, in consideration of the nature and variability of subsurface materials, as well as the natural tendency of infiltration structures to become less efficient with time, the infiltration rates measured in the testing should be modified to use at least a factor of safety (F) of F=3 for preliminary design purposes.

8.3 Review of Geotechnical Feasibility Criteria

8.3.1 Overview

It is common that seven factors be considered by the project geotechnical professional while assessing the feasibility of infiltration related to geotechnical conditions. These factors are:

1) Soil and Geologic Conditions
2) Settlement and Volume Change
3) Slope Stability
4) Utility Considerations
5) Groundwater Mounding
6) Retaining Walls and Foundations
7) Other Factors

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

8.3.2 Soil and Conditions

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

- Unit 1, Fill. The upper 1 to 13 feet of the subsurface is predominantly silty sandy fill characteristic of a relatively dense sand.
- Unit 2, Pauba Formation. Light to dark brown sandstone/siltstone of the Pauba Formation was encountered below the overlying fill materials occurs from about 3 feet depth to a 13 feet bgs.

8.3.3 Settlement and Volume Change

The sandy Unit 1 soils have very low expansion potential. These soils will not be prone to swelling upon wetting. These soils will not be prone to hydro-collapse on wetting.

8.3.4 Slope Stability

BMPs will not be located near slopes. There are no material slopes on site, nor are any planned.

8.3.5 Utilities

Infiltration can potentially damage subsurface and underground utilities. BMPs should be sited a minimum of 10 feet away from underground utilities. The locations at our percolation borings are located within 10 feet of an existing utility line. Infiltration testing from percolation boring P-1 results in erroneous results and it was determined that a nearby utility trench was located within the area. It is recommended to located drainage management areas (DMAs) at least 10 feet away from utilities. Where DMAs are located near utility lines, it is recommended to line the sidewalls of DMA systems with an impermeable liner.

8.3.6 Groundwater Mounding

Stormwater infiltration can result in groundwater mounding during wet periods, affecting utilities, pavements, flat work, and foundations.

8.3.7 Retaining Walls and Foundations

BMPs should not be located near foundations. BMPs should be sited a minimum of 25 feet away from any foundations or retaining walls.

8.3.8 Other Factors

The location at P-1 is near an existing private storm drain line. For this reason, the infiltration rate at this location should not be considered as representative of the site.

8.4 Suitability of the Site for Stormwater Infiltration

In consideration of the known geology of the site and the indications of the site-specific testing, the site allows for partial infiltration. Stormwater DMAs should be located away from existing utility lines.

9.0 PAVEMENTS

9.1 Overview

9.1.1 General

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

9.1.2 Design to Limit Infiltration

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

An important consideration in the design and construction of pavements is surface and subsurface drainage. Where standing water develops, either on the pavement surface or within the base course, softening of the subgrade and other problems related to the deterioration of the pavement can be expected. Furthermore, good drainage should minimize the risk of the subgrade materials becoming saturated over a long period of time. The following recommendations should be considered to limit the amount of excess moisture, which can reach the subgrade soils:

- site grading at a minimum 2% grade away from the pavements;
- compaction of any utility trenches for landscaped areas to the same criteria as the pavement subgrade;
- sealing all landscaped areas in or adjacent to pavements to minimize or prevent moisture migration to subgrade soils near pavements; and,
- concrete curbs bordering landscaped areas should have a deepened edge to provide a cutoff for moisture flow beneath pavements (generally, the edge of the curb can be extended an additional twelve inches below the base of the curb).

9.1.3 Maintenance

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

9.1.4 Review and Surveillance

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

9.2 Subgrade Preparation

Grading for paved areas should be as described in Section 6.4, densifying pavement subgrade to at least 95% relative compaction after ASTM D1557 (the 'modified Proctor').

After the completion of compaction/densification, areas to receive pavements should be proof-rolled. A loaded dump truck or similar should be used to aid in identifying localized soft or unsuitable material. Any soft or unsuitable materials encountered during this proof-rolling should be removed, replaced with an approved backfill, and compacted. The Geotechnical Engineer can provide alternative options such as using geogrid and/or geotextile to stabilize the subgrade at the time of construction, if necessary.

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

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

9.3 Flexible Pavements

Provided the subgrade in paved areas is prepared per the recommendations in Section 9.2, an R-value of 30 can be assumed. Table 9-1 provides recommended sections for flexible pavements. The recommended pavement sections are for planning purposes only. Additional R-value testing should be performed on actual soils at the design subgrade levels to confirm the pavement design.

Table 9-1. Preliminary Recommendations for Flexible Pavements

Area	Assumed Subgrade R-Value	Traffic Index	Asphalt Thickness (in)	Base Course Thickness (in)
Auto Driveways/Parking	30	5.0	3.0	6.0
Roadways	30	6.0	4.0	7.0

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

9.4 Rigid Pavements

The flexible pavement specifications used in roadways and parking stalls may not be adequate for truck loading and turnaround areas, if such features are planned. In this event, NOVA recommends that a rigid concrete pavement section be provided. The pavement section should consist of 6 inches of concrete over a 6-inch base course. The aggregate base materials should also be placed at a minimum relative compaction of 95%. The concrete should be obtained from a mix design that conforms with the minimum properties shown in Table 9-2.

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

Table 9-2. Recommendations for Concrete Pavements

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

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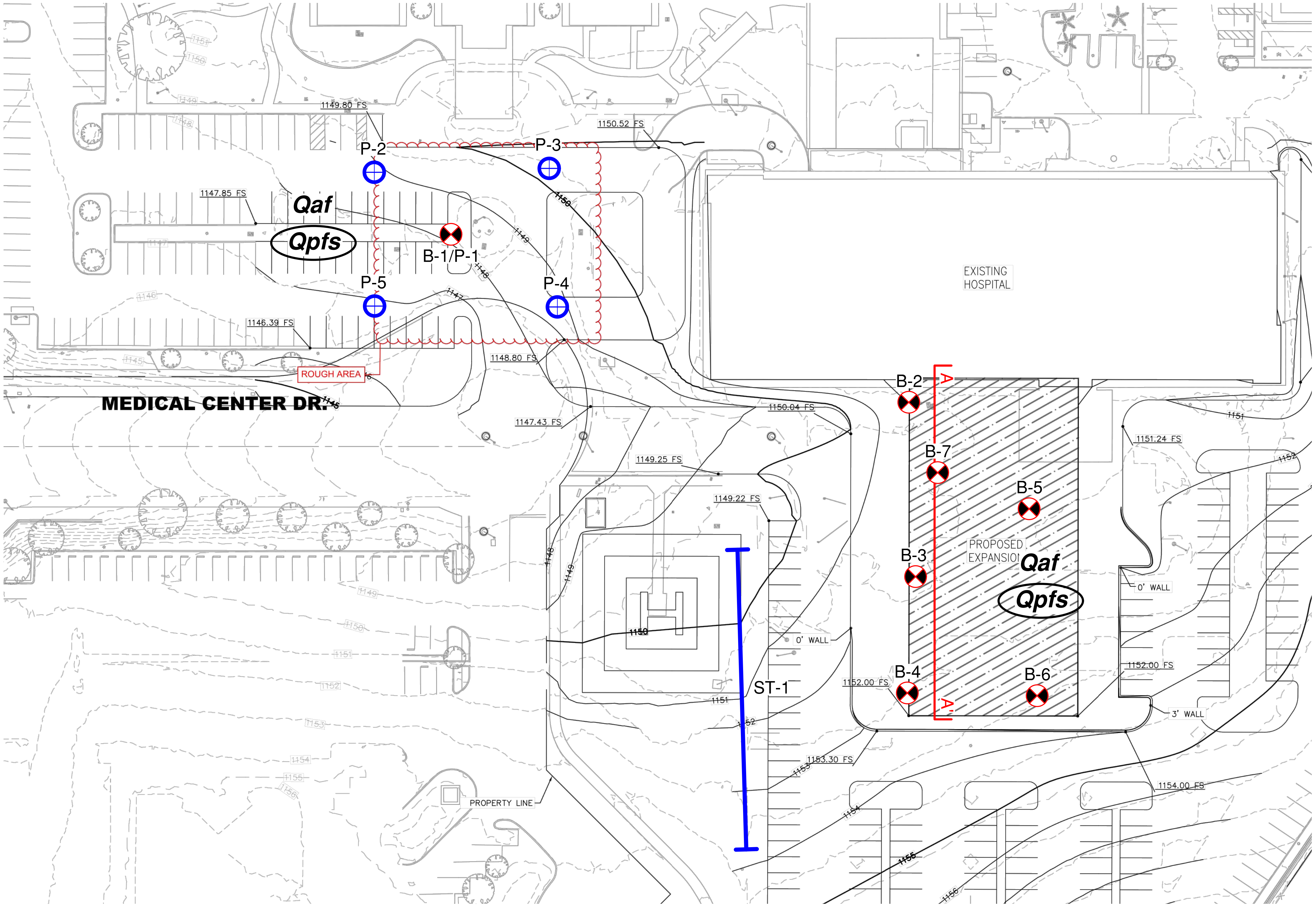
PLATES



KEY TO SYMBOLS

Qaf	FILL	B-7 ⊗	LOCATION OF GEOTECHNICAL BORING	A —— A'	GEOLOGIC CROSS-SECTION
Qpfs	PAUBA FORMATION, CIRCLED WHERE BURIED	P-5 ⊕	LOCATION OF PERCOLATION TEST BORING	ST-1	SEISMIC TRAVERSE LINE LOCATION

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PHONE: 213-261-4040

PHASE 2 ROUGH GRADING

NOVA
24632 SAN JUAN AVE, SUITE 100
DANA POINT, CALIFORNIA

(949) 388-7710
WWW.USA-NOVA.COM

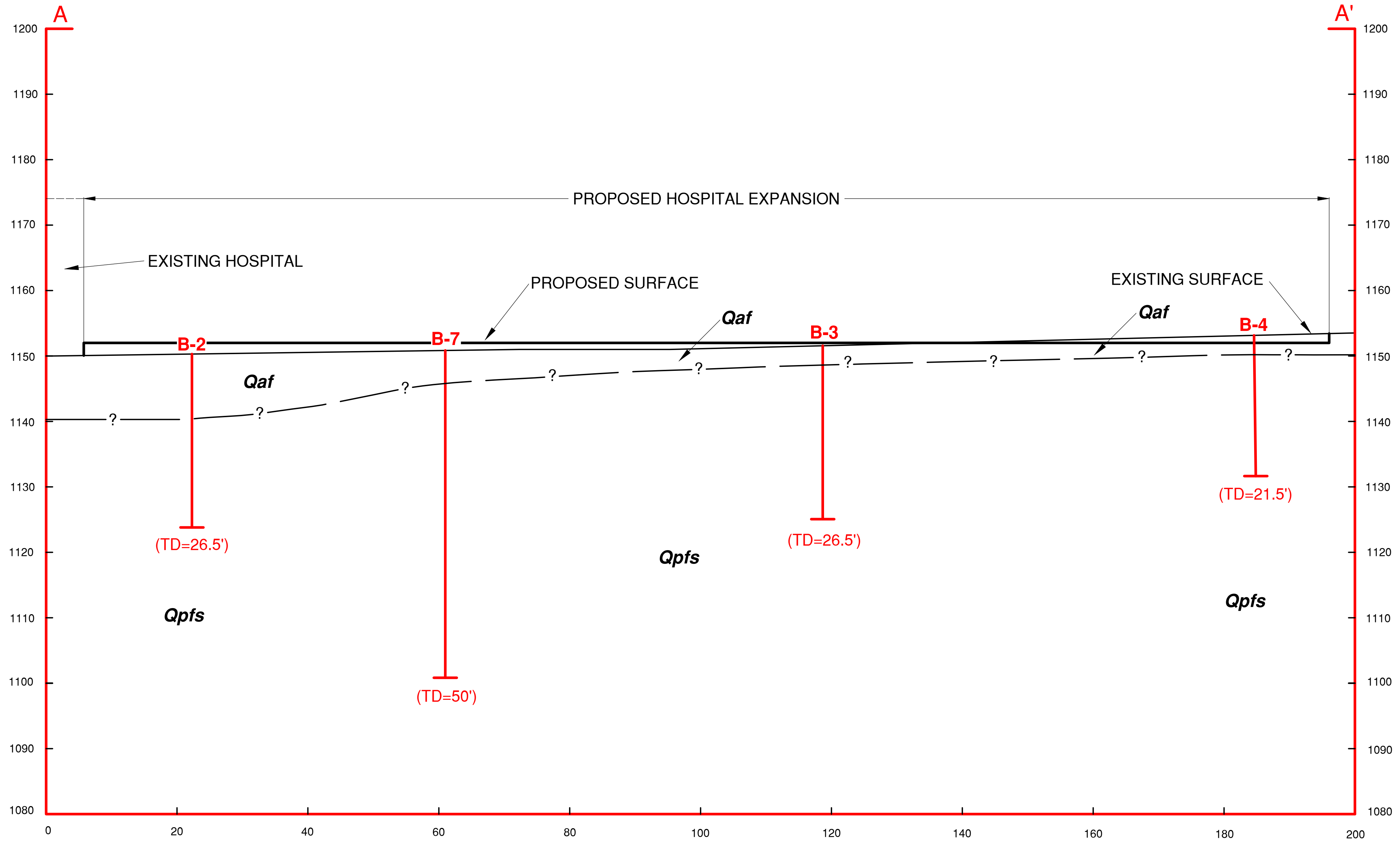
UHS MEDICAL CENTER BUILDING
25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

PROJECT NO: 3019061
DATE: DEC 2019
DRAWN BY: DTW
REVIEWED BY: JDB

SUBSURFACE
EXPLORATION MAP

0 60' 120'

PLATE 1



KEY TO SYMBOLS

Qaf	FILL
Qpfs	PAUBA FORMATION, CIRCLED WHERE BURIED
B-7	LOCATION OF GEOTECHNICAL BORING
?	GEOLOGIC CONTACT, QUERIED WHERE INFERRED

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GEOLOGIC
CROSS-SECTION AA'

0 20' 40'

PLATE 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

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

APPENDIX B

LOGS OF BORINGS

BORING LOG B-1/ P-1

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1147 FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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						**ASPHALT: 3.5 INCHES, AGGREGATE BASE: 5.5 INCHES**		
				SM		**FILL (Qaf):** SILTY SAND; LIGHT TO DARK BROWN, DAMP TO MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE GRAVEL. DECREASING GRAVEL CONTENT.		
5				SC		CLAYEY SAND; LIGHT TO DARK BROWN, MOIST, MEDIUM DENSE, TRACE IRON STAINING, TRACE GRAVEL.		
10				SM		SILTY SAND WITH CLAY; LIGHT TO DARK BROWN, MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED.		
				SM		**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; LIGHT TO DARK BROWN, MOIST, MEDIUM DENSE TO DENSE, FINE TO COARSE GRAINED.		
15						BORING TERMINATED AT 15 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH CUTTINGS. CAPPED WITH AC COLD PATCH.		
20								
25								
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

LOGGED BY:	TDT	DATE:	DEC 2019
REVIEWED BY:	JDB	PROJECT NO.:	3019061

NOVA

APPENDIX B.1

BORING LOG B-2

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1149FT MSL (GOOGLE EARTH)

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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						**ASPHALT: 4 INCHES, AGGREGATE BASE; 6 INCHES**		
				SM		**FILL (Qaf):** SILTY SAND; DARK BROWN, DAMP, LOOSE, MEDIUM TO COARSE GRAINED, TRACE IRON STAINING.		
			■		11			
5			⧄		2	VERY LOOSE.		
10			⧄	ML	26	**PAUBA FORMATION (Qpfs):** SANDY SILTSTONE; DARK BROWN, DAMP, VERY STIFF, FINE GRAINED, TRACE MICA, TRACE IRON STAINING.	SA	
15			⧄		26			
20			⧄	SM	32	SILTY SANDSTONE; LIGHT GRAY-BROWN, DAMP, VERY DENSE, MEDIUM TO COARSE GRAINED, ABUNDANT MICA, SOME IRON STAINING.		
25			⧄		57	SCATTERED MICA.		
						BORING TERMINATED AT 26.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH BORING CUTTINGS. CAPPED WITH AC COLD PATCH.		
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE

MURRIETA, CALIFORNIA

NOVA

LOGGED BY:	TDT	DATE:	DEC 2019
REVIEWED BY:	JDB	PROJECT NO.:	3019061

APPENDIX B.2

BORING LOG B-3

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1149FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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						**ASPHALT: 4 INCHES, AGGREGATE BASE; 6 INCHES**		
				ML		**FILL (Qaf):** SANDY SILT; DARK BROWN, DRY TO DAMP, HARD, FINE TO MEDIUM GRAINED, SCATTERED MICA.		
			▨		69			
				SM		**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; DARK BROWN, DAMP, VERY DENSE, FINE GRAINED, TRACE MICA, TRACE IRON STAINING.		
5			▨		>70#			
10			⧄		37	SANDY SILT INTERBED.		
15			▨		50/ 4"	LIGHT BROWN, DRY TO DAMP, VERY DENSE, FINE TO MEDIUM GRAINED, TRACE MICA.		
20			⧄		40			
25			⧄		39	SILTY SANDSTONE; LIGHT GRAY-BROWN, DAMP, VERY DENSE, MEDIUM TO COARSE GRAINED, SCATTERED MICA, SOME IRON STAINING.		
						BORING TERMINATED AT 26.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH BORING CUTTINGS. CAPPED WITH AC COLD PATCH.		
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE

MURRIETA, CALIFORNIA

LOGGED BY:	TDT	DATE:	DEC 2019
REVIEWED BY:	JDB	PROJECT NO.: 3019061	

NOVA

APPENDIX B.3

BORING LOG B-4

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1151FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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						**ASPHALT: 4 INCHES, AGGREGATE BASE; 6 INCHES**		
				SM		**FILL (Qaf):** SILTY SAND; LIGHT TO DARK BROWN, DAMP, DENSE, FINE TO MEDIUM GRAINED, SCATTERED MICA, TRACE IRON STAINING.		
			■		48			
				SM		**PAUBA FORMATION (Qpfs):** SIL;TY SANDSTONE; BROWN, DAMP TO MOIST, VERY DENSE, FINE GRAINED, TRACE MICA, SCATTERED IRON STAINING, SILTSTONE INTERBEDS.		
5			⧄		20			
10			⧄		35	LIGHT BROWN TO LIGHT GRAY, FINE TO COARSE GRAINED, SOME IRON STAINING, SOME MICA.		
15			⧄		37	LIGHT BROWN, DRY TO DAMP, VERY DENSE, FINE TO MEDIUM GRAINED, TRACE MICA.		
20			⧄		42			
						BORING TERMINATED AT 21.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH BORING CUTTINGS. CAPPED WITH AC COLD PATCH.		
25								
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

NOVA

LOGGED BY:	TDT	DATE:	DEC 2019
REVIEWED BY:	JDB	PROJECT NO.:	3019061

APPENDIX B.4

BORING LOG B-5

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1151FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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						**ASPHALT: 4 INCHES, AGGREGATE BASE; 6 INCHES**		
				ML		**FILL (Qaf):** SANDY SILT; RED BROWN, DAMP, HARD, FINE GRAINED, TRACE COARSE GRAINS, SCATTERED MICA, TRACE IRON STAINING.		
			■		57#		SA	117.1 PCF, @ 9.9%
5			⧄		8	FINE GRAINED, ABUNDANT MICA.		
				SM		**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; BROWN, DAMP TO MOIST, VERY DENSE, FINE GRAINED, TRACE MICA, SCATTERED IRON STAINING, SILTSTONE INTERBEDS.		
10			⧄		22	SANDY SILT INTERBEDS; RED BROWN, DAMP STIFF, FINE GRAINED, SCATTERED MICA, TRACE IRON STAINING.		
15			⧄		30			
20			⧄		44	LIGHT GRAY TO LIGHT BROWN, DAMP, MEDIUM DENSE, FINE TO MEDIUM GRAINED WITH VERY FINE SAND, ABUNDANT MICA, TRACE IRON STAINING.		
						BORING TERMINATED AT 21.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH BORING CUTTINGS. CAPPED WITH AC COLD PATCH.		
25								
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

NOVA

LOGGED BY: TDT
DATE: DEC 2019
REVIEWED BY: JDB
PROJECT NO.: 3019061

APPENDIX B.5

BORING LOG B-6

DATE EXCAVATED: AUGUST 19, 2019
EQUIPMENT: CME 75 DRILL RIG
EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING
GPS COORD.:
GROUNDWATER DEPTH: NOT ENCOUNTERED
ELEVATION: ± 1151FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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						**ASPHALT: 4 INCHES, AGGREGATE BASE; 6 INCHES**		
				SM		**FILL (Qaf):** SANDY SILT; RED BROWN, DAMP, VERY STIFF, FINE TO MEDIUM GRAINED, TRACE MICA.		
			■		36			
5			⧄		16			
				SM		**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; LIGHT GRAY, DAMP, DENSE, FINE GRAINED, SOME MICA, SILTY INTERBEDS.		
10			⧄		41			
15			⧄		36	FINE TO MEDIUM GRAINED, SCATTERED MICA.		
20			⧄	SW	44	WELL GRADED SANDSTONE; LIGHT TO DARK GRAY, DAMP, VERY DENSE, FINE TO COARSE GRAINED, SCATTERED MICA, TRACE IRON STAINING, TRACE GRAVEL.		
						BORING TERMINATED AT 21.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH BORING CUTTINGS. CAPPED WITH AC COLD PATCH.		
25								
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

NOVA

LOGGED BY: TDT
DATE: DEC 2019
REVIEWED BY: JDB
PROJECT NO.: 3019061

BORING LOG B-7

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

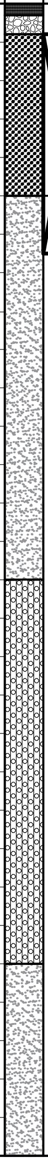
GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1150FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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



DEPTH (FT)	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					**ASPHALT: 4 INCHES, AGGREGATE BASE; 6 INCHES**		
	☒		SC		**FILL (Qaf):** CLAYEY SAND; BROWN, DAMP, LOOSE, FINE TO COARSE GRAINED, SCATTERED MICA, TRACE GRAVEL.	MD CR RV	131.2 PCF, @ 8.4% SO_4 = 0.022% (220 PPM) R-VALUE = 25
		■		8			
5		■	SM	28	**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; LIGHT GRAY, MOIST, MEDIUM DENSE, FINE GRAINED, SOME MICA, SILTSTONE INTERBEDS.		116.5 PCF, @ 14.7%
10		⧄		20	ABUNDANT MICA.		
15		■	ML	56	SANDY SILTSTONE; OLIVE BROWN, MOIST, VERY DENSE, SCATTERED TO SOME MICA, SCATTERED IRON STAINING.		117.2 PCF, @ 15.9%
20		⧄		30	DENSE, TRACE IRON STAINING.	SA	
25		■	SM	>70	LIGHT BROWN TO LIGHT GRAY, DAMP, VERY DENSE, FINE TO COARSE GRAINED, SCATTERED MICA.		
30							

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE

MURRIETA, CALIFORNIA

LOGGED BY: TDT

DATE: DEC 2019

REVIEWED BY: JDB

PROJECT NO.: 3019061

NOVA

APPENDIX B.7

CONTINUED BORING LOG B-7

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1150 FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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			⧄	SW	47	WELL GRADED SANDSTONE; LIGHT TO DARK GRAY, DENSE, FINE TO COARSE GRAINED, TRACE FINE GRAVEL, TRACE IRON STAINING.		
35			■		>70	VERY DENSE, FINE TO MEDIUM GRAINED, SOME MICA.		
40			⧄		>50			
45			■		>70	FINE TO COARSE GRAINED,		
			⧄		>70			
50						BORING TERMINATED AT 50 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH BORING CUTTINGS.		
55								
60								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

NOVA

LOGGED BY: TDT

DATE: DEC 2019

REVIEWED BY: JDB

PROJECT NO.: 3019061

APPENDIX B.8

BORING LOG B-1/ P-1

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1147 FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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						**ASPHALT: 3.5 INCHES, AGGREGATE BASE: 5.5 INCHES**		
				SM		**FILL (Qaf):** SILTY SAND; LIGHT TO DARK BROWN, DAMP TO MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE GRAVEL. DECREASING GRAVEL CONTENT.		
5				SC		CLAYEY SAND; LIGHT TO DARK BROWN, MOIST, MEDIUM DENSE, TRACE IRON STAINING, TRACE GRAVEL.		
10				SM		SILTY SAND WITH CLAY; LIGHT TO DARK BROWN, MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED.		
				SM		**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; LIGHT TO DARK BROWN, MOIST, MEDIUM DENSE TO DENSE, FINE TO COARSE GRAINED.		
15						BORING TERMINATED AT 15 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH CUTTINGS. CAPPED WITH AC COLD PATCH.		
20								
25								
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

LOGGED BY: TDT | DATE: DEC 2019

REVIEWED BY: JDB | PROJECT NO.: 3019061

NOVA

APPENDIX B.9

BORING LOG P-2

DATE EXCAVATED: AUGUST 19, 2019
EQUIPMENT: CME 75 DRILL RIG
EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING
GPS COORD.:
GROUNDWATER DEPTH: NOT ENCOUNTERED
ELEVATION: ± 1148 FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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						**ASPHALT: 4 INCHES, AGGREGATE BASE: 6 INCHES**		
				SC		**FILL (Qaf):** CLAYEY SAND; RED BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, TRACE MICA.		
5								
				SM		**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; LIGHT TO DARK BROWN, MOIST, MEDIUM DENSE TO DENSE, FINE TO COARSE GRAINED.		
10						BORING TERMINATED AT 10 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH CUTTINGS. CAPPED WITH AC COLD PATCH.		
15								
20								
25								
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

LOGGED BY:	TDT	DATE:	DEC 2019
REVIEWED BY:	JDB	PROJECT NO.:	3019061

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APPENDIX B.10

BORING LOG P-3

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

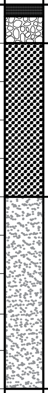
GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1148 FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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



DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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						**ASPHALT: 4 INCHES, AGGREGATE BASE: 10 INCHES**		
				SC		**FILL (Qaf):** CLAYEY SAND; RED BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, TRACE MICA.		
5				SM		**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; LIGHT TO DARK BROWN, MOIST, MEDIUM DENSE TO DENSE, FINE TO COARSE GRAINED.		
10						BORING TERMINATED AT 10 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH CUTTINGS. CAPPED WITH AC COLD PATCH.		
15								
20								
25								
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

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LOGGED BY:	TDT	DATE:	DEC 2019
REVIEWED BY:	JDB	PROJECT NO.: 3019061	APPENDIX B.11

BORING LOG P-4

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1147 FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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 5	▩			SC		**FILL (Qaf):** CLAYEY SAND; BROWN TO RED BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, TRACE MICA, TRACE GRAVEL.		
10	░			SM		**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; LIGHT BROWN, MOIST, MEDIUM DENSE TO DENSE, FINE TO MEDIUM GRAINED, SCATTERED COARSE GRAINS.		
15 20 25 30						BORING TERMINATED AT 11 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH CUTTINGS.		

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE

MURRIETA, CALIFORNIA

LOGGED BY:	TDT	DATE:	DEC 2019
REVIEWED BY:	JDB	PROJECT NO.:	3019061

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APPENDIX B.12

BORING LOG P-5

DATE EXCAVATED: AUGUST 19, 2019

EQUIPMENT: CME 75 DRILL RIG

EXCAVATION DESCRIPTION: 8 INCH DIAMETER AUGER BORING

GPS COORD.:

GROUNDWATER DEPTH: NOT ENCOUNTERED

ELEVATION: ± 1147 FT MSL (GOOGLE EARTH)

LAB TEST ABBREVIATIONS

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

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	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	▩			SC		**FILL (Qaf):** CLAYEY SAND; LIGHT TO DARK BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, TRACE MICA.		
5	▩							
	▒			SM		**PAUBA FORMATION (Qpfs):** SILTY SANDSTONE; LIGHT TO DARK BROWN, MOIST, MEDIUM DENSE TO DENSE, FINE TO MEDIUM GRAINED.		
10						BORING TERMINATED AT 10 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH CUTTINGS.		
15								
20								
25								
30								

KEY TO SYMBOLS

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

25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

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LOGGED BY:	TDT	DATE:	DEC 2019
REVIEWED BY:	JDB	PROJECT NO.:	3019061

APPENDIX B.13

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 in the exploration logs.*
- **DENSITY OF SOIL IN PLACE (ASTM D2937):** *In-place moisture contents and dry densities were determined for representative soil samples. This information was an aid to classification and permitted recognition of variations in material consistency with depth. The dry unit weight is determined in pounds per cubic foot, and the in-place moisture content is determined as a percentage of the soil's dry weight. The results are summarized in the exploration logs.*
- **MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT (ASTM D1557 METHOD A,B,C):** *The maximum dry density and optimum moisture content of typical soils were determined in the laboratory in accordance with ASTM Standard Test D1557, Method A, Method B, Method C.*
- **CORROSIVITY TEST (CAL. TEST METHOD 417, 422, 643):** *Soil PH, and minimum resistivity tests were performed on a representative soil sample in general accordance with test method CT 643. The sulfate and chloride content of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively.*
- **DIRECT SHEAR (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.*
- **R-VALUE (ASTM D2844):** *The resistance Value, or R-Value, for near-surface site soils were evaluated in general accordance with California Test (CT) 301 and ASTM D2844. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results.*
- **GRADATION ANALYSIS (ASTM C136 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 C136 and/or ASTM D422.*

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LAB TEST SUMMARY

UHS RANCHO SPRINGS MEDICAL CENTER RENOVATION
25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

BY: DTW
DATE: DECEMBER 2019
PROJECT: 3019061

APPENDIX C

LABORATORY ANALYTICAL RESULTS

Density of Soil in Place (ASTM D2937)

Sample Location	Sample Depth (ft)	Moisture (%)	Dry Density (pcf)
B-5	2.5'-4.0'	9.9	117.1
B-7	5.0'-6.5'	14.7	116.5
B-7	15.0'-16.5'	15.9	117.2

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

Sample Location	Sample Depth (ft.)	R-Value
B-7	0.0'-5.0'	25

Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

Sample Location	Sample Depth (ft.)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-7	0.0'-5.0'	131.2	8.4

Direct Shear (ASTM D3080)

Sample Location	Depth (feet)	Soil Description	Peak Friction Angle (degrees)	Peak Apparent Cohesion (psf)
B-7	0.0'-5.0'	Brown Clayey Sand	34°	757

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

Sample Location	Sample Depth (ft.)	pH	Resistivity (Ohm-cm)	Sulfate Content		Chloride Content	
				(ppm)	(%)	(ppm)	(%)
B-7	0.0'-5.0'	8.3	1300	220	0.022	75	0.007

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LAB TEST RESULTS

UHS RANCHO SPRINGS MEDICAL CENTER RENOVATION

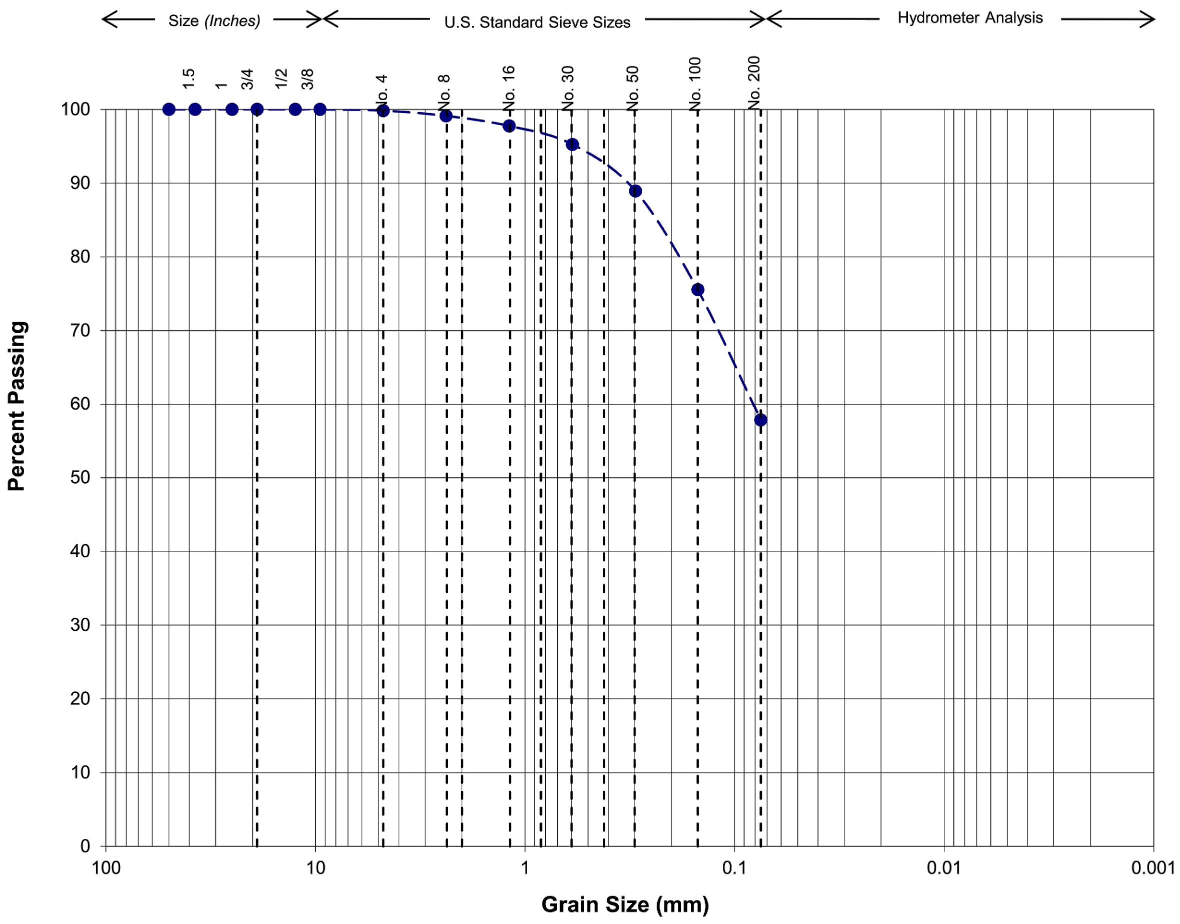
25500 MEDICAL CENTER DRIVE

MURRIETA, CALIFORNIA

BY: DTW

DATE: DECEMBER 2019

PROJECT: 3019061



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-2

Depth (ft): 10.0'-11.5' & 15.0'-16.5' (COMBINED)

USCS Soil Type: ML

Passing No. 200 (%): 58

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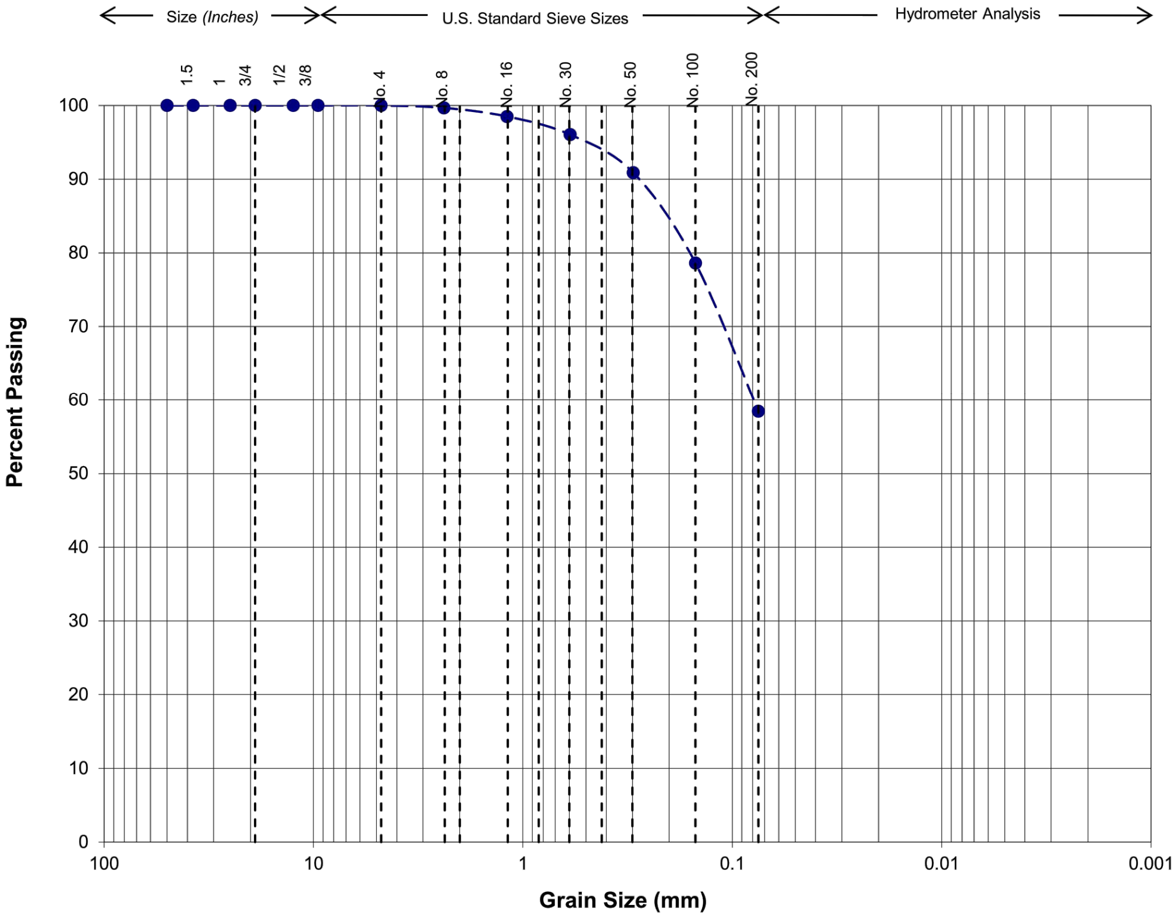
GRADATION ANALYSIS TEST RESULTS

UHS RANCHO SPRINGS MEDICAL CENTER RENOVATION
25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

BY: DTW
DATE: DECEMBER 2019
PROJECT: 3019061



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-5

Depth (ft): 2.5'-4.0'

USCS Soil Type: ML

Passing No. 200 (%): 58

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GRADATION ANALYSIS TEST RESULTS

UHS RANCHO SPRINGS MEDICAL CENTER RENOVATION

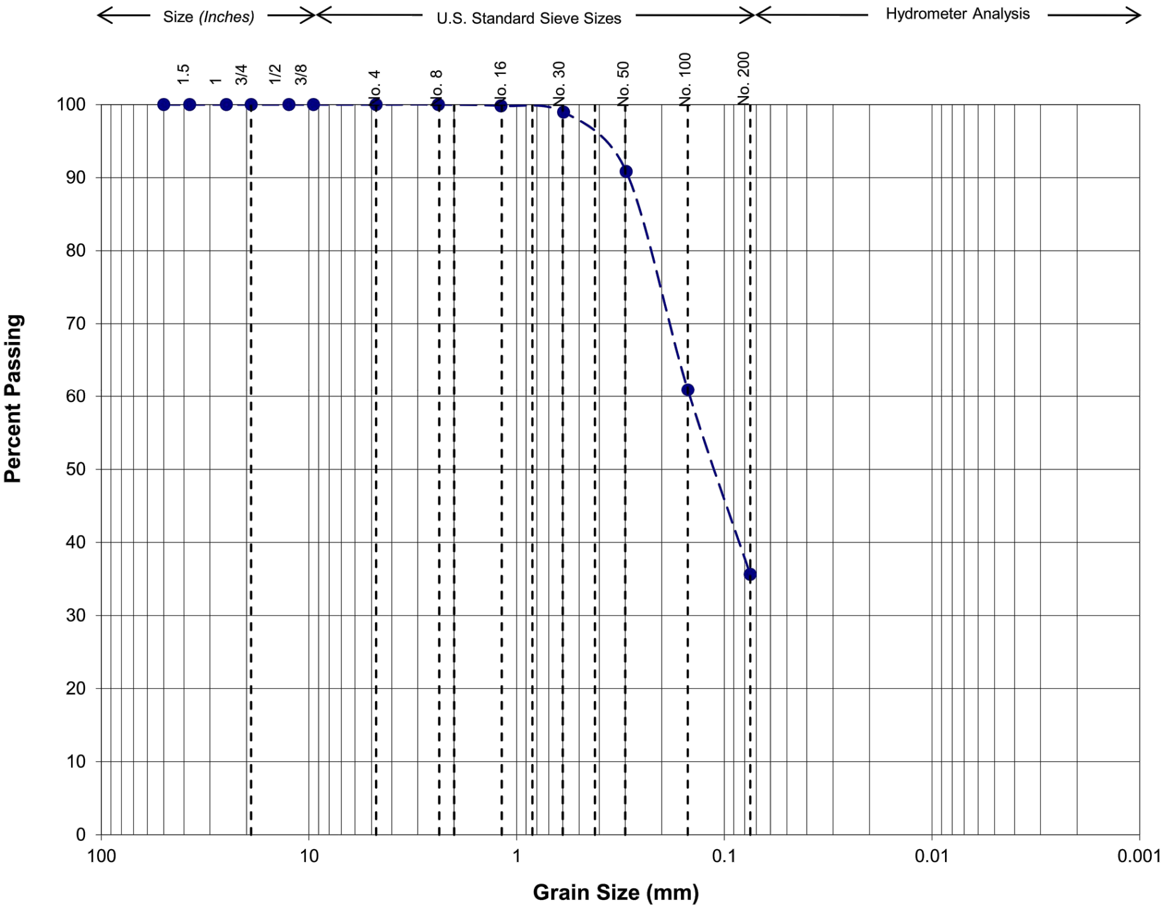
25500 MEDICAL CENTER DRIVE

MURRIETA, CALIFORNIA

BY: DTW

DATE: DECEMBER 2019

PROJECT: 3019061



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-7

Depth (ft): 20.0'-21.5'

USCS Soil Type: SM

Passing No. 200 (%): 36

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GRADATION ANALYSIS TEST RESULTS

UHS RANCHO SPRINGS MEDICAL CENTER RENOVATION
25500 MEDICAL CENTER DRIVE
MURRIETA, CALIFORNIA

BY: DTW

DATE: DECEMBER 2019

PROJECT: 3019061

APPENDIX D

STORMWATER INFILTRATION

PERCOLATION TEST DATA SHEET P - 1

Project:	25500 Med Ctr Road	Project No:	3019061	Date:	20-Aug-19
Test Hole No: P- **1**		Tested By:	Tim Taverentti		
Depth of test Hole:	15' (180")	USCS Soil Classification: Silty Sand with Clay (SM)			
Test Hole Dimensions (inches)				Length	Width
Diameter (if round) =	8	Sides (if rectangular) =			

Sandy Soil Criteria Test*

Trail No.	Start Time	Stop Time	Time Interval (min.)	Intital Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1							
2							

* If two consecutive measurements show that six inches of water seps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at lease 0.25".

Trail No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (ft)	Final Depth to Water (ft)	Change in Water Level (in)	Percolation Rate (min/ in)
1	8:30	9:00	30	6.35	7.15	9.60	3.13
2	9:15	9:46	31	7.35	11.25	46.80	0.66
3	9:54	10:29	35	6.00	10.26	51.12	0.68
4	10:30	10:55	25	6.00	9.95	47.40	0.53

NOTE: Location appears to be within an existing utility trench. Testing halted after 2.5 hours.

PERCOLATION TEST DATA SHEET

P - __2___

Project:	25500 Med Ctr Road	Project No:	3019061	Date:	20-Aug-19
Test Hole No: P - 2		Tested By:	Tim Tavernetti		
Depth of test Hole:	10' (120")	USCS Soil Classification: Silty Sand (SM)			
Test Hole Dimensions (inches)				Length	Width
Diameter (if round) =	8	Sides (if rectangular) =			

Sandy Soil Criteria Test*

Trail No.	Start Time	Stop Time	Time Interval (min.)	Intital Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1							
2							

* If two consecutive measurements show that six inches of water seps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at lease 0.25".

Trail No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (ft)	Final Depth to Water (ft)	Change in Water Level (in)	Percolation Rate (min/ in)
1	8:43	9:00	17	4.05	4.71	7.92	2.15
2	9:20	9:46	35	3.85	4.55	8.40	4.17
3	9:58	10:29	34	3.75	4.46	8.52	3.99
4	10:34	10:55	23	3.70	4.24	6.48	3.55
5	10:59	11:23	24	3.60	4.15	6.60	3.64
6	11:25	11:57	32	3.72	4.32	7.20	4.44
7	11:59	12:28	29	3.90	4.40	6.00	4.83
8	12:30	13:01	31	4.00	4.52	6.24	4.97
9	13:04	13:31	27	3.94	4.41	5.64	4.79
10	13:32	14:07	35	3.89	4.70	9.72	3.60
11	14:09	14:39	30	3.90	4.40	6.00	5.00

PERCOLATION TEST DATA SHEET P - 3

Project:	25500 Med Ctr Road	Project No:	3019061	Date:	20-Aug-19
Test Hole No: P - 3		Tested By:	Tim Tavernetti		
Depth of test Hole:	10' (120")	USCS Soil Classification: Silty Sand (SM)			
Test Hole Dimensions (inches)				Length	Width
Diameter (if round) =	8	Sides (if rectangular) =			

Sandy Soil Criteria Test*

Trail No.	Start Time	Stop Time	Time Interval (min.)	Intital Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1							
2							

* If two consecutive measurements show that six inches of water seps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at lease 0.25".

Trail No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (ft)	Final Depth to Water (ft)	Change in Water Level (in)	Percolation Rate (min/ in)
1	8:50	9:21	31	5.65	6.00	4.20	7.38
2	9:25	10:00	35	4.30	4.75	5.40	6.48
3	10:03	10:36	33	4.05	4.72	8.04	4.10
4	10:39	11:01	22	4.26	4.85	7.08	3.11
5	11:02	11:32	30	3.32	4.11	9.48	3.16
6	11:34	12:02	28	4.45	5.16	8.52	3.29
7	12:04	12:38	34	4.30	5.00	8.40	4.05
8	12:42	13:05	23	4.20	4.77	6.84	3.36
9	13:07	13:38	31	4.10	4.93	9.96	3.11
10	13:39	10:09	30	4.25	4.95	8.40	3.57
11	14:10	14:40	30	4.20	4.85	7.80	3.85

PERCOLATION TEST DATA SHEET P - __4___

Project:	25500 Med Ctr Road	Project No:	3019061	Date:	20-Aug-19
Test Hole No: P - 4		Tested By:	Tim Tavernetti		
Depth of test Hole:	11' (132")	USCS Soil Classification: Silty Sand (SM)			
Test Hole Dimensions (inches)				Length	Width
Diameter (if round) =	8	Sides (if rectangular) =			

Sandy Soil Criteria Test*

Trail No.	Start Time	Stop Time	Time Interval (min.)	Intital Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1							
2							

* If two consecutive measurements show that six inches of water seps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at lease 0.25".

Trail No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (ft)	Final Depth to Water (ft)	Change in Water Level (in)	Percolation Rate (min/ in)
1	8:50	9:27	32	2.95	5.40	29.40	1.09
2	9:32	10:03	31	2.11	5.20	37.08	0.84
3	10:05	10:41	36	4.25	5.85	19.20	1.88
4	10:44	11:14	30	4.40	5.77	16.44	1.82
5	11:17	11:48	31	4.40	5.80	16.80	1.85
6	11:50	12:20	30	4.40	5.71	15.72	1.91
7	12:23	12:53	30	5.22	6.12	10.80	2.78
8	12:54	13:18	24	5.15	6.20	12.60	1.90
9	13:23	13:51	28	4.50	5.57	12.84	2.18
10	13:52	14:20	28	4.80	5.90	13.20	2.12
11	14:22	14:52	30	4.60	5.80	14.40	2.08

PERCOLATION TEST DATA SHEET P - __5___

Project:	25500 Med Ctr Road	Project No:	3019061	Date:	20-Aug-19
Test Hole No: P - 5		Tested By:	Tim Tavernetti		
Depth of test Hole:	10' (120")	USCS Soil Classification: Silty Sand (SM)			
Test Hole Dimensions (inches)				Length	Width
Diameter (if round) =	8	Sides (if rectangular) =			

Sandy Soil Criteria Test*

Trail No.	Start Time	Stop Time	Time Interval (min.)	Intital Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1							
2							

* If two consecutive measurements show that six inches of water seps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at lease 0.25".

Trail No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (ft)	Final Depth to Water (ft)	Change in Water Level (in)	Percolation Rate (min/ in)
1	8:37	9:08	31	5.15	5.25	1.20	25.83
2	9:10	9:43	33	3.70	4.15	5.40	6.11
3	9:45	10:20	35	3.90	4.4	6.00	5.83
4	10:22	10:50	38	2.25	3.65	16.80	2.26
5	10:53	11:19	26	2.30	3.15	10.20	2.55
6	11:21	11:51	30	2.40	2.95	6.60	4.55
7	11:53	12:24	29	2.35	2.81	5.52	5.25
8	12:26	12:52	26	2.33	2.75	5.04	5.16
9	12:53	13:24	31	2.11	2.7	7.08	4.38
10	13:29	14:08	39	2.15	2.60	5.40	7.22
11	14:06	14:37	31	1.50	2.30	9.60	3.23